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November 16, 2009
File No.: 04.02.18.02
Project No. 357891

Paul Kramer
California Energy Commission
Systems Assessment and Facilities Siting Division
1516 9th Street
Sacramento, CA 95814-5504

RE: Applicants Testimony
Ivanpah Solar Electric Generating System (07-AFC-5)

Dear Mr. Kramer:

On behalf of Solar Partners I, LLC, Solar Partners II, LLC, Solar Partners IV, LLC, and Solar Partners VIII, LLC, please find attached five copies of the Applicant's Testimony.

Please call me if you have any questions.

Sincerely,

CH2M HILL

John L. Carrier

John L. Carrier, J.D.
Program Manager

Enclosure
c: POS List
Project File

Ivanpah Solar Electric Generating System (ISEGS)

(07-AFC-5)

Applicant's Testimony

Submitted to the

California Energy Commission

Submitted by

Solar Partners I, LLC; Solar Partners II, LLC; Solar Partners IV, LLC; and Solar Partners VIII, LLC

November 16, 2009

With Assistance from

CH2MHILL
2485 Natomas Park Drive
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Contents

Project Description

(Including FSA/DEIS Sections: Executive Summary, Introduction, Project Description, Facility Design, Power Plant Efficiency, Power Plant Reliability and General Conditions)

I. Introduction

A. Name: Steve De Young, John Woolard, Todd Stewart, Tom Reagan and John Carrier

B. Qualifications: The panel's qualifications are as noted in their resumes contained in Appendix A.

C. Prior Filings: In addition to the statements herein, this testimony includes by reference the following documents submitted in this proceeding:

- Application for Certification (Vol. 1) [Exhibit 1]
- Applicant's Response to CEC Staff Requests, Data Response Set 1A, dated January 14, 2008. Responses to Data Requests 1 through 6 [Exhibit 4].
- Applicant's Response to CEC Staff Requests, Data Response Set 1B, dated February 11, 2008. Responses to Data Request 5 and 6 [Exhibit 5].
- Applicant's Response to CEC Staff Requests, Data Response Set 1D, dated May 9, 2008. Responses to Data Requests 4 and 6 [Exhibit 7].
- Applicant's Response to CEC Staff Requests, Data Response Set 2A, dated June 10, 2008. Responses to Data Requests 130 through 132 [Exhibit 20].
- Applicant's Response to CEC Staff Requests, Data Response Set 2B, dated July 22, 2008. Responses to Data Request 131 [Exhibit 21].
- Applicant's Response to CEC Staff Requests, Data Response Set 2I, dated May 18, 2009, Responses to Data Requests 130 and 131 [Exhibit 28]
- Applicant's Response to CEC Staff Requests, Data Response Set 2J, dated June 17, 2009, Responses to Data Request 131 [Exhibit 29].
- Applicant's Response to CEC Staff Requests, Data Response Set 2K, dated June 30, 2009, Responses to Data Requests 125 [Exhibit 30].
- Applicant's Response to CEC Staff Requests, Data Response Set 2KR, dated September 10, 2009, Responses to Data Requests 125 [Exhibit 31].

To the best of our knowledge, all of the facts contained in this testimony (including all referenced documents) are true and correct. To the extent this testimony contains opinions, such opinions are our own. We make these statements, and render these opinions freely and under oath for the purpose of constituting sworn testimony in this proceeding.

II. Summary of Testimony

A. Opening Statement of John Woolard, President and CEO of BrightSource Energy

I, John M. Woolard, serve as President and Chief Executive Officer of BrightSource Energy. In this role, I bring two decades of experience in the energy and environmental sectors as an executive, entrepreneur and investor. Prior to joining BrightSource Energy, I co-founded Silicon Energy and was President, Chief Executive Officer and Chairman of the Board from 1997 to 2003. I joined the executive team at Itron, Inc. in 2003 following its acquisition of Silicon Energy, and was Vice President of Software Solutions and subsequently the Vice President of Strategy and Business Development. I previously held positions with VantagePoint Venture Partner's CleanTech Group, Lawrence Berkeley National Labs, and Pacific Gas & Electric Company.

My academic background includes a Master of Business Administration degree from the Haas School of Business at U.C. Berkeley, as well as a Master in Environmental Planning and a Bachelor of Arts degree from the University of Virginia. I am an Aspen Institute Crown Fellow, serve on the Tuolumne River Trust Advisory Committee and the East Bay Zoological Society Board of Trustees, and am a Life Member of the Sierra Club.

I am very pleased to have this opportunity to provide testimony in regards to the permitting of the Ivanpah Solar Energy Generating System (Ivanpah SEGS). The following testimony first outlines the broader policy context that led us to form BrightSource Energy, and for BrightSource to initiate the Ivanpah SEGS project. My testimony then discusses the vital role that the project will play in helping our state meet its environmental, energy and economic goals. Lastly, my testimony addresses the importance of the Ivanpah SEGS project to our company, and to making it possible for our technology, which leads the solar industry not only in performance and cost-effectiveness but in efficiency using dry-cooling, to serve California's goals at other sites as well.

I'd like to begin my testimony by expressing my thanks to the staff of both the California Energy Commission (CEC) and the U.S. Bureau of Land Management (BLM) for their efforts to date. By drafting the joint Final Staff Assessment / Draft Environmental Impact Statement (FSA/DEIS), the staff of both agencies have contributed a significant step towards delivering the clean, renewable energy our economy and our environment so desperately need. The joint FSA/DEIS for the Ivanpah SEGS is the first of its kind, and marks a critically-important milestone in the construction of California's first utility-scale solar energy project in nearly two decades.

My career in energy, and at BrightSource Energy, is rooted in my academic background, beginning with my environmental planning degree. While studying for a Master's in Environmental Planning at the University of Virginia, two things became clear to me: one, that carbon accumulation in the atmosphere was the single biggest problem we face as stewards and inhabitants of this planet, and two, that our greatest impact on carbon emissions comes from the way we produce and consume energy.

I have since dedicated my career to finding clean energy solutions that reduce our dependence on fossil fuels and that help us avoid further impact to the environment, including species extinction. In 1998, I started Silicon Energy – one of the nation's leading providers of energy efficiency and demand side management software. We helped our customers save more than three gigawatts of power for the country - equivalent to the annual production of two to three nuclear power plants – before the company was acquired by another entity, Itron, where I continued to work for two years.

Following my work with Silicon Valley Energy and Itron, I spent six months at the Lawrence Berkeley National Laboratory informally studying the relationships between climate and energy. During this period I was able to spend time with leading scientists, economists, and public policy experts in energy efficiency, solar and wind energy– and to remove myself from the daily business cycle in order to reflect, strategize, and plan. I emerged from my time at the Berkeley Labs focused on solar energy, and its particularly important role in the challenge of de-carbonizing our power plants at a scale that would have a meaningful impact on global warming.

The challenge the world faces is immense. According to the International Energy Agency, to stabilize CO_2 in the atmosphere at 450 ppm - the consensus target adopted by the scientific community –we will need to build the equivalent of 4,900 gigawatts of new carbon free power plants over the next 20 years.[i] That's 245 new carbon free power plants, each the size of a nuclear plant, every year.

The data is clear – we will only be able to address climate change if we build renewables at scale. This is not to say that we should not invest in energy efficiency and distributed renewable generation. From my work at Silicon Valley Energy providing gigawatts in energy savings, I can testify to the virtues of energy efficiency with first-hand experience. California has correctly made energy efficiency our highest priority resource in meeting our clean energy goals. Distributed renewable energy sources, such as rooftop solar, also have an important role, and deserve significant resources. Yet even if we run the table and implement energy efficiency and rooftop solar to the maximum extent reasonably practicable, we still need to build thousands of gigawatts of utility-scale renewable plants to stabilize CO_2 in the atmosphere at 450 ppm.

Our state's and nation's policymakers are responding to this challenge. Governor Schwarzenegger recently signed an Executive Order requiring California's utilities to obtain one third of their energy from renewable resources. Interior Secretary Salazar has joined Governor Schwarzenegger in a recent Memorandum of Understanding to prioritize renewable energy, coordinating their efforts and dedicating resources and attention to this important issue. At the national level, Congress is debating legislation imposing a national renewable energy mandate, as well as other climate change measures.

Meeting these policy challenges will require an investment in a diverse portfolio of renewable energy resources. With the right infrastructure in place, our state and national systems will enjoy a reliable mix of wind, geothermal, hydroelectric, and solar power with a minimum of conventional power plants. Utility-scale solar is a keystone to this renewable energy mix, providing quantities of power at peak, and complementing the production profiles of wind and other resources.

California in particular is blessed with an abundance of the high quality sun necessary to develop cost-effective utility-scale solar power. Our state's solar resources are also ideally located close to large population centers, where demand for solar power during the day is at its peak. Foregoing this resource would do irreparable damage to our ability to meet the requirements of California's landmark renewable energy and climate laws. Put simply: we will fail to meet our goals if we do not build utility-scale solar.

BrightSource Energy was founded on the idea that combining California and the U.S. southwest's unique solar characteristics with advanced and environmentally-responsible utility-scale solar technology could reliably deliver cost-effective, clean energy to one of the biggest energy markets in the world. Our technology represents a major step forward for the solar thermal industry, yet is built on the solid foundation of our technical team's experience building the nine solar thermal SEGS plants in California during the 1980's and early 1990's—which include the largest solar thermal project built anywhere in the world to date.

The BrightSource Energy Luz Power Tower 550 (LPT 550) technology has been proven at our demonstration facility in Israel. This technology is producing the world's highest temperature steam for solar energy, and has been validated by an independent engineering firm. Our technology employs dry-cooling, as discussed further below, and minimizes the need for concrete pads and other impacts to the land.

California's largest utilities have also recognized the value of this technology. We have signed contracts for over 2.6 gigawatts of solar power with Pacific Gas & Electric Company (PG&E) and Southern California Edison Company (SCE). The California Public Utilities Commission (CPUC) has approved the PG&E contracts, the first two of which are for two of the three plants comprising the Ivanpah SEGS, and is currently reviewing the SCE contracts, including the contract for the third of the Ivanpah SEGS plants. Our PG&E and SCE contracts represent approximately one-third of all of the announced solar thermal utility-scale contracts in the nation. Our technology, and the contracts for supplying the energy from that technology to California's largest utilities, uniquely positions BrightSource to help California meet our state's ambitious, yet achievable, renewable energy goals.

Ivanpah SEGS will be the first BrightSource Energy project to meet the PG&E and SCE contracts, and the first BrightSource project to help meet our state's 33 percent clean energy and climate change requirement. When constructed, Ivanpah SEGS will produce more solar energy than all of the rooftop solar that was installed in the nation last year, and will more than double the amount of solar thermal energy produced in the U.S. today.

Ivanpah SEGS is estimated to directly create 1,000 jobs at the peak of construction and 86 permanent jobs, as well as to indirectly create more jobs needed to supply materials and services during the Ivanpah SEGS ' construction and operation. The project is estimated to directly produce $650 million in wages and $400 million in state and local taxes over its thirty-year lifecycle, and again to spawn additional indirect economic activity in the area.

Ivanpah SEGS will also avoid more than 13 million tons of CO_2 emissions over its lifecycle, as well as 85 percent of the air emissions from an equally-sized natural gas plant. The plants will employ dry-cooling, which will reduce water usage by 90 percent, allowing Ivanpah SEGS to use approximately 30 times less water than competing technologies using wet cooling. The project will use roughly 100 acre feet of water – the equivalent of 300 homes' annual water usage, and far less than the amount used by the adjacent golf course or nearby casinos. While dry-cooling comes at an additional cost, we believe that this proven technology must be used to help conserve precious desert water.

Ivanpah SEGS ′ environmental considerations to reduce development impacts also include a low-impact design and use of a currently-used high-voltage transmission pathway that transects the site. The low impact design utilizes BrightSource's proprietary hanging heliostats, which minimize the need for grading and concrete pads required for competing technologies.

The Ivanpah SEGS project has been identified as a "fast-track" priority by the U.S. Department of Interior for obtaining federal stimulus benefits for California under the 2009 American Recovery and Reinvestment Act (ARRA). The project has also been selected as one of sixteen short-listed applicants to receive a loan guarantee under the U.S. Department of Energy (DOE) 1703 program, established by the 2005 Energy Policy Act, and is the only utility-scale solar project so selected.

The BrightSource energy team takes its responsibility for designing and developing environmentally-responsible solar energy very seriously, and as a leading utility-scale solar project has approached every facet of its work with the goal of setting a good example for projects that follow. This effort started with our selection of the Ivanpah location for its proximity to existing high-voltage transmission lines, which cross the site, to existing roads and other infrastructure on and adjacent to the site, and for other reasonably suitable environmental characteristics. It continued with our extensive biological surveying efforts, our low-impact design, technology selection and commitment to dry-cooling. This work, and these commitments, are reflected in our AFC Ivanpah SEGS application, originally filed in August 2007, and in our achievement of data adequacy status within three months, on October 31, 2007. We have invested thousands of hours and millions of dollars in preparing a thoughtful application, responding to requests for information, and participating in the Commission's review process.

We do have substantial concerns regarding the FSA/DEIS, most of which are technical in nature and will be addressed by other witnesses. It is important for the Commission, and the BLM, to understand that financing and building the renewable energy projects California needs are neither easy nor simple tasks. Prior to the current economic crisis, these tasks, particularly for new technologies like ours that will provide greater environmental and economic performance, would have been difficult enough. Under present economic conditions, even with DOE loan guarantees, these challenges have grown far steeper.

Layering on unnecessary burdens that are novel in nature or application, or that appear disproportionate – such as effectively unbounded and extreme mitigation requirements – represent a serious threat to the viability of this project and others that follow. The excesses of the FSA/DEIS could, if let stand, make the Ivanpah SEGS project effectively impossible, threatening BrightSource's ability to deliver on contracts not just at Ivanpah but at other locations. These contracts, including contracts already approved by the CPUC and others currently under review, are being relied upon by California's major utilities to meet their Renewables Portfolio Standard obligations, and as a foundation to meet the 33% renewable energy requirement under California's AB 32 climate change law. The excesses of the FSA/DEIS could also be expected to drive many of the other utility-scale solar projects California needs for a least-GHG, reliable and renewable energy infrastructure out of the state, or to drive the utility-scale solar developers relying on the California market out of business entirely. Further delays and regulatory complications have already negatively impacted how the state is perceived by the utility-scale solar industry, leading development to advance elsewhere and increasing the cost and impact of transmission needed to serve generation further removed from California's load centers. As these projects move to other states, we jeopardize reaching the environmental and economic goals at the heart of California's renewable energy law.

The Commission must be vigilant in ensuring that this project is treated equitably with other conventional generating projects licensed by the Commission, and that California projects have some reasonable parity with projects located in adjacent states. This is not the time to add new regulatory hurdles, to lower the thresholds of significance or to vastly exceed previously-required mitigation measures. Renewable energy, and utility-scale solar in particular, is in its infancy, with great promise to compete with conventional energy economically while yielding tremendous environmental and public health benefits. Overburdening this fledgling industry will cause it to be stillborn, ending that promise before it has truly begun, to the detriment of the California's desert and other ecosystems as well as to its public health and economy.

It is imperative for this project, for BrightSource's future, and for the future of the utility-scale solar industry – again, a keystone to California's renewable energy future - that the Commission's final decision redresses these excesses and provides an appropriately balanced foundation for this project and other projects to come.

We appreciate the efforts that the BLM, the CEC and their staff have made to reach this point. Without the staff's strong commitment to California's and the nation's renewable energy and climate goals, this important renewable energy facility—and its contribution to a clean, reliable and sustainable energy economy for California and the nation—will not be possible. I would like to close this testimony as it began, by thanking the staff in advance for the work they are dedicated to perform; their efforts will enable California to deliver on its great promise to be the world leader in both renewable energy and the fight against climate change.

B. Summary

Solar Partners I, LLC; Solar Partners II, LLC; Solar Partners VIII, LLC, the owners of the three separate solar plant sites, and Solar Partners IV, LLC, the owner of shared facilities required by the three solar plant sites propose to develop a solar facility (together referred to as the Ivanpah Solar Electric Generating System, or Ivanpah SEGS) in the Ivanpah valley about 4.5 miles southwest of Primm, NV. These four companies are Delaware limited liability companies. BrightSource Energy Inc. (BSE), a Delaware corporation, is a technology and development company, and the parent company of the Solar Partners entities.

Ivanpah SEGS will consist of Ivanpah 1 through 3, three independent solar thermal electric generating facilities (or plants) that will be co-located approximately 1.6 miles west of the Ivanpah Dry Lake, in San Bernardino County, California. The project site will be located on federal property managed by the Bureau of Land Management (BLM). The three Ivanpah SEGS facilities will have a combined nominal net rating of 400 megawatts (MW). The project is planned to be constructed in three phases: Ivanpah 1 (nominal 100 MW), Ivanpah 2 (nominal 100 MW), and Ivanpah 3 (nominal 200 MW).

The total Ivanpah SEGS project area would affect approximately 4,062 acres (Table PD-1). Ivanpah 1 will require about 913.5 acres (1.43 square miles) and Ivanpah 2 will require about 920.7 acres (1.44 square miles), while Ivanpah 3 is larger and will require approximately 1,836.3 acres (2.9 square miles). The project boundary for Ivanpah 1, 2, and 3 will cover a total of 3,670.5 acres (5.7 square miles). Additionally, there will be a common area between Ivanpah 1 and 2 (approximately 377.5 acres), called the Construction Logistics Area, that will include the

Southern California Edison (SCE) substation and shared facilities (administration/storage building, groundwater production wells, and portions of the linear facilities). Portions of this common area will be used during construction for staging, laydown, and temporary offices. Additionally, approximately 13.7 acres will be used for construction of the gas tap station and gas line, the dirt road to the mining claim, and paving of a portion of Colosseum Road.

TABLE PD-1
Area Affected by Ivanpah SEGS

FACILITY DESCRIPTION	Length (Feet)	Acres
Ivanpah 1		913.5
Ivanpah 2		920.7
Ivanpah 3		1,836.3
Kern River Gas Line Tap Station and construction laydown area	2,010	1.3
Gas line corridor to Ivanpah 3 (50 feet wide)		2.3
Construction Logistics Area		377.5
Improvements to Colosseum Road from Golf Club to project (50' construction corridor)	8,440	9.7
12' trail to access mining claim – new dirt road	1,490	0.4
TOTAL AFFECTED AREA		**4,061.6**

Note: totals do not add due to independent rounding.

A low-impact design (LID) approach for will be used for the Ivanpah Solar Electric Generating System (SEGS). This approach focuses on preserving undeveloped land and minimizing stormwater generation. In the *Review of Low Impact Development Policies: Removing Institutional Barriers to Adoption,* the Low Impact Development Center (LIDC) states:

> *The underlying principle of LID is that undeveloped land does not present a stormwater runoff or pollution problem. The evolved natural hydrology of any given site manages water in the most efficient manner. This most often translates to high rates of infiltration, vegetative interception, and evapotranspiration.*

Use of LID attempts to offset the inevitable consequences of development and changes in land cover by preserving or mimicking natural hydrology. It is a source control option that minimizes stormwater pollution by recognizing that the greatest efficiencies are gained by minimizing stormwater generation. This is a process that begins with functional conservation of watershed resources, reducing impacts of development, and then using innovative management practices to meet the stormwater objective; it is not the use of the management practices alone.

Project Design Elements

This section describes the elements associated with the proposed project design, including the heliostat (mirror) fields, the power block, water supply and treatment, wastewater management, shared utility corridors, substation and switchyard, networks of access roads and maintenance paths, fire protection systems, and an administration and maintenance complex.

Each of the three proposed solar plants will consist of heliostat fields surrounding a power block, which is supplied with the necessary utilities through a utility corridor. Each of the solar plants will be connected to SCE's planned step-up substation, which will in turn tie into SCE's electric-power transmission network (or grid) through an existing (115-kilovolt [kV]) transmission line that runs across the project area. Each of the design elements are described below.

Heliostat Fields

The 100-MW plants (Ivanpah 1 and 2) will each have heliostat arrays consisting of up to 55,000 heliostats.[1] The 200-MW plant (Ivanpah 3) will have heliostat arrays consisting of up to 104,000 heliostats.[2] The heliostat arrays would be arranged around a single centralized solar power tower (SPT). The heliostats would automatically track the sun during the day and reflect the solar energy to the boiler on top of the SPT.

Each heliostat mirror is 7.2 feet high by 10.5 feet wide (2.20 meters by 3.20 meters) yielding a reflecting surface of 75.6 square feet (7.04 square meters). Each heliostat consists of two mirrors mounted on a single pylon, along with a computer-programmed aiming control system that directs the motion of the heliostat to track the movement of the sun. Communication cables connecting the heliostats between one another will be strung aboveground.

The aiming control system and the layout of solar fields are optimally designed to focus sunlight on to the SPT in a manner that maximizes steam output. The aiming control system uses optimization software to instruct the solar field controller where each heliostat should aim to maximize solar energy collection and output. This patent-pending software system accounts for the light flux intensity and distribution required for the SPT boiler, and various other conditions such as sun radiation, wind, air pressure, and the number of heliostats available for tracking. When computing the optimal aiming policy, the aiming control system factors in the differences between heliostats with respect to their tracking accuracy, the intensity of the beam they reflect (both of these factors depend mainly on the distance to the receiver), the shape of the beam, and other relevant aspects. The optimization software will also prevent the mirrors from being aimed toward the freeway or the golf club at an angle that would reflect sunlight near the ground surface.

Power Block

Each solar power plant (Ivanpah 1 through 3) will have a power block located in the approximate center of the heliostat array. The power block will include an SPT, a receiver boiler, a steam turbine generator (STG) set, air-cooled condensers, and other auxiliary systems. This section describes the SPTs and receiving boilers, and the power block systems to be installed in each plant.

Solar Power Tower and Receiving Boiler

The SPT is a metal structure designed specifically to support the boiler and efficiently move high-quality steam through a STG at its base. The SPT (i.e., the support structure) would be about 120 meters high (approximately 393 feet). The receiving boiler (which sits on top of the support structure) would be 20 meters tall (approximately 66 feet) including the added height

[1] However, the power purchase agreement states that both Ivanpah 1 and 2 would have no more than 70,000 heliostats.

[2] However, the power purchase agreement states that Ivanpah 3 would have no more than 140,000 heliostats.

for upper steam drum and protective ceramic insulation panels. Overall, the tower height would be 140 meters (approximately 459 feet). Additionally, a Federal Aviation Administration (FAA)-required lighting and a lightening pole will extend above the top of the towers approximately 5 to 10 feet. The height of the SPT allows heliostats from significant distances to accurately reflect sunlight to the receiving boiler. The receiving boiler is a traditional high-efficiency boiler positioned on top of the SPT. The boiler converts the concentrated energy of the sun reflected from the heliostats into superheated steam. The boilers will be supplied by conventional boiler manufacturers providing performance warranties and industry best practices, and will comply with standard boiler design parameters. The boiler's tubes are coated with a material that maximizes energy absorbance. The boiler has steam generation, superheating, and reheating sections and is designed to generate superheated steam at a pressure of 160 bars and a temperature of 550 degrees Celsius (°C).

Power Block System

The power block system proposed for this project is the same as that used in traditional power-generation facilities to convert steam to electricity. The power block consists of a conventional Rankine-cycle STG with a reheat cycle, and auxiliary functions of heat rejection, water treatment, water disposal, and grid interconnection capabilities. The integration of high-efficiency pre-existing turbine technologies provides performance warranties and enables the system to maximize thermal-to-electricity efficiencies. To minimize water use, air (rather than water) will be used to cool the steam.

Each plant will have a backup diesel generator to provide power to operate boiler recirculation pumps, firewater pumps, and other small consumers in the event of an emergency when power might otherwise be unavailable.

Water Supply and Treatment

Two new groundwater production wells will be drilled and developed to provide raw water for the Ivanpah SEGS project. The two wells will be located near the northwest corner of Ivanpah 1. The wells, and their respective pumping systems, will be sized for 100-percent redundancy. Groundwater will be used to supply domestic and industrial water needs. These wells are anticipated to supply water to all three plants to be used as make-up water.

Make-up water for the steam system will be treated by means of a mixed-bed ion-exchange system to produce feedwater-quality water for use in the boiler system. The ion exchange resigns will be sent offsite for regeneration. Drinking water will either be brought onsite or a small filter/purification system would be used to provide potable water for sanitary uses (sinks, showers, and toilets) within the plants.

Wastewater Management

A package treatment plant will be used at the administration and maintenance complex to treat wastewater. Portable toilets will be placed in the power block areas of each the three solar facilities. Portable toilets will be serviced by a waste management firm on a regular basis, depending on the number of toilets and staff at each facility.

Utility Corridors

Due to the size of the facilities, it will be necessary to route several utilities between the individual facilities (internal utility corridors) and the combined facilities (external utility corridors). This section describes the utility corridors—specifically, the internal and external utility corridors, electrical transmission system, natural gas system, and water supply system—and how they will function at each SEGS plant.

Internal Utility Corridors

Within each SEGS facility there will be a utility corridor required for the overhead electrical lines and fiber-optic cables from the switchyard to the SCE substation. Additionally, an underground utility corridor will contain water and natural gas lines. These underground corridors will run parallel to the local access roads between the facilities and the common area.

The two groundwater production wells will be located northwest of Ivanpah 1. These wells will be connected via an approximately 1,075-foot-long underground water line to the main trunk line going to the administration/warehouse building, and then from there to Ivanpah 1, 2, and 3.

The internal electrical transmission interconnections will link each plant to the power grid by connecting the plant switchyard to the new SCE substation (Ivanpah substation). The substation will be located between Ivanpah 1 and Ivanpah 2 on the north side of the existing transmission corridor.

External Utility Corridor(s)

External to the SEGS project, utilities including natural gas pipelines, telecommunications, and transmission lines will require upgrades or new construction. These utilities will either provide services to the facilities (natural gas pipeline and telecommunications), or transmit the electrical energy generated at the facilities (transmission lines).

Electrical Transmission and Telecommunication Systems

Gen-tie Lines. Ivanpah 1, 2, and 3 would be interconnected to an existing SCE grid through an upgraded SCE 115-kV line passing between Ivanpah 1 and 2 on a northeast-southwest utility corridor. SCE will upgrade the existing 115-kV transmission line between the new Ivanpah substation and the El Dorado substation to 220 kV. This SCE upgrade is designed to serve other projects planned in the general vicinity and is not being built specifically for the Ivanpah SEGS project. It will provide sufficient capacity for the Ivanpah SEGS project and other projects anticipated by SCE. A substation will be constructed between Ivanpah 1 and 2 that will be used to connect the Ivanpah SEGS to the electrical grid.

The 115-kV transmission generation tie line (gen-tie line) from the edge of the Ivanpah 1 solar field to the substation would be approximately 2,870 feet long. The Ivanpah 2 and 3 gen-tie lines extend approximately 2,300 feet and 12,760 feet, respectively, from their switchyards before coming together. The combined gen-tie line (double-circuit) would then extend approximately 1,900 feet from the southern end of Ivanpah 2 to the substation. There would be a 12-foot-wide dirt service road running alongside the gen-tie lines.

Each circuit would be supported by single-pole structure at appropriate intervals with final heights as determined during detailed design. The shared gen-tie line for Ivanpah 2 and 3 would be carried on a double-circuit pole. The lines would be insulated from the poles using porcelain insulators.

Substation and Switchyard

Ivanpah 1, 2, and 3 would be interconnected to the existing SCE grid through an upgraded El Dorado– Baker–Coolwater-Dunn Siding-Mountain Pass 115-kV line passing between Ivanpah 1 and 2 on a northeast-southwest utility corridor. A 115/220-kV substation would be constructed between Ivanpah 1 and 2 that would be used to connect the Ivanpah SEGS to the electrical grid. The substation dimensions would be about 830 feet wide by 850 feet long—approximately 16.1 acres. Additionally, a 24-foot-wide asphalt road about 1,760 feet long will be needed to connect the substation to the re-routed Colosseum Road (on the south side of Ivanpah 2).

Telecommunication Line

The proposed Ivanpah substation would also require new telecommunication infrastructure to be installed to provide protective relay circuit, Supervisory Control and Data Acquisition (SCADA) circuit, data, and telephone services. The telecommunication path from Ivanpah substation to local carrier facility interface in the Mountain Pass area consists of approximately 8 miles of fiber-optic cable to be installed overhead on existing poles and new underground conduits to be constructed in the substation and telecom carrier interface point. This fiber-optic route consists of two segments. The first segment is from Ivanpah substation to Mountain Pass substation using the existing Nipton 33-kV distribution line poles built along the transmission line corridor that crosses between Ivanpah 1 and 2. The second segment would be from Mountain Pass substation to the telecommunications facility approximately 1.5 miles away at an interface point to be designated by the local telecommunication carrier. The fiber-optic cable would be installed on the existing Earth 12-kV distribution line poles.

Natural Gas System

Natural gas would be used as a supplementary fuel for project operation. Each phase of the project includes a small package natural gas-fired start-up boiler to provide heat for solar plant start-up and during temporary cloud cover. Natural gas would be obtained by the construction of a new 6-mile-long, 4- to 6-inch distribution pipeline from the existing Kern River Gas Transmission (KRGT) pipeline located approximately 0.5 mile north of the Ivanpah 3 site. A permanent gas metering station and a temporary construction area would be located at the point of connection. From the tap station, the natural gas line would run south along the western edge of Ivanpah 3 to a metering station near its southeast corner. Although the gas line and metering station would be within the area that was surveyed, they would be located outside the project's fenced heliostat fields and a dirt access road would follow the pipeline so that the gas company has access to it for maintenance.

From the metering station at Ivanpah 3, the gas line would continue along the eastern edge of Ivanpah 2 to another metering station outside the southeast corner of Ivanpah 2 west of Colosseum Road that would serve Ivanpah 1 and 2. Again, the gas line and metering station would be located within the project area, but outside the fenced heliostat field. From that metering station, the gas line to Ivanpah 1 would be located alongside or under the 30-foot-wide paved access road that goes from Colosseum Road past the administration/warehouse building to Ivanpah 1's power block.

A gas-metering station would be required at the KRGT tap point to measure and record gas volumes. Additionally, facilities would be installed to regulate the gas pressure and to remove any liquids or solid particles. Construction activities related to the metering station and metering sets would include grading a pad and installing above- and below-ground gas piping, metering equipment, gas conditioning, pressure regulation, and pigging facilities. Either a distribution line or photovoltaic cells and batteries would be used for metering station operation lighting and communication equipment. Perimeter chain-link fencing for security would also be installed.

Access Roads and Maintenance Paths

Project access would be from Colosseum Road to the project entrance road. Colosseum Road is an existing dirt road, which will be paved (30 feet wide, two lanes) for a 1.9-mile length from the Primm Valley Golf Club to the project site.[3] The project would re-route a portion of Colosseum Road around the southern end of the Ivanpah 2 plant site for a distance of 1.2 miles, which will also be a 30-foot paved, two-lane road, then continue as a 12-foot-wide dirt or gravel road for approximately 2,452 feet to connect to the point where the existing Colosseum dirt road would exit the Ivanpah 2 site boundary. Additionally, paved access roads would be created to access the power blocks of the three Ivanpah plant sites within the fenced solar sites.

Within the heliostat fields, paths will be located concentrically around the power block (or in the case of Ivanpah 3, around the power towers) to provide access to the heliostat mirrors for maintenance and cleaning. The paths will be located between every other row of heliostats and will not be graded. There also will be a maintenance path on the inside perimeter of the project boundary fence. These paths will be used for plant security and to monitor and maintain the perimeter and tortoise fencing.

Additionally, dirt roads will be installed diagonally through the heliostat fields and used for access to the heliostat maintenance paths. These dirt roads will follow existing topography.

Rerouted Trails

Existing dirt trails that traverse the site will be re-routed, either around the project site or to a proposed paved access road. Each re-routed dirt trail will be 8 to 12 feet wide (to match the existing trail) and will be reconnected to the original dirt trail on the other side of the project site. Permanent tortoise gates will be installed to prevent tortoises from entering internal roads.

Fire Protection Systems

Each power block will have 150,000 gallons of water in the raw water tank dedicated to fire suppression.

Administration and Maintenance Complex

An administration, warehouse, and maintenance complex would be located between the relocated Colosseum Road and the entrance to the Ivanpah 1 solar plant. It would include parking and landscape areas. The complex would require about 8.9 acres and would be served by power from the Ivanpah substation, water from the water supply wells, and gas from the main gas trunk line running from the KRGT line to the Ivanpah 1 power block.

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[3] A portion of this road has recently been paved from the golf club to its wells, but likely lacks a sufficient road base.

Facility Reliability

This subsection discusses the expected facility availability, equipment redundancy, fuel availability, water availability, and project quality control measures.

Facility Availability

Because of the Ivanpah SEGS system needs, it is anticipated that the facility will normally operate at high average annual capacity factors during periods of sunlight.

Ivanpah SEGS will be designed for an operating life of 30 years. Reliability and availability projections are based on this operating life. Operation and maintenance procedures will be consistent with industry standard practices to maintain the useful life status of plant components.

The percent of time that the power plants are projected to be operated is defined as the service factor. The service factor considers the amount of time that a unit is operating and generating power, whether at full or partial load. The projected service factor for the power block, which considers projected percent of time of operation, differs from the equivalent availability factor (EAF), which considers the projected percent of energy production capacity achievable.

The EAF may be defined as a weighted average of the percent of full energy production capacity achievable. The projected equivalent availability factor for the Ivanpah SEGS is estimated to be approximately 92 to 98 percent.

The EAF, which is a weighted average of the percent of energy production capacity achievable, differs from the availability of a unit, which is the percent of time that a unit is available for operation, whether at full load, partial load, or standby.

Redundancy of Critical Components

The following subsection identifies equipment redundancy as it applies to project availability. A summary of equipment redundancy is shown in the following table. Final design could differ.

TABLE PD-2
Major Equipment Redundancy

Description	Number	Note
Solar Receiver Boilers	Three trains – Ivanpah 1 & 2 Four trains – Ivanpah 3	Steam turbine bypass system allows all boiler trains to operate at base load with the steam turbine out of service for 30 seconds until heliostat defocusing.
Solar boiler Superheater	Three – One per plant	See note above pertaining to Solar Receiver Boilers.
STG	Three – One per plant	See note above pertaining to Solar Receiver Boilers.
Boiler feedwater pumps	One – 100 percent per boiler	One spare for all Solar Receiver Boilers.
Condensate pumps	Three – 50 percent capacity per plant	—

TABLE PD-2
Major Equipment Redundancy

Description	Number	Note
Condenser	One per plant	Condenser must be in operation for plant operation or operation of boilers in steam turbine bypass mode. The condenser will be provided with split water boxes to allow online tube cleaning and repair.
Circulating water pumps	Two – 60 percent capacity per receiver-boiler	The facility may operate at reduced load with one of the two circulating water pumps in service.
Fuel gas booster compressors	One – 100 percent capacity per plant	
Demineralizer system	One – 100 percent capacity per plant	

III. Proposed Licensing Conditions

The FSA/DEIS for the project filed by the CEC recommends that 14 Conditions of Certification be adopted to address general conditions including compliance monitoring and closure plan issues: COMPLIANCE-1 through COMPLIANCE-14. These are acceptable.

IV. Correlation to FSA and Hearing Topics:

- Executive Summary, Introduction, Project Description, Facility Design, Power Plant Efficiency, Power Plant Reliability and General Conditions

[i] http://www.worldenergyoutlook.org/docs/weo2009/climate_change_excerpt.pdf

Electric Transmission

I. Introduction

A. Name: Roger Gray

B. **Qualifications:** Mr. Gray's qualifications are as noted in his resume contained in Appendix A.

C. Prior Filings: In addition to the statements herein, this testimony includes by reference the following documents submitted in this proceeding:

- Application for Certification, Volume 1 [Exhibit 1]
- Data Adequacy Supplement A [Exhibit 2]
- Comments to the PSA [Exhibit 57]
- Applicant's Response to CEC Staff Requests, Data Response Set 1A, dated January 14, 2008. Responses to Data Requests 91 through 96 [Exhibit 4].
- Applicant's Response to CEC Staff Requests, Data Response Set 1B, dated February 11, 2008. Responses to Data Requests 93 [Exhibit 5].
- DPT 2 System Impact Study Report (Confidential) [Exhibit 54]

To the best of my knowledge, all of the facts contained in this Section of the Applicant's testimony (including all referenced documents) are true and correct. To the extent this testimony contains opinions, such opinions are my own based upon my professional judgment. I make these statements, and render these opinions freely and under oath for the purpose of constituting sworn testimony in this proceeding.

II. Summary of Testimony

A. Affected Environment

The Applicant proposes to develop a solar energy project called the Ivanpah Solar Electric Generating System (Ivanpah SEGS). It will be located in southern California's Mojave Desert, near the Nevada border, to the west of Ivanpah Dry Lake, in San Bernardino County, California, on federal land managed by the Bureau of Land Management (BLM). It will likely be constructed in three phases: two 100-megawatt (MW) phases (known as Ivanpah 1 and 2) and a 200-MW phase (Ivanpah 3). The phasing would most likely be Ivanpah 1 (the southernmost site), followed by Ivanpah 2 (the middle site), and then Ivanpah 3 (the 200-MW plant on the north), though the order of construction may change. Each 100-MW site requires about 900 acres; the 200-MW site is about 1,800 acres. The total area required for all three phases, including the Administration/Operations and Maintenance building and substation, is approximately 4060 acres. The Applicant has applied for right-of-way grants for the land from BLM.

B. Facility Discussion

The heliostat (or mirror) fields focus solar energy on the power tower receivers near the center of each of the heliostat arrays (the 100-MW plants have three arrays and the 200-MW plant has four arrays). In each plant, one Rankine-cycle reheat steam turbine receives live steam from the solar boilers and reheat steam from one solar reheater—located in the power block at the top of its own tower. The solar field and power generation equipment are started each morning after sunrise and insolation build-up, and shut down in the evening when insolation drops below the level required to keep the turbine online.

Ivanpah 1, Ivanpah 2, and Ivanpah 3 will be interconnected to the Southern California Edison (SCE) grid through an upgraded SCE 115-kilovolt (kV) line passing through the site on a northeast-southwest right-of-way already granted to SCE. SCE submitted on May 28, 2009, its Certification for Public Convenience and Necessity (CPCN) for the El Dorado-Ivanpah Transmission Project (EITP) to the CPUC for review and approval. The EITP includes: (1) the construction by SCE of a new 220-kV/115-kV double breaker double bus substation between the Ivanpah 1 and Ivanpah 2 project sites (called the Ivanpah Substation); (2) the replacement of the existing 115-kV transmission line from the El Dorado Substation with a double-circuit 220-kV overhead line that will be interconnected into the new substation; (3) various substation and transmission lines improvements such as breakers and transformer banks and supporting telecommunications facilities; and other facilities not related to the ISEGS project.

The new Ivanpah Substation and system upgrades will be for the benefit of Ivanpah and other interconnecting customers in the region, as well as future growth. The Ivanpah Substation and capability to interconnect the first Ivanpah SEGS project is expected to be completed before the first Ivanpah SEGS project comes on line. In the event of a delay in the EITP project, the Applicant has included in the LGIA for the first Ivanpah project, a provision to connect to interconnect earlier using a temporary interconnection configuration. Power from each Ivanpah plant will be interconnected to the California Independent System Operator (CAISO) grid via 115-kV generator tie lines (gen-tie lines) to the new Ivanpah Substation.

Each phase of the project includes a small package natural gas-fired start-up boiler to provide heat for plant start-up and during temporary cloud cover. The project's natural gas system will be connected to the Kern River Gas Transmission Line, which passes less than half a mile to the north of the project site.

Raw water will be drawn daily from one of two onsite wells, located south of Ivanpah 2. Each well will have sufficient capacity to supply water for all three phases. Groundwater will go through a treatment system for use as boiler make-up water and to wash the heliostat mirrors. To save water in the site's desert environment, each plant will use a dry-cooling condenser. Water consumption is reduced to less than 10 percent of a comparable wet cooling system plant, and therefore considered minimal (estimated at no more than 100 acre-feet/year for all three phases).

The site for Ivanpah SEGS was selected, in part, for its proximity to existing transmission corridors, and potential interconnection substations. Los Angeles Department of Water and Power (LADWP) and SCE both have multiple transmission or sub-transmission lines in the Ivanpah corridor adjacent to the proposed project sites.

C. Proposed Transmission Interconnection

Ivanpah SEGS applied for interconnection to the SCE grid via the SCE 115-kV El Dorado-Baker-Cool Water-Dunn Siding-Mountain Pass line passing between the Ivanpah 1 and 2 sites along a northeast-southwest right-of-way. Initially, SCE's feasibility study for Ivanpah 1 indicated that a double-circuit 115-kV upgrade between the Mountain Pass and El Dorado substations would be required to interconnect Ivanpah 1. However, to accommodate other projects in the interconnection queue, SCE has subsequently proposed significant upgrades to the El Dorado-Mountain Pass line, as described previously. SCE' s planned EITP can accommodate all 3 Ivanpah SEGS plants.

Ivanpah SEGS will be connected to the SCE grid via 3 new 115-kV generation tie lines (gen-tie lines) between each plant and the new Ivanpah Substation. Each of the power plants will have a 115-kV breaker at the plant switchyard on the 115-kV terminals of the plant's generator step-up (GSU) transformer. Power from each plant will be routed to the new Ivanpah Substation via three single-circuit transmission lines. The Ivanpah 2 and Ivanpah 3 lines will merge approximately 1,800 feet east-northeast of the new substation into one double-circuit line before entering the Ivanpah Substation switchyard (see Figure 3.1-1 in Section 3 of the AFC).

Ivanpah SEGS 115-kV Switchyard Characteristics

Each proposed Ivanpah SEGS 115-kV plant switchyard will consist of one 115-kV circuit breaker on the 115-kV terminals of the GSU transformer. The Ivanpah 1, 2, and 3 switchyards and all associated equipment will be designed for the maximum short-circuit and load-flow design conditions of the installation. Surge suppression bushings will be used on the pull-off structures to minimize lightening and switching surges. Each of the generators will be connected to the low voltage side of the GSU transformer via a generator breaker. A tap between the generator breaker and low voltage terminals of the GSU transformer will serve the plant auxiliary loads via a 13.8 to 4.16-kV Unit Auxiliary Transformer (UAT). Each plant UAT will be adequately sized to provide power to all auxiliary loads within each facility. Startup and standby power will be supplied through the GSU and unit auxiliary transformers. Auxiliary controls and protective relay systems for the 115-kV switchyard will be located in a control building or the switchyard building separate from the power plant.

Generator Tie-Line Characteristics

The proposed gen-tie lines will be engineered for operation at 115 kV, nominal and appropriate for the maximum generation for each Ivanpah generator. Each circuit will either be installed within conduits underground, or be supported by single-pole structures at appropriate intervals with final heights as determined during detailed design. Final design will be compliant with all LORS and provisions within the Large Generator Interconnect Agreements (LGIA) for each Unit.

Ivanpah 1 and 2

The Ivanpah 1 and 2 facilities' interconnections are electrically identical except for the routing of each unit's gen-tie line. Power will leave Ivanpah 1 and 2 through each facility's respective 13.8-kV generator circuit breaker, a 13.8 to 115-kV GSU transformer, and a 115-kV circuit breaker at the plant switchyard. The power would then be transmitted to the new Ivanpah Substation located between Ivanpah 1 and 2.

The conductors for Ivanpah 1 and 2 will be selected based on the maximum operating output capability of each facility determined by final heat balances.

The Ivanpah 1 gen-tie line will exit the property to the northwest either via underground conduits or on a single-circuit pole line and cross the facility entrance road proceeding westerly to the Ivanpah Substation. This 115-kV line will be routed under the existing LADWP 500-kV Marketplace-Adelanto line near tower MKP-ADL1 3 4/3 where conductor height is maximized to ensure that all clearances as specified in the National Electric Safety Code (NESC) can be met in final design. The 115-kV gen-tie line (shown in Figure 3.1-1 of the AFC) would continue over the existing SCE 33-kV distribution circuit and into the Ivanpah substation from the southeast. SCE has stated that it will evaluate the location, re-routing or undergrounding of the distribution circuit during detailed design efforts.

The Ivanpah 2 gen-tie line will be routed within the facility property to the south either via underground conduits or on a single-circuit pole line and merge at the Ivanpah 2 property line with the Ivanpah 3 gen-tie line onto a double-circuit pole. The two circuits will be routed along a 115-kV double-circuit pole line before entering the Ivanpah Substation from the north.

Ivanpah 3

The electrical configuration of Ivanpah 3 is identical to Ivanpah 1 and 2 except for the routing of the 115-kV gen-tie line. Power will leave Ivanpah 3 through a 13.8-kV generator circuit breaker, a 13.8 to 115-kV GSU transformer, and a 115-kV circuit breaker at the plant switchyard.

The Ivanpah 3 single-circuit gen-tie line is routed within the Ivanpah 3 property to the south. It then proceeds over the fence line between Ivanpah 2 and 3, inside of the Ivanpah 2 site boundary south, then east along the Ivanpah 2 fence line to minimize land impacts.

At the Ivanpah 2 southern site boundary, the Ivanpah 3 gen-tie line merges with the Ivanpah 2 gen-tie line onto a 115-kV double-circuit pole. The two circuits will be routed along a double-circuit pole line before entering the Ivanpah Substation from the north .

Considerations at the Ivanpah Substation

The design of the Ivanpah Substation and associated transmission line upgrades will be performed by SCE and is discussed in the CPCN filing SCE made to the CPUC on May 28, 2009.

D. Transmission Interconnection System Impact Study and Facility Study Reports

System Impact Study Reports have been completed for Ivanpah 1, 2, and 3. Facility Study Reports (FSR) have been completed and issued in DRAFT for comment for Ivanpah 1, 2, and 3. Copies of the FSR's will be docketed upon completion of all comments by the parties.

In all cases, SCE-required interconnection and network upgrades will be incorporated based on the findings of the studies and agreed to by the parties involved including the Project Owner, Transmission Operator (TO), and the CAISO.

E. Transmission System Safety and Nuisances

This section discusses safety and nuisance issues associated with the proposed electrical interconnection of Ivanpah SEGS with the CAISO electrical grid.

Electrical Clearances

The proposed Ivanpah SEGS transmission interconnection will be designed to meet all national, state, and local code clearance requirements. Since the designer must take into consideration many different situations, the generalized dimensions provided in Figures 3.3-1 through 3.3-9 of the AFC should be regarded only as reference for the EMF calculations. The minimum ground clearance for a 115-kV transmission line per the NESC is 23.06 feet, based on the road-crossing minimum. This is the design clearance for the maximum operating temperature of the line. Under normal conditions, the line operates well below maximum conductor temperature, and thus, the average clearance is much greater than the minimum.

Electric and Magnetic Fields and Audible Noise

The AFC discusses in detail the Electrical Effects, Magnetic Fields, and Audible Noise issues. This testimony will only reiterate the conclusions. The discussions in the AFC are incorporated by reference.

Transmission Line EMF Reduction

While the State of California does not set a statutory limit for electric and magnetic field levels, the CPUC, which regulates electric transmission lines, mandates EMF reduction as a practicable design criterion for new and upgraded electrical facilities. As a result of this mandate, the regulated electric utilities have developed their own design guidelines to reduce EMF at each new facility. The CEC, which regulates transmission lines to the first point of connection, requires generators to follow the existing guidelines that are in use by local electric utilities or transmission-system owners.

In keeping with the goal of EMF reduction, the interconnections of Ivanpah 1, 2, and 3 will be designed and constructed using the principles outlined in the SCE publication, "EMF Design Guidelines for Electrical Facilities" (EMF Research and Education, 2004). These guidelines explicitly incorporate the directives of the CPUC by developing design procedures compliant with Decision 93-11-013 and General Orders 95, 128, and 131-D. That is, when the transmission line structures, conductors, and rights-of-way are designed and routed according to the SCE guidelines, the transmission line is consistent with the CPUC mandate.

The primary techniques for reducing EMF anywhere along a transmission line are to:

1. Increase the pole height for overhead design
2. Use compact pole-head configuration
3. Minimize the current on the line
4. Optimize the configuration of the phases (A, B, C)

Anticipated EMF levels have been calculated for the Ivanpah 1, 2 and 3 interconnections as designed. If required, the pre- and post-interconnections verification measurements will be made consistent with IEEE guidelines and will provide sample readings of EMF at the edge of right-of-way. Additional measurements will be made upon request for locations of particular concern.

EMF and Audible Noise Conclusions

The public exposure to EMF and audible noise levels due to the proposed interconnection of Ivanpah SEGS are well within accepted levels, notwithstanding the fact that there will be very little public exposure at the site. The effect of the added EMF and corona noise would be well below the levels produced by the existing LADWP 500-kV line. SCE has stated that the existing 115-kV El Dorado-Baker-Cool Water-Dunn Siding-Mountain Pass line passes under the existing 500-kV and 230-kV transmission lines 22 times along its routing. The Ivanpah 1 crossing with the 500-kV LADWP line is not expected to contribute any additional significant EMF effects over existing conditions based on the following considerations:

- The 115-kV El Dorado-Baker-Cool Water-Dunn Siding-Mountain Pass line carries more current than the Ivanpah 1 gen-tie line will, and therefore, it would produce greater EMFs at ground level than the proposed crossings.
- These crossings are accommodated by configuring the 115-kV circuit low and wide to clear the 500-kV lines. In addition, Ivanpah 1 will be installed near the 500-kV tower to ensure maximum height and minimum EMF effects.
- There are no residences within 2 miles of the proposed Ivanpah SEGS site; therefore, no extended EMF exposure to the public is likely.

F. Cumulative Impacts

Compliance with LORS and codes in the design of ISEGS will not create any significant adverse cumulative impacts.

G. Mitigation

As there are no significant adverse impacts identified to result from the compliance with LORS, Codes, Standards, etc. in designing the facility. As such no mitigation is warranted for this area. Applicant intends to comply with laws, ordinances, regulations, and standards (LORS), and codes that govern power plant facility design.

III. Proposed Licensing Conditions

The FSA/DEIS for the project filed by the CEC and BLM recommends that four Condition of Certification be adopted to address transmission line safety and nuisance issues, TLSN-1 through TLSN-4. Our proposed changes to these conditions are set forth below. The FSA/DEIS also recommends that seven Conditions of Certification be adopted to address transmission system engineering issues, TSE-1 through TSE-7. The Applicant only proposes changes to conditions TSE-5 and TSE-6; the others are acceptable.

Proposed Revisions to TLSN-1 through 4

The Applicant proposes changes to make terminology in the FSA/DEIS consistent with terminology used by the CAISO and the Transmission System Operator.

TLSN-1 The project owner shall construct the proposed ~~transmission~~ generation tie lines to the first point of interconnection according to the requirements of California Public Utility Commission's GO-95, GO-52, GO-131-D, Title 8, and Group 2. High Voltage Electrical Safety Orders, sections 2700 through 2974 of the California Code of Regulations, and Southern California Edison's EMF-reduction guidelines.

Verification: At least 30 days before starting the ~~transmission~~ generation tie lines or related structures and facilities, the project owner shall submit to BLM's Authorized Officer and the Compliance Project Manager (CPM) a letter signed by a California registered electrical engineer affirming that the lines will be constructed according to the requirements stated in the condition.

TLSN-2 The project owner shall use a qualified individual to measure the strengths of the electric and magnetic fields from the line at the points of maximum intensity along the route for which the applicant provided specific estimates. The measurements shall be made before and after energization according to the American National Standard Institute/Institute of Electrical and Electronic Engineers (ANSI/IEEE) standard procedures. These measurements shall be completed no later than 6 months after the start of operations.

Verification: The project owner shall file copies of the pre-and post-energization measurements with BLM's Authorized Officer and the CPM within 60 days after completion of the measurements.

TLSN-3 The project owner shall ensure that the rights-of-way of the proposed ~~transmission~~ generation tie lines are kept free of combustible material, as required under the provisions of section 4292 of the Public Resources Code and section 1250 of Title 14 of the California Code of Regulations.

Verification: During the first 5 years of plant operation, the project owner shall provide a summary of inspection results and any fire prevention activities carried out along the right-of-way and provide such summaries in the Annual Compliance Report to be provided to BLM's Authorized Officer and the CPM.

TLSN-4 The project owner shall ensure that all permanent metallic objects within the right-of-way of the project-related generation tie lines are grounded according to industry standards regardless of ownership.

Verification: At least 30 days before the lines are energized, the project owner shall transmit to BLM's Authorized Officer and the CPM a letter confirming compliance with this condition.

Proposed Revisions to TSE-5

Inclusion of detailed engineering data that specifies exact generator tie line lengths, size, and design criteria is inappropriate in a COC. This information will be developed during the detailed design phase and reviewed and approved by the CBO and reviewed and approved by SCE for compliance with its requirements as defined in the LGIA document for each facility. The comment on the configuration of the SCE substation is to bring to the attention of the Staff, that SCE typically has proposed a double buss-double breaker arrangement. ~~Again, it is inappropriate for Staff to be defining design criteria for the Transmission System Operator, whose designs are under the purview of the PRC and CPUC.~~

TSE-5 The project owner shall ensure that the design, construction, and operation of the proposed transmission facilities will conform to all applicable LORS, ~~including the requirements listed below~~. The project owner shall submit the required number of copies of the design drawings and calculations as determined by the CBO.

1. ~~The Ivanpah 1 will be interconnected to the SCE grid via a segment of 115kV, 477 kcmil-ACSR, approximately 5,800 feet long single circuit.~~

   ~~The Ivanpah #2 will be interconnected to the SCE grid via a segment of 115-kV, 477 kcmil-ACSR, approximately 3900 feet long single circuit and a segment of 115kV, 477- kcmil, approximately 1400 feet long double circuit generator tie-line.~~

   ~~The Ivanpah #3 generator tie line would be approximately 14,100 feet long, single circuit, 115kV line built with 1510 kcmil ACSR and would merge into a 115kV double circuit with the Ivanpah #2 generator tie line.~~

   The proposed Ivanpah substation would use a double bus breaker- and a half configuration with 3-bays and 5 positions or other configuration as may be approved in SCE's Certification of Public Convenience and Necessity (CPCN).

* * *

6. Termination and interconnection facilities shall comply with applicable SCE interconnection standards.
7. The project owner shall provide to BLM's Authorized Officer and the CPM:
   a. The final Detailed Facility Study (DFS) including a description of facility upgrades, operational mitigation measures, and/or Special Protection System (SPS) sequencing and timing if applicable,
   b. Executed project owner, Transmission System Operator, and California ISO Large Generator ~~Facility~~ Interconnection Agreement (LGIA).

Verification: At least 60 days prior to the start of construction of transmission facilities (or a lessor number of days mutually agreed to by the project owner and CBO), the project owner shall submit to the CBO for approval:

* * *

Proposed Revisions to TSE-6

These comments bring the COC into agreement with the Large Generator Interconnect Agreement (LGIA) document signed by the Project Owner, Transmission System Operator (TO), and the CAISO. The LGIA document is the controlling agreement for the project interconnection process.

TSE-6 The project owner shall provide the following Notice to the California Independent System Operator (California ISO) prior to synchronizing the facility with the California transmission system as required in the LGIA:

~~1. At least one weekConsistent with the LGIA, prior to synchronizing the facility with the grid for testing, provide the California ISO a letternotice pursuant to the LGIA stating the proposed date of synchronization; and~~

~~2. At least one business day Consistent with the LGIA, prior to synchronizing the facility with the grid for testing, provide telephone notification pursuant to the LGIA to the California ISO Outage Coordination Department.~~

Verification: The project owner shall provide copies of the California ISO ~~letter~~ notice to BLM's Authorized Officer and the CPM when it is sent to the California ~~I~~ISO~~SO one week~~ ~~prior to initial synchronization with the grid~~. A report of the conversation with the California ISO shall be provided electronically to BLM's Authorized Officer and the CPM one day before synchronizing the facility with the California transmission system for the first time.

IV. Correlation to FSA and Hearing Topics:

- Transmission Line Safety and Nuisance
- Transmission System Engineering

Air Quality

I. Introduction

A. Name: Steve Hill and Gary Rubenstein

B. Qualifications: Mr. Hill's and Mr. Rubenstein's qualifications are as noted in their resumes contained in Appendix A.

C. Prior Filings: In addition to the statements herein, this testimony includes by reference the following documents submitted in this proceeding:

- Application for Certification, Volumes 1&2. [Exhibit 1]
- Data Adequacy Supplement A [Exhibit 2]
- Comments to the PSA [Exhibit 57]
- Air Dispersion Modeling Protocol, Ivanpah Solar Electric Generating Station, dated June 18, 2007 [Exhibit 50]
- Applicant's Response to CEC Staff Requests, Data Response Set 1A, dated January 14, 2008. Responses to Data Requests 7 through 12 [Exhibit 4].
- Applicant's Response to CEC Staff Requests, Data Response Set 1B, dated February 11, 2008. Responses to Data Request 9 [Exhibit 5].
- Applicant's Response to CEC Staff Requests, Data Response Set 1D, dated May 9, 2008. Responses to Data Requests 8 and 9 [Exhibit 7].
- Applicant's Response to CEC Staff Requests, Data Response Set 2A, dated June 10, 2008. Responses to Data Requests 117 through 120 [Exhibit 20].
- Applicant's Response to CEC Staff Requests, Supplemental Data Response Set 1A, dated August 12, 2008. Responses to Data Request AQ-1 [Exhibit 32].
- Letter dated September 18, 2007, BrightSource Energy (John Woolard) to MDAQMD (Sam Oktay) submitting application for ATC/PTO. [Exhibit 53]
- Letter dated August 20, 2007, Sierra Research (Steve Hill) to MDAQMD (Richard Wales) requesting information needed to conduct a cumulative impact analysis. [Exhibit 51]
- Letter dated November 8, 2008, Sierra Research (Steve Hill) to MDAQMD (Sam Oktay) providing comments on PDOC. [Exhibit 55]
- Letter dated March 31, 2009, Sierra Research (Steve Hill) to MDAQMD (Sam Oktay) requesting revision to FDOC. [Exhibit 59]

- Letter dated June 24, 2009, Sierra Research (Steve Hill) to MDAQMD (Sam Oktay) requesting revision to FDOC [Exhibit 61]

Documents Prepared by Others

- Letter dated August 23, 2007 from Mojave Desert Air Quality Management District (Alan De Salvio) to Sierra Research (Steve Hill) describing stationary sources within 6 miles of the Project. [Exhibit 52]
- Letter dated February 15, 2008 from Mojave Desert Air Quality Management District (Alan De Salvio) to Jack Caswell (California Energy Commission) providing a Preliminary Decision/Determination. [Exhibit 58]
- Letter dated December 3, 2008 from Mojave Desert Air Quality Management District (Alan De Salvio) to Jack Caswell (California Energy Commission) providing a Final Decision/Determination. [Exhibit 56]
- Mojave Desert Air Quality Management District Revision A Final Determination of Compliance, dated April 9, 2009 [Exhibit 60]
- Mojave Desert Air Quality Management District Revision B Final Determination of Compliance, dated July 15, 2009 [Exhibit 62]

To the best of our knowledge, all of the facts contained in this testimony (including all referenced documents) are true and correct. To the extent this testimony contains opinions, such opinions are our own. We make these statements, and render these opinions freely and under oath for the purpose of constituting sworn testimony in this proceeding.

II. Summary of Testimony

Air pollutant emissions from the proposed Ivanpah SEGS result from operation of the natural-gas-fired boilers used to bring the system to operating temperature in the morning, and keep it at temperature during transient cloud cover, and from additional supporting equipment. These emissions will be controlled through the use of the best available pollution control technology. This project is an important component of the State's renewable energy program, and is a bellwether for commercial solar energy projects throughout southern California. The project will be located in the Ivanpah Valley, where air quality levels are within most (but not all) air quality standards. The air quality impacts of the Project were evaluated and shown to satisfy all state and federal air quality requirements. This conclusion was confirmed, after extensive reviews by the Mojave Desert Air Quality Management District (MDAQMD, or Air District), in the Final Determination of Compliance issued on July 15, 2009.

A. Existing Air Quality

The U.S. Environmental Protection Agency (USEPA) and California Air Resources Board have each established ambient air quality standards to protect public health and welfare. Both state and national ambient air quality standards consist of two parts: an allowable concentration of a pollutant, and an averaging time over which the concentration is to be measured. Allowable concentrations are based on the results of studies of the effects of pollutants on human health, crops and vegetation. The averaging times are based on whether the damage caused by the pollutant is more likely to occur during exposures to a high concentration for a short time (one hour, for instance), or to a relatively lower average concentration over a longer period.

Air quality standards have been set for ozone, carbon monoxide, nitrogen dioxide, sulfur dioxide, particulate sulfates, respirable particulate matter (PM_{10}) and fine particulate matter ($PM_{2.5}$). Four ambient air monitoring stations were used to characterize air quality at the project site. These stations were used because of their proximity to the project site and because they record area-wide ambient conditions rather than the localized impacts of any particular facility. All of the ambient air quality data that were relied upon were taken from publications and data sources prepared by the California Air Resources Board (CARB). Ambient concentrations of ozone, nitrogen dioxide, carbon monoxide and PM_{10} were taken from a monitoring station located in Barstow. $PM_{2.5}$ measurements were collected in Trona. Ambient concentrations of sulfur dioxide were measured at Trona, while ambient concentrations of sulfates were measured in Riverside County. The monitoring stations for each pollutant are summarized in Table 1:

TABLE 1
Ambient Air Monitoring Station Locations

Pollutant	Station	Distance from Project Site
Ozone	Barstow, Trona	100 mi W-SW, 110 mi W-NW
Nitrogen Dioxide	Barstow, Trona	100 mi W-SW, 110 mi W-NW
Carbon Monoxide	Barstow	100 mi W-SW
PM_{10}	Barstow, Trona	100 mi W-SW, 110 mi W-NW
$PM_{2.5}$	Big Bear City	120 mi SW
Sulfates	Riverside County	~175 mi SW
Sulfur Dioxide	Trona	110 mi W-NW

All of these monitoring stations (except Riverside County) are located in the Mojave Desert Air Basin, the same air basin in which the project is located. Each of these monitoring stations is the closest station to the project site for the pollutant monitored. Because these stations are the closest to the project site, and are generally located in or just downwind of more heavily developed areas, the concentrations recorded at these stations are believed to be representative of, or more conservative (higher) than, concentrations expected to be found at the project site.

Ozone

Ozone is generated by a complex series of chemical reactions between VOC and NOx in the presence of ultraviolet radiation. Ambient ozone concentrations follow a seasonal pattern: higher in the summertime and lower in the wintertime. At certain times, the general area can provide ideal conditions for the formation of ozone due to the persistent temperature inversions, clear skies, mountain ranges that trap the air mass, and exhaust emissions from millions of vehicles and stationary sources. The entire MDAQMD has been designated nonattainment of the ozone CAAQS. CARB has classified the MDAQMD as a moderate ozone nonattainment area based on a 110 ppb ozone design value monitored at Barstow, California on April 29, 1989.

The eastern portion of San Bernardino County (including the project site) has been designated by USEPA as "unclassified/attainment" for the federal one-hour and eight-hour ozone standards.

Carbon Monoxide

CO is a product of inefficient combustion, principally from automobiles and other mobile sources of pollution. In many areas of California, CO emissions from wood-burning stoves and fireplaces can also be measurable contributors to ambient CO levels. Industrial sources typically contribute less than 10 percent of ambient CO levels. Peak CO levels usually occur during winter due to a combination of higher emission rates and calm weather conditions with strong, ground-based inversions. The MDAQMD is classified as an attainment area for CO with respect to both state and national standards.

There have been no violations of either the state or federal CO standards since at least 1997.

Nitrogen Dioxide

Atmospheric NO2 is formed primarily from reactions between nitric oxide (NO) and oxygen or ozone. NO is formed during high temperature combustion processes, when the nitrogen and oxygen in the combustion air combine. Although NO is less harmful than NO2, it can be converted to NO2 in the atmosphere within minutes to hours, depending on the composition and temperature of the atmosphere. For purposes of state and federal air quality planning, the MDAQMD is in attainment for NO2.

There have been no violations of either the state one-hour standard or the federal annual average NO2 standard since at least 1997.

Sulfur Dioxide and Sulfates

SO2 is produced when any sulfur-containing fuel is burned. It is also emitted by chemical plants that treat, or refine, sulfur or sulfur-containing chemicals. Natural gas contains only a small amount of sulfur, typically about 0.2 grains per standard cubic foot, while fuel oils contain larger amounts, typically in the range of 15 ppm (for ultra-low sulfur Diesel fuel) to 4 percent (for marine bunker fuels). Peak, but low, concentrations of SO2 occur at different times of the year in different parts of California, depending on local fuel characteristics, weather, and topography. The MDAQMD is considered to be in attainment for SO2 for purposes of state and federal air quality planning.

Particulate sulfates result from the further oxidation of sulfur dioxide in the atmosphere. Sulfate levels have also been well below state standards. (There are no federal standards for sulfates.)

PM10

Particulates in the air are caused by a combination of wind-blown fugitive dust; particles emitted from combustion sources and manufacturing processes; sea salts; and organic, sulfate, and nitrate aerosols formed in the air from emitted hydrocarbons, sulfur oxides, and nitrogen oxides, respectively. In 1984, CARB adopted standards for PM10 and phased out the total suspended particulate (TSP) standards that had been in effect previously. PM10 standards were substituted for TSP standards because PM10 corresponds to the size range of particulates that can be inhaled into the lungs (respired), and therefore, is a better measure to use in assessing potential health effects. In 1987, USEPA also replaced national TSP standards with PM10 standards. San Bernardino County is nonattainment for the federal PM10 standard, and MDAQMD is a nonattainment area for the state standard.

The maximum 24-hour and the annual average PM10 levels exceed the state standards, but the annual average PM10 levels have remained below the federal standards since 1997.

PM2.5

The NAAQS for particulates were revised by USEPA with new standards that went into effect on September 16, 1997; two new PM2.5 standards were added at that time. In June 2002, CARB established a new annual standard for PM2.5. PM2.5 data have been collected at the Big Bear City monitoring station since 1999. The 24-hour average concentrations have not exceeded the federal standard since 1997; however, there are not enough data available to draw conclusions regarding trends in the 3-year average of 98th percentile values. Annual average PM2.5 levels have been below both the federal standard and the state standard. Eastern San Bernardino County, where the Ivanpah SEGS is located, is unclassified for the state PM2.5 standard, and is unclassified/attainment for the federal standard.

B. Environmental Impacts

Air emissions will result from the operation of the boilers, emergency standby engine, and Diesel fire pump. Fugitive dust emissions will also result from maintenance activities (e.g., mirror washing). Air pollutant emissions from the Ivanpah SEGS are shown in the Application for Certification and in the Final Staff Assessment/Draft Environmental Impact Statement (FSA/DEIS). These emissions have been calculated based on the maximum capacity of the equipment, consistent with operating limits expected to be imposed as permit conditions, and thus represent a worst case. Actual emissions during plant operation are expected to be much lower than the levels shown in the FSA/DEIS.

C. Regulatory Requirements

The project's emissions and air quality impacts are required to comply with various local, state, and federal laws, regulations, and standards. In addition to the California Energy Commission's review, the air quality impacts of the Ivanpah SEGS have been reviewed by the MDAQMD.

The requirements applicable to the Ivanpah SEGS include new source review (NSR) requirements and a number of prohibitory rules.[1] The NSR program applies to the facility as a whole, and is designed to ensure that new projects are developed in a manner that will not interfere with meeting health- and welfare-based ambient air quality standards. Prohibitory rules apply to specific pieces of equipment, rather than to the facility as a whole. They impose specific limits on emissions, including opacity and odors, and are enforced through permit conditions. Compliance with all of these rules is demonstrated in the Application for Certification, and has been confirmed in the Final Determination of Compliance issued by the Air District.

The main air quality requirements applicable to the Ivanpah SEGS are summarized below.

- Best Available Control Technology (BACT): Emissions of all pollutants will be kept as low as possible by using clean natural gas as the fuel for the boilers. Because natural gas is a

[1] The Ivanpah SEGS is not subject to federal PSD review, since emissions from the project do not exceed federal PSD trigger levels.

clean-burning fuel, emissions of sulfur dioxide (SO2), precursor organic compounds (POC, or hydrocarbons), and particulate matter (PM10) will be very low. To minimize emissions of oxides of nitrogen (NOx) and carbon monoxide (CO), the boilers will use special combustion systems, known as low-NOx burners.

- Offsets: Both Air District and Energy Commission rules require that overall air quality does not deteriorate as a result of the project. Air Districts have set emission thresholds for each pollutant. Projects above the thresholds must mitigate emission increases by providing emission offsets. Projects below the thresholds are generally deemed by the District to be too small to require project-by-project offsets; the District mitigates these through its regional air quality planning process. All emissions from the project are below District thresholds for offset requirements.

- Ambient Air Quality Impacts: The impact of the Ivanpah SEGS on ambient air quality was evaluated using dispersion models approved by the USEPA. Worst-case ground-level impacts were assessed for various meteorological and operating conditions (flat terrain, elevated terrain/hillsides, fumigation, part-load and full-load operations, and startups). The worst-case ground-level impacts were added to existing (background) concentrations from nearby monitoring stations to determine the total ambient concentrations. These total concentrations were then compared with the ambient air quality standards. As confirmed in the Final Staff Assessment, the project will result in concentrations well below the most stringent air quality standards. Even when combined with existing background levels, the proposed project will not cause a new violation of any state or federal air quality standard. The project will add a small amount (approximately one-tenth of one percent) to existing PM_{10} concentrations at the point of maximum impact.

- Screening Health Risk Assessment: A screening level health risk assessment was performed to evaluate the potential impact of emissions of potentially toxic compounds that result from the combustion of natural gas. This assessment demonstrated that the facility will not pose a significant health risk. The worst-case cancer risk for the plant is well below the level of 10 in one million that is considered significant, and is well below the level of 1 in one million that triggers additional control technology requirements.

- A protocol for a cumulative air quality impact analysis of the Ivanpah SEGS was prepared and included in the Application for Certification. Consultation with the MDAQMD indicated that there were no sources of emissions that had the potential to contribute with the Project, to a significant air quality impact. The ambient air quality impact analysis discussed above included the combination of worst case project impacts with maximum concentrations in the ambient air (reflecting the operation of existing sources); this analysis also demonstrates that the Ivanpah SEGS will not create any new cumulative impacts.

III. Proposed Licensing Conditions

The proposed conditions of certification include the conditions required by MDAQMD (AQ-1 to AQ-39). These conditions ensure compliance with state, federal, and local air quality standards. The applicant has reviewed these conditions, and finds them acceptable.

The proposed conditions of certification related to air quality also include those proposed by the California Energy Commission Staff (CEC Staff) (AQ-SC1 to AQ-SC10) as supplements to the requirements of the MDAQMD, principally related to mitigation of construction-related impacts. The Applicant has reviewed these conditions, and has substantive concerns with several of them. None of these concerns, however, relate to the quantity of mitigation provided, or to conclusions regarding the significance of project air quality impacts. These concerns are described in more detail below. Except as noted in below, the Applicant has no objections to the CEC Staff's proposed air quality conditions of certification.

Proposed Revisions to AQ-SC3

Staff has recommended that the main access roads through the facility to the power block areas be paved with asphalt as a means to reduce dust. However, the heavy construction equipment would quickly destroy the paved roads adding addition unwarranted costs.

Staff has also recommended that all disturbed areas be stabilized with a non toxic soil stabilizer or soil weighting agent, and has stated that any stabilizer or weighting agent used shall have no other environmental impacts. Soil stabilizers are often used on roadways and other high-traffic areas in order to minimize erosion and wear-and-tear on the road. Acrylic compounds can be used as stabilizers.

Application of a polymeric substance over several hundred acres of desert soil is very likely to affect the water absorption and runoff characteristics of the area. It is also likely to affect the quantity, type, and health of vegetation that can grow.

The dust mitigation measures included in the PSA adequately address the control of dust from the project. The emission reductions expected from staff's new proposal have not been quantified, costs have not been estimated, and the potential impacts on water and biological resources have not been assessed.

AQ-SC3 Construction Fugitive Dust Control: The AQCMM shall submit documentation to the BLM's Authorized Officer and CPM in each Monthly Compliance Report that demonstrates compliance with the following mitigation measures for the purposes of preventing all fugitive dust plumes from leaving the project. Any deviation from the following mitigation measures shall require prior BLM Authorized Officer and CPM notification and approval.

A. The main access roads through the facility to the power block areas will be paved using "crusher run" material prior to construction, and use dust control during construction. The road to the power block will be paved with asphalt at the conclusion of construction. ~~prior to initiating construction in the main power block area, and delivery~~ Delivery areas for operations materials (chemicals, replacement parts, etc.) will be paved prior to taking initial deliveries.

B. All unpaved construction roads and unpaved operational site roads, as they are being constructed, shall be stabilized with a non-toxic soil stabilizer or soil weighting agent that can be determined to be both as efficient or more efficient for fugitive dust control as ARB approved soil stabilizers, and shall not increase any other environmental impacts including loss of vegetation. ~~All other disturbed areas in the project and linear construction sites shall be watered as frequently as necessary~~

~~during grading and stabilized with a non-toxic soil stabilizer or soil weighting agent to comply with the dust mitigation objectives of Condition of Certification **AQ-SC4**. The frequency of watering can be reduced or eliminated during periods of precipitation.~~

* * *

I. Construction areas adjacent to any paved lower elevation roadway shall be provided with sandbags or other equivalently effective measures to prevent run-off to roadways, or other similar run-off control measures as specified in the Storm Water Pollution Prevention Plan (SWPPP)~~, only when such SWPPP measures are necessary so that this condition does not conflict with the requirements of the SWPPP~~.

J. All paved roads within the construction site shall be swept ~~at least twice~~ daily or as needed (~~or~~ less during periods of precipitation) on days when construction activity occurs to prevent the accumulation of dirt and debris.

K. At least the first 500 feet of any paved public roadway exiting the construction site or exiting other unpaved roads en route from the construction site or construction staging areas shall be swept ~~at least twice daily~~as needed (~~or~~ less during periods of precipitation) on days when construction activity occurs or on any other day when dirt or runoff resulting from the construction site activities is visible on the public paved roadways.

* * *

Proposed Revisions to AQ-SC5

Staff added a requirement to use Tier 3 Diesel engines in all construction equipment. The basis for this new requirement was not presented in the FSA/DEIS. Construction emissions presented in the Application for Certification were based on Tier 2 Diesel engines. Staff did not estimate emission reductions expected from the proposed new requirement.

All contractors must comply with applicable state and federal requirements concerning the engines used in their construction fleets. These take the form of limits on emissions from new equipment (USEPA Certification standards), and fleet requirements designed to phase out older engines.

CEC staff's proposed requirement goes beyond state and federal law, requiring the use of new (Tier 3) engines for all project construction activities. Assuming that Tier 3 engines are available, and owned by construction contractors, the requirement to use Tier 3 engines on this project will only result in non-Tier 3 engines being displaced to other projects. Net emissions to the region will be unaffected by the requirement.

Furthermore, if the construction contractor does not own sufficient equipment powered by Tier 3 engines, the new requirement would obligate the contractor to either rent Tier 3 equipment or revise the construction schedule to make use of the available equipment. If rental equipment is not available, then Tier 2 equipment may be used. If Tier 2 equipment is not available, Tier 1 equipment must be retrofit to Tier 2 standards.

Delays in construction that are caused by the lack of availability of Tier 2 or Tier 3 engines will result in increased worker trips, with resulting increased emissions.

This new requirement places a potentially heavy burden on the construction contractor, without sufficient justification (in the form of quantified emission reductions), or adequate analysis of the resulting cost. State and federal regulations, already in place, require that old engines in a contractor's fleet be phased out for new ones. This new requirement places a potentially significant burden on the applicant without corresponding benefit to the environment.

Proposed Revisions to AQ-SC5

AQ-SC5

* * *

b. All construction diesel engines with a rating of 50 hp or higher shall meet, at a minimum, the Tier 2 or 3 California Emission Standards for Off-Road Compression-Ignition Engines, as specified in California Code of Regulations, Title 13, section 2423(b)(1), unless a good faith effort that is certified by the on-site AQCMM demonstrates that such engine is not available for a particular item of equipment. This good faith effort shall be documented with signed written correspondence by the appropriate construction contractors along with documented correspondence with at least two construction equipment rental firms. In the event that a Tier 2 or 3 engine is not available for any off-road equipment larger than 100 hp, that equipment shall be equipped with a Tier ~~2~~ 1 engine, or an engine that is equipped with retrofit controls to reduce exhaust emissions of nitrogen oxides (NOx) and diesel particulate matter (DPM) to no more than Tier ~~2~~ 1 levels unless certified by engine manufacturers or the on-site AQCMM that the use of such devices is not practical for specific engine types. For purposes of this condition, the use of such devices is "not practical" for the following, as well as other, reasons.

1. There is no available retrofit control device that has been verified by either the California Air Resources Board or U.S. Environmental Protection Agency to control the engine in question to Tier ~~2~~ 1 equivalent emission levels and the highest level of available control using retrofit or Tier 1 engines is being used for the engine in question; or

* * *

Proposed Revisions to AQ-SC6

Staff has proposed a requirement that all dedicated vehicles used for maintenance activities be new model year on-road vehicles, or other vehicles that meet on-road emission standards. Applicant agrees that any dedicated vehicles purchased for this project should be new model year vehicles.

As discussed in the Applicant's comments on the PSA, some activities (e.g., mirror washing) cannot be performed by light-duty on-road vehicles. It is not clear at this time if even heavy-duty on-road vehicles would be appropriate for this service. The FSA does not evaluate the feasibility of the proposed condition.

The Applicant proposed to revise the condition to require the use of new model year vehicles for all activities and to leave the decision as to whether the vehicle selected is an on-road or off-road vehicle to the facility manager.

AQ-SC6 The project owner, when obtaining dedicated vehicles for mirror washing activities and other facility maintenance activities, shall only obtain new model year vehicles that meet California on-road vehicle emission standards for the model year when obtained, except for those activities that cannot be performed by on-road vehicles.

~~Other vehicle/fuel types may be allowed assuming that the emission profile for those vehicles, including fugitive dust generation emissions, is comparable to the vehicles types identified in this condition.~~

Proposed Revisions to AQ-SC7

The requirement to apply soil stabilizer to all disturbed areas, and not just to roadways, was discussed above in the comments on AQ-SC3. This requirement is also included in Condition AQ-SC7. For the same reasons previously mentioned, the requirement should be deleted from AQ-SC7.

AQ-SC7 The project owner shall provide a site operations dust control plan, including all applicable fugitive dust control measures identified in AQ-SC3 that would be applicable to reducing fugitive dust from ongoing operations; that:

* * *

The site operations fugitive dust control plan shall include the use of durable non-toxic soil stabilizers on all regularly used unpaved roads ~~and disturbed off-road areas within the project boundaries~~, and shall include the inspection and maintenance procedures that will be undertaken to ensure that the unpaved roads remain stabilized. The soil stabilizer used shall be a non-toxic soil stabilizer or soil weighting agent that can be determined to be both as efficient or more efficient for fugitive dust control as ARB approved soil stabilizers, and shall not increase any other environmental impacts including loss of vegetation. The performance and application of the fugitive dust controls shall also be measured against and meet the performance requirements of condition AQ-SC4.

* * *

Verification: At least 60 days prior to start of commercial operation, the project owner shall submit to the BLM's Authorized Officer for review and comment and to the CPM for review and approval a copy of the plan that identifies the dust and erosion control procedures, including effectiveness and environmental data for the proposed soil stabilizer, that will be used during operation of the project and that identifies all locations of the speed limit signs. The CPM will notify the project owner of any necessary modifications to the plan within 30 days from the date of receipt. At least 60 days after commercial operation, the project owner shall provide to the BLM's Authorized Officer and the CPM a report identifying the locations of all speed limit signs, and a copy of the project employee and contractor training manual that clearly identifies that project employees and contractors are required to comply with the dust and erosion control procedures and on-site speed limits.

Proposed Revisions to AQ-SC9

Staff wishes to review and approve the selection of emergency engines. The condition is acceptable if it includes an obligation for timely review of the proposed purchase.

AQ-SC9 The emergency generator and fire pump engines procured for this project will meet or exceed the NSPS Subpart IIII emission standards for the model year that corresponds to their date of purchase.

Verification: The project owner shall submit the emergency engine specifications to the CPM at least 30 days prior to purchasing the engines for review and approval. Unless the CPM disapproves the purchase within 15 days of the receipt of the specifications, the purchase shall be deemed to be approved.

Proposed Revisions to AQ-SC10

Staff proposed a new condition limiting the fraction of project heat input that could be derived from the boilers. The justification for this new condition was "Staff recommends Condition of Certification AQ-SC10 to ensure that the boiler operation does not exceed the amount that was modeled in the applicant's air quality modeling analysis and to formalize the applicant's stipulation that 'Heat input from natural gas will not exceed 5 percent of the heat input from the sun, on an annual basis.' "

The proposed condition is not necessary to ensure that the boiler operation does not exceed the amount that was modeled in the air quality modeling analysis—that is already achieved by Condition of Certification AQ-22, which limits daily and annual hours of operation.

The Applicant would like to retain the flexibility to operate the boilers at the levels modeled in the air quality modeling analysis, even if the heat input from the sun falls below 95 percent of the total system input. There is no environmental regulatory basis for imposing the 5 percent limit on boiler operation. Condition of Certification AQ-SC10 should be deleted.

~~**AQ-SC10** The ISEGS 1, ISEGS 2, and ISEGS 3 boilers shall not exceed a total annual natural gas fuel heat input that is more than 5 percent of the total annual heat input from the sun for ISEGS1, ISEGS2, and ISEGS 3, respectively.~~

~~Verification: Annual natural gas fuel heat input data and annual solar heat input data for the ISEGS 1, ISEGS 2, and ISEGS 3 units showing compliance with this condition shall be provided in the Annual Compliance Report (COMPLIANCE-7).~~

IV. Correlation to FSA and Hearing Topics:

- Air Quality

Biological Resources

I. Introduction

A. Name: John Cleckler, Mark Cochran, Amy Hiss, Geof Spaulding, Ann Howald, Russ Huddleston, John Carrier, Steve De Young, and Andy Sanders.

B. Qualifications: The qualifications of the various authors are as noted in their resumes contained in Appendix A.

C. Prior Filings: In addition to the statements herein, this testimony includes by reference the following documents submitted in this proceeding:

- Application for Certification, Volumes 1&2. [Exhibit 1]
- Data Adequacy Supplement A [Exhibit 2]
- Comments to the PSA [Exhibit 57]
- Applicant's Response to CEC Staff Requests, Data Response Set 1A, dated January 14, 2008, Responses to Data Requests 13 through 32 [Exhibit 4].
- Applicant's Response to CEC Staff Requests, Data Response Set 1B, dated February 11, 2008, Responses to Data Requests 13, 14, 19, 20, 23 through 24, 26, 29 and 30 [Exhibit 5].
- Applicant's Response to CEC Staff Requests, Data Response Set 1C, dated March 10, 2008, Responses to Data Requests 23 [Exhibit 6].
- Applicant's Response to CEC Staff Requests, Data Response Set 1D, dated May 9, 2008, Responses to Data Request 26 [Exhibit 7].
- Applicant's Response to CEC Staff Requests, Data Response Set 1E, dated July 22, 2008, Responses to Data Requests 13, 14, 21, 22, and 29 [Exhibit 8].
- Applicant's Response to CEC Staff Requests, Data Response Set 1F, dated August 6, 2008, Responses to Data Requests 13 and 14 [Exhibit 9].
- Applicant's Response to CEC Staff Requests, Data Response Set 1G, dated September 10, 2008, Responses to Data Requests 19 and 29 [Exhibit 10].
- Applicant's Response to CEC Staff Requests, Data Response Set 1H, dated September 12, 2008, Responses to Data Requests 29 [Exhibit 11].
- Applicant's Response to CEC Staff Requests, Data Response Set 1I, dated October 24, 2008, Responses to Data Request 24 [Exhibit 12]
- Applicant's Response to CEC Staff Requests, Data Response Set 1K, dated May 27, 2008, Responses to Data Request 19 [Exhibit 14].

- Applicant's Response to CEC Staff Requests, Data Response Set 1L, dated June 2, 2008, Responses to Data Request 19 [Exhibit 15].
- Applicant's Response to CEC Staff Requests, Data Response Set 1M, dated June 3, 2008, Responses to Data Requests 19 [Exhibit 16].
- Applicant's Response to CEC Staff Requests, Data Response Set 2A, dated June 10, 2008, Responses to Data Request 124 [Exhibit 20].
- Applicant's Response to CEC Staff Requests, Data Response Set 2B, dated July 22, 2008, Responses to Data Request 124 [Exhibit 21].
- Applicant's Response to CEC Staff Requests, Data Response Set 2C, dated August 6, 2008. Responses to Data Request 125 [Exhibit 22].
- Applicant's Response to CEC Staff Requests, Data Response Set 2D, dated September 12, 2008, Responses to Data Request 124 [Exhibit 23].
- Applicant's Response to CEC Staff Requests, Data Response Set 2K, dated June 30, 2009, Responses to Data Requests 30 and 31 [Exhibit].
- Applicant's Response to CEC Staff Requests, Data Response Set 2KR, dated September 10, 2009, Responses to Data Requests 125 [Exhibit 31].
- Applicant's Response to CEC Staff Requests, Supplemental Data Response Set 1A, dated August 12, 2008, Responses to Data Request BR-1 [Exhibit 32].
- Applicant's Response to CEC Staff Requests, Supplemental Data Response Set 1B, dated August 22, 2008, Responses to Data Request BR-2 [Exhibit 33].
- Applicant's Response to CEC Staff Requests, Supplemental Data Response Set 1D, dated September 24, 2008, Responses to Data Request BR-3 [Exhibit 35].
- Applicant's Response to CEC Staff Requests, Supplemental Data Response Set 1E, dated November 21, 2008, Responses to Data Request BR-4 [Exhibit 36].
- Applicant's Response to CEC Staff Requests, Supplemental Data Response Set 2A, dated March 19, 2009, Responses to Data Request BR-5 [Exhibit 38].
- Applicant's Response to CEC Staff Requests, Supplemental Data Response Set 2D, dated May 27, 2009, Responses to Data Requests BR-6 [Exhibit 41].
- Applicant's Response to CEC Staff Requests, Supplemental Data Response Set 2E, dated June 3, 2009, Responses to Data Request BR-7 [Exhibit 42].
- Applicant's Response to CEC Staff Requests, Supplemental Data Response Set 2G, dated June 9, 2009, Responses to Data Requests BR-8 through BR-10 [Exhibit 44].
- Applicant's Response to CEC Staff Requests, Supplemental Data Response Set 2I, dated August 10, 2009, Responses to Data Request BR-5 [Exhibit 46].
- Applicant's Response to CEC Staff Requests, Supplemental Data Response Set 2J, dated August 12, 2009, Responses to Data Request BR-5 [Exhibit 47]

- Letter to John Kessler from the Applicant regarding Applicant's Biological Resources Mitigation, dated August 7, 2009 [Exhibit 57]

To the best of our knowledge, all of the facts contained in this testimony (including all referenced documents) are true and correct. To the extent this testimony contains opinions, such opinions are our own. We make these statements, and render these opinions freely and under oath for the purpose of constituting sworn testimony in this proceeding.

II. Summary of Testimony

A. Regional Overview

The Applicant proposes to develop a solar facility composed of three adjacent solar energy plants collectively referred to as the Ivanaph Solar Electric Generating System, or Ivanaph SEGS in the Mojave Desert region of San Bernardino County, California. The proposed project is located land administered by the Bureau of Land Management approximately 3.1 miles from the Nevada border and within the western end of Ivanpah Valley.

The project area lies on an alluvial fan, or bajada with a maximum elevation of 3,150 feet, that extends eastward from the Clark Mountains to Ivanpah Dry Lake. The alluvial fan is braided by many ephemeral wash drainage features that flow eastward through the project area and eventually to Ivanpah Dry Lake.

The Ivanpah SEGS site is primarily dominated by a Mojave Creosote Bush Scrub vegetation community with assemblage of cacti and annuals. The site is accessed by dirt roads and has been used to various degrees for recreation, Off-road vehicle (OHV) use, and livestock grazing, among other activities permitted under BLM's multiple-use mandate.

The Primm Valley Golf Club is a golf course located 0.5 mile east of the project area. There are no residential units associated with the golf course. However, the golf course has several water features. The closest community is the town of Primm on the Nevada side of the state line. A retail and casino center along the I-15 corridor, with only a few residential facilities for casino employees, is located approximately 4.5 miles northeast of the project area. The town of Jean, Nevada is located approximately 15 miles north of Primm along I-15. The southern outskirts of greater Las Vegas are approximately 32 linear miles north-northeast of the project area.

The Ivanpah SEGS project is located approximately three miles south of the Mojave National Preserve but is not located within or immediately adjacent to a BLM-designated Desert Wildlife Management Area (DWMA), area of critical environmental concern (ACEC), or Wildlife Habitat Management Areas (WHMAs); or U.S. Fish and Wildlife Service (Service) designated critical habitat.

Prior to conducting surveys, an assessment of habitat conditions was conducted to determine the likelihood of special status wildlife species occurrence. The presence or potential presence of biological resources was determined from information gathered from field surveys of the project area, published and unpublished literature, and natural resource agency databases. Results of this pre-field investigation were compiled into a target species table that was used to guide and focus the survey effort.

Surveys conducted for biological resources in the project area focused on threatened, endangered, and other special-status wildlife species that could potentially occur onsite. Field surveys included general reconnaissance, USFWS protocol-level desert tortoise surveys, western burrowing owl surveys, and winter and spring bird surveys. Other species were also searched for during the general reconnaissance and USFWS protocol-level surveys, including: American badger, roosting bats, and nesting and migratory birds, including the burrowing owl.

The surveys included the Ivanpah SEGS site, an area one mile from the plant site, the areas within 1,000 feet of either side of the proposed utility alignments, and desert tortoise zones of influence (ZOI) transects that extended beyond the project boundary. Results of the special status wildlife surveys and conservation measures that will be implemented to avoid impacts to these species are described in the following sections.

B. Special-Status Wildlife Species

Desert Tortoise

There is only one federal and state threatened or endangered species on the Ivanpah site, the Desert Tortoise. The Desert Tortoise is listed as Federally Threatened and California Threatened. No other federal or state threatened or endangered animal or plant species are one site.

The Ivanpah SEGS site is not located within critical wild lands nor is it located within one of the last habitats of any endangered species. The only wildlife species present that is protected by either the State or the Federal Endangered Species act is the desert tortoise. This is both a State and Federally listed Threatened species. The U.S. Fish and Wildlife Service (USFWS) is the federal agency responsible for protecting this species and its habitat. One primary tool for protection is the designation of critical habitat. On February 8, 1994, the USFWS designated ***6.4 million acres*** as critical habitat within 12 critical habitat units[1] for the desert tortoise in portions of California, Nevada, Arizona, and Utah. Critical habitat is designated to identify the key biological and physical needs of this species and *key areas* for recovery. Conservation actions are focused within these areas. The Ivanpah SEGS project is not located within those 6.4 million acres and is by no means in an area critical to the survival of this species.

In 1990, USFWS developed the *Desert Tortoise (Mojave Population) Recovery Plan*. As part of this plan, six population units, called "recovery units," were identified using published and unpublished data on genetic variability, morphology, and behavior patterns of populations as well as ecosystem types.2 The location of the proposed Ivanpah SEGS project is not within protected habitat for the desert tortoise nor does it contain a dense population of desert tortoises within its 6.3-square-mile boundary. Although the BLM and USFWS have consistently considered the Northern Ivanpah Valley Unit to be good tortoise habitat, they have not found it suitable for inclusion in a Desert Wildlife Management Area (DWMA), Area of Critical Environmental Concern (ACEC), or critical habitat primarily due to isolation by I-15 and the surrounding highlands, the small size of the area, existing development (e.g., the Primm Valley Golf Club), and development pressure.

At the time of its inception, the Ivanpah DWMA (located south of I-15) was determined to contain between 5 and 250 tortoises per square mile. About 20 square miles of that area

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[1] Federal Register, Vol. 59, No. 26, Feb. 8, 1994: 5820-5866; http://ecos.fws.gov/docs/federal_register/fr2519.pdf

~~2 http://www.tortoise-tracks.org/publications/berry2.html~~

supported densities of 200 to 250 tortoises3 compared to the project site, which has a density of less than 5 per square mile.

In the Final EIS for the NEMO, the BLM has designated the Ivanpah site the lowest habitat value as Category III. [BLM classifies habitat as Category I (prime habitat value), Category II (moderate habitat value) and Category III (lowest habitat value).] It is true that the Ivanpah Valley contains some area of high-quality desert tortoise habitat. However, it is important to distinguish between (1) this general statement about the entire Ivanpah Valley and (2) the specific statements in the NEMO regarding the Ivanpah Solar Project site.

For areas like the Ivanpah site that are located outside of Areas of Critical Environmental Concern and outside "critical habitat" for endangered species, the BLM's Final EIS for the NEMO calls for a 1:1 mitigation ratio, indicating the lowest quality habitat:

> *Compensation shall be required by BLM for disturbances of desert tortoise habitat at the rate of 1 acre for each acre disturbed*; this is the same as the current requirement in BLM's Desert Tortoise Statewide Management Policy. Funds collected from project proponents shall be directed to habitat enhancement, rehabilitation or acquisition in the Eastern Mojave Recovery Unit. Proponents may also implement enhancement or rehabilitation projects or donate lands directly, at BLM discretion. (BLM Final EIS for NEMO, p. A-18, emphasis added.)

Thus, in considering the Ivanpah site, it is critical to focus on (1) the site specific determinations made by BLM in the NEMO Final EIS and (2) all other areas in the Ivanpah Valley.

Only twenty-five (25) live Desert Tortoises were encountered on the 4,062 acre Ivanpah Solar Project Site during the 2007 and 2008 USFWS protocol tortoise surveys. USFWS recommends a maximum Desert Tortoise density of 39 Desert Tortoise per Square Kilometer. (USFWS 2008b.) The Ivanpah Solar Project site is approximately 16.45 Square Kilometers. Based on USFWS's recommended maximum density, the Ivanpah site could support six hundred fifty-one (651) Desert Tortoise, not twenty-five (25). This is twenty-six times the number of Desert Tortoises actually found during on-the-ground surveys of the Project site.

Other Special Status Species

An evaluation of impacts to other special-status species (i.e., plant and animal species other than federal- and state-listed species under the federal and California Endangered Species Acts) was conducted, including evaluation of the following: species proposed for those listings; federal Candidate species, federal Species of Concern; California Species of Special Concern; California Fully Protected Species under the Fish and Game Code; and plant species designated as Rare, Threatened, or Endangered by the California Native Plant Society (CNPS).

The surveys included the Ivanpah SEGS site, an area one mile from the plant site, the areas within 1,000 feet of either side of the proposed utility alignments, and desert tortoise zones of influence (ZOI) transects that extended beyond the project boundary. Results of the special status wildlife surveys and conservation measures that will be implemented to avoid impacts to these species are described in the following sections.

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[3] Desert Tortoise (Mojave Population) Recovery Plan, Appendix F at http://ecos.fws.gov/docs/recovery_plans/1994/940628.pdf

Table BR-1 presents the special-status wildlife species that were identified onsite and those presumed to occur. A total of 25 live desert tortoises were identified during the protocol-level desert tortoise surveys. Other finds included 97 desert tortoise carcasses, 214 burrows, and 50 other tortoise sign (BSE 2007a). Tortoise sign and density was greatest in Ivanpah 1 at the southern boundary of the project site and was less dense as the survey moved towards the Clark Mountains and Ivanpah 3. No other federally- or state-listed wildlife species were identified at the project site. Banded Gila monster has been recorded in the project vicinity but this species has not been documented onsite. Because the banded Gila monster is very difficult to detect, even under the best of conditions, it is presumed present.

Several migratory birds were observed during the surveys and likely use the project area for foraging and nesting. These include: burrowing owl, golden eagle, Vaux's swift, loggerhead shrike, Brewer's sparrow, Crissal thrasher, and Le Conte's thrasher. Other raptors such as eagles, hawks, and falcons may use the site for foraging but are not expected to nest onsite due to a lack of nest areas other than the nearby transmission line towers. Bat species roost in caves and crevices in the surrounding mountains and may forage within the project site. Bighorn sheep occur in the surrounding mountains, and they may, on rare occasions, forage or move through parts of the project site. The American badger, a California species of special concern, was observed within the project site.

TABLE BR-1
Species Potential Distribution and/or Suitable Habitat in the Ivanpah Solar Electric Generating System Project Area

Species Name	Status*	Project Impacts
Desert Tortoise *Gopherus agassizii*	FT, CT	Twenty-five live desert tortoises were identified during protocol level surveys. Project development will result in the temporary loss of approximately 4,062 acres of desert tortoise habitat. However, the site is not lost "in perpetuity," since the Applicant must restore the project site at the end of the Right of Way Grant and provide the BLM with a bond as security for site restoration.
Banded Gila monster *Heloderma suspectum cinctum*		There are few records of this species in California and none on the Ivanpah site.
Western burrowing owl *Athene cunicularia hypugaea*	FSC, CSC, MB	Burrowing owl sign has been observed within the project area during the 2008 surveys, but not during the 2007 surveys. Conservation measures will be implemented to minimize and avoid the potential for burrowing owls to be harmed during construction and operation.
Golden eagle *Aquila chrysaetos*	FSC, CSC, FP, BLM SS	Golden eagles were detected on the ISEGS project site, but are unlikely to nest there because of the absence of suitable nesting habitat. Conservation measures will be implemented to minimize and avoid the potential for birds to be harmed during construction and operation.
Migratory birds including: Loggerhead shrike *Lanius ludovicianus* LeConte's thrasher *Toxostoma lecontei* Bendire's thrasher *Toxostoma bendirei* Crissal thrasher *Toxostoma crissale*		Various migratory birds have been observed within the project area and on site. Conservation measures will be implemented to minimize and avoid the potential for birds to be harmed during construction and operation.

TABLE BR-1
Species Potential Distribution and/or Suitable Habitat in the Ivanpah Solar Electric Generating System Project Area

Species Name	Status*	Project Impacts
Brewer's sparrow *Spizella breweri*		
Bats including: Townsend's big-eared bat *Corynorhinus townsendii* Pallid bat *Antrozous pallidus* Long-legged myotis *Myotis volans*		Bats are observed roosting in the surrounding mountains and hills, but not on the project site.
Nelson's bighorn sheep *Ovis canadensis nelsoni*	FSS, BLM SS	Nelson's Bighorn Sheep have been observed in the Clark Mountains, but not on the project site.
American badger *Taxidea taxus*	CSC	The American Badger was found on the project site during surveys Conservation measures will be implemented to minimize and avoid the potential for badgers to be harmed during construction and operation.

* Federal-, state-, and CNPS-listed species:
CE: California Endangered
CSC: California Species of Special Concern
FE: Federally Endangered
FT: Federally Threatened
FP: California Fully Protected Species
SC: Federal Species of Concern
1A: CNPS-Presumed Extinct in California
MB: Migratory Bird Treaty Act

Project Area

The Ivanpah SEGS site is primarily dominated by a Mojave Creosote Bush Scrub vegetation community with assemblage of cacti and annuals. The site is accessed by dirt roads and has been used to various degrees for recreation, Off-road vehicle (OHV) trains and livestock grazing, among other activities permitted by BLM's multiple-use directives. The site provides habitat for variety of common and special-status plant and wildlife species of the western Mojave Desert.

Based on the initial assessment of the habitat, more focused investigations were conducted for desert tortoise, American badger, roosting bats, and nesting and migratory birds, including the burrowing owl. As a result of these surveys it was concluded that these species do have the potential to occur within the project area. In addition to the federal and state listed desert tortoise, other special-status species confirmed within the project area included: burrowing owl, golden eagle, Vaux's swift, loggerhead shrike, Brewer's sparrow, Crissal thrasher, Le Conte's thrasher, and American badger.

Twenty-five tortoises were identified during the surveys. The banded Gila monster was not found on-site and is considered rare in California. A variety of migratory birds were observed and were likely using the project area for foraging and nesting. Burrowing owl sign was observed on site and this species may be encountered nesting on the site in the spring and summer or as a wintering bird. Other raptors such as eagles, hawks, and falcons may use the site for foraging due to a lack of obvious potential nesting opportunities other than the nearby transmission line towers. Bat species roost in caves and crevices in the surrounding mountains and may forage within the project site.

Big horn sheep do occur in the surrounding mountains, but not on the project site. Applicant also disagrees with the conclusion that the impacts to the Nelson's Bighorn Sheep are significant. Notwithstanding these facts, Applicant has made initial contacts with the Society for the Conservation of Bighorn Sheep and communicated Applicant's willingness and commitment to work with the Society in installing one or more artificial water sources for Nelson's bighorn sheep, outside the regulatory process.

Construction Impacts to Wildlife

Project development will result in the temporary loss of approximately 4,062 acres of habitat for variety of common and special-status wildlife species. However, the site is not lost "in perpetuity," since the Applicant must restore the project site at the end of the Right of Way Grant and provide the BLM with a bond as security for site restoration. Wildlife will be directly and indirectly impacted by the physical clearing of the site. Wildlife occurring in the site will be impacted or displaced. Those occurring adjacent to the site may be temporarily impacted by the construction activity levels, noise, increased vehicle traffic, dust, night-time lighting, and habitat fragmentation. Without appropriate mitigation and conservation measures, the increased construction activity may also attract or provide subsidized resources for an increased number of native and non-native predators. Operation activities may affect the normal behavior of various wildlife species and the high heat produced by the concentration of solar energy has the potential to cause serious harm to birds that fly between the receiving tower and the mirror arrays.

The project includes design features that are intended to minimize and avoid impacts to listed species, special status, and common species. The Applicant will also implement a comprehensive list of conservation measures to minimize and avoid the indirect and direct impacts during construction and operation. These include preconstruction desert tortoise clearing and relocation, typical environmental awareness and biological monitoring as well as funding of desert tortoise recovery actions. Desert tortoises will be removed from the site and relocated to appropriate habitat nearby. Tortoise relocation will include post relocation monitoring through and agency-approved plan. Efforts will be made to properly relocate and/or exclude other encountered wildlife such as burrowing owls, badgers, and Gila monsters. Some tortoise recovery actions have the potential to also benefit other wildlife species that will be affect by the proposed project. Areas of temporary effects will undergo restoration immediately following construction as outlined in the Closure, Revegetation and Rehabilitation Plan (CRRP). The CRPP also describes revegetation and rehabilitation activities that will be implemented for the project site following decommission.

Potential project impacts associated with operation of the Ivanpah SEGS were evaluated to determine whether biological resources would be significantly affected. Potential direct and indirect impacts associated with operation of Ivanpah SEGS and the mitigation measures designed to avoid or minimize those potential impacts include the following and other measures described in the Applicant's testimony:

- Transmission lines and poles will be designed and constructed with appropriate spacing between conductors and/or bonding wires to avoid electrocution of large birds, as described in APLIC 1996 "raptor-friendly" guidelines (APLIC 1996).

- Generally, continuous low noise levels from operations does not adversely impact wildlife, as wildlife usually becomes accustomed to routine background noise. Bright night lighting could disturb wildlife (e.g., nesting birds, foraging mammals, and flying insects). Night lighting is also suspected to distract and/or attract migratory birds to areas and, if the lights are on tall structures, collisions could occur. The area is not within migratory pathways, and lighting would be low on the structures, pointed downwards, and hooded to minimize impacts.
- Operations could attract increased numbers of native and non-native predators. The Applicant will exclude wildlife from water collecting basins, contain food-related trash, and implement an agency-approved raven control plan.

Cumulative Impacts to Wildlife

In conjunction with the increased planned development of the Ivanpah Valley including additional renewable energy development, new airport construction, proposed high speed rail corridors, road improvement, additional casino development, and associated infill, Ivanpah SEGS will have cumulative impacts on the natural habitat of the western Mojave Desert and the species that occur there.

Permitting Overview

Applicable Federal, State, and local LORS are shown in Table 5.2-1 of the AFC. These LORS were reviewed and consultations with the appropriate agencies were made to determine if the proposed project could affect sensitive biological resources. Through on site field surveys, agency consultations and guidance, project design modifications, and proposed protection measures, the Ivanpah SEGS project will conform to all applicable LORS for protection of biological resources.

The desert tortoise is a federally-listed threatened species and formal consultation with the USFWS is required to comply with the federal Endangered Species Act. Section 7 consultation has been initiated through the preparation and submittal of a Biological Assessment (BA) that describes the proposed project and effects of the project to the USFWS. Following review of the BA, the USFWS will issue a Biological Opinion (BO) that will specify mitigation measures that must be implemented for the protection of the desert tortoise.

Measures as outlined in the Conditions of Certification BIO-6, 8, 9, 11, 12, 13, and 14 will avoid and minimize impacts to non-listed special status wildlife species. These are summarized below, and in detail, later in this testimony.

Mitigation for Wildlife

Proposed conservation protection measures to avoid and minimize impacts to biological resources within and adjacent to the Ivanpah SEGS project area would include:

1. Conduct Worker Environmental Awareness Training for all construction personnel.
2. Conduct construction monitoring by a qualified Designated Biologist and onsite Biological Monitors during construction activities near sensitive habitats.
3. Prepare a Biological Resources Mitigation Implementation and Monitoring Plan (BRMIMP) that details how the Applicant would implement any protection measures or conditions of permits developed to ensure that actions authorized, funded, or carried out by state or

federal lead agencies are not likely to jeopardize the continued existence of endangered or threatened species.

4. An agency-approved desert tortoise relocation plan and raven control plan would be adopted and implemented.
5. Funding will be provided to the BLM to implement desert tortoise recovery actions at a ratio of 1:1, and an additional 2:1 mitigation will be provided for a total mitigation ratio of 3:1.

C. Botany

Rare Plant Survey Methods

Protocol-level surveys for special-status plants were conducted throughout the project area in spring and early summer of 2007 and spring of 2008. Reconnaissance-level surveys of the one-mile buffer area surrounding the project site were performed in 2007. The protocol-level surveys for special-status plants were floristic in nature and followed the USFWS Guidelines (USFWS 1996a) and the recommendations of the CDFG (CDFG 2000) and CNPS (CNPS 2001).

As part of protocol level surveys, reference site visits to known special-status plant populations were performed in 2007 and 2008 to determine the progress of the growing season and to orient key team members to characteristics necessary for correct identification. In a few cases, reference population checks were performed in October and November of 2007, and April and May of 2008, outside of the main field survey efforts, to confirm species identifications or view known populations of special-status plants in the project vicinity.

Vegetation types within the project area were classified according to Holland (1986) and mapped in 2007. Vegetation within the project area includes Mojave Creosote Bush Scrub (including four subtypes), Mojave Yucca – Nevada Ephedra Scrub, and Mojave Wash Scrub. The predominant vegetation throughout the project area is the Larrea-Ambrosia subtype of Mojave Creosote Bush Scrub. Limestone features, which occur mainly in the one-mile buffer, are vegetated by the limestone-associated Larrea scrub subtype and Mojave Yucca – Nevada Ephedra Scrub. Larger ephemeral wash drainage features are vegetated with Mojave Wash Scrub.

The 2007 and 2008 surveys also included invasive weeds. Few weeds were found in 2007 because it was a very dry year. In 2008, a wetter year, five species of weeds were mapped and documented. Red brome (*Bromus madritensis* ssp. *rubens*) was the most commonly encountered weed. It was widespread throughout the project area, but dense concentrations were found only in the northern and northwestern parts of the project area. The other four weed species: Saharan mustard (*Brassica tournefortii*), cheat grass (*Bromus tectorum*), Russian thistle (*Salsola* sp.), and London rocket (*Sisymbrium irio*)) were each found in fewer than five locations, in low numbers.

A census of all individuals of California barrel cactus (*Ferocactus cylindraceus* var. *lecontei*) and clustered barrel cactus (*Echinocactus polycephalus* var. *polycephalus*) was completed throughout the project area in 2007 and 2008. A total of 2,869 individuals of California barrel cactus and 3,501 individuals of clustered barrel cactus were mapped within the project area.

Rare Plant Survey Results

Nine special-status plant species were identified in the project area. These are described in detail in the 2008 Rare Plant Survey Report prepared by GANDA (2008). None of the special status plant species identified onsite are federally or state-listed. The CEC Staff recommendations focus on five species designated as "rare" by the California Native Plant Society: Mojave milkweed, desert pincushion, nine-awned pappus grass, Parish's club-cholla, and Rusby's desert mallow. The total number of individuals and localities for each of these five species is summarized in the following table.

TABLE BR-2
Numbers of Special-Status Plant Individuals and Localities – Ivanpah SEGS

Species	Total Number of Individuals	Total Number of Localities
Mojave milkweed (*Asclepias nyctaginifolia*)	202	60
Desert pincushion (*Coryphantha chlorantha*)	599	291
Nine-awned pappus grass (Enneapogon desvauxii)	8,145	182
Parish's club-cholla (*Grusonia (=Opuntia) parishii*)	339	143
Rusby's desert mallow (*Sphaeralcea rusbyi var. eremicola*)	15	12

Notes:
Data for 2007 and 2008 are combined in this summary table

Special status plant species for which impacts are not considered to be significant are not included in this table. Refer to the 2008 Rare Plant Survey Report (GANDA 2008) for more detail.

Only one of the five special-status plant species, Rusby's desert mallow, is considered sensitive by the Bureau of Land Management (BLM).

Areas of Controversy

As outlined in Section 6.2 of the FSA/DEIS, and in Condition of Certification BIO-18, the mitigation approach preferred by the CEC Staff emphasizes avoidance and protection of rare plant localities. As discussed later in this testimony, Applicant proposes similar avoidance and minimization measure sin Applicant's revised BIO-18. Complete loss of significant portions of Ivanpah 1 and 3 such as that suggested in FSA/DEIS Biological Resources Figure 2 is not possible without compromising design to an unacceptable level. The number of heliostats Staff proposes to remove from the project and the location of those heliostats on the northern portions of Ivanpah 1 and 3 would make the project infeasible.

The Applicant has proposed avoidance and minimization strategies in BIO-18, as revised, that will minimize and avoid potential impacts to special status plants in these areas. The best opportunities for rare plant protection are within the construction logistics area and the utility corridor.

The applicant will ask the Staff to meet and confer to provide the opportunity to work with CEC and BLM staff to develop a more species-specific rare plant mitigation plan that will provide avoidance and minimization while allowing the project to be built in a feasible configuration.

Regarding the BLM-approved Weed Management Plan (WMP), it was developed following templates provided by the BLM, and subsequently review comments were received from the BLM's weed management specialist and incorporated into what is considered the approved WMP moving forward. Results of the biological resources surveys were factored into the WMP, and the potential for additional invasive discussed and incorporated into WMP action protocols.

Botany References

California Department of Fish and Game (CDFG). 2000. Guidelines for assessing the effects of proposed projects on rare, threatened and endangered plants and natural communities. May 8, 2000. Available at: http://www.dfg.ca.gov/whdab/pdfs/guideplt.pdf

California Native Plant Society (CNPS). 2001. Inventory of rare and endangered plants of California. California Native Plant Society. Special Publication #1, Sixth Edition.

Holland, R. F. 1986. Preliminary descriptions of the terrestrial natural communities of California. California Department of Fish and Game. Unpublished report.

U.S. Fish and Wildlife Service (USFWS). 1996a. Guidelines for Conducting and Reporting Botanical Inventories for Federally Listed, Proposed, and Candidate Plants. USFWS, September 23, 1996. Available at: http://www.fws.gov/sacramento/es/documents/listed_plant_survey_guidelines.htm

D. Waters of the U.S. (Wetlands)

Survey Methods

A preliminary data review using aerial photographs and other data sources combined with a site reconnaissance survey in March 2007 identified numerous west to east trending ephemeral washes throughout the project area. Given the size of the project area and the myriad of features present, drainages were characterized and mapped by a combination of field work and office review of high resolution aerial photography. The field surveys were conducted along linear transects established approximately 1,000 feet apart that were configured roughly perpendicular to the ephemeral drainages. Prior to field work, input and approval of the methodology was solicited from the U.S. Army Corps of Engineers (USACE).

The formal wetland delineation was conducted from April 16 through 20, and May 21 through 24, 2007. The total survey area delineated included Ivanpah 1, 2, and 3 as well as a 1,000-foot buffer area for each of the three project sites, access roads, and linear utility corridors. Data were recorded using GPS at each point where an ephemeral wash intersected the transect line.

Data collected included general characteristics of the wash, including average channel width, evidence of flow, and general vegetation. Field data were then incorporated into a geographic information system (GIS). Data points collected along the transect lines were plotted on recent aerial photographs, with 2-foot resolution, and the drainage features within the survey area were manually digitized using the field data as reference locations. The project boundary was slightly modified in 2008 and the additional washes in the new area were digitized based on high-resolution aerial photographs from 2008. The ephemeral drainages, by size category, and more detail on the wetland delineation methodology employed, is contained in the Final Wetland Delineation Report (2008).

Based on the field data, each wash was then assigned a size category class between 1 and 5. Category 1 washes are large ephemeral drainages over 36 feet wide. The largest category 1 wash mapped was 85 feet wide. Category 2 washes are relatively large ephemeral drainages over 20 feet wide and no more than 35 feet wide. Category 3 washes are over 10 feet wide and no more than 20 feet wide. Category 4 includes ephemeral washes over 4 feet wide and no more than 10 feet wide. Category 1, 2, 3 and 4 washes include single, large channels with well-defined bed and banks, as well as broad, but weakly expressed, assemblages of braided erosional channels. Category 5 includes weakly expressed erosional/flow channels that generally lack defined cut banks and are no more than 4 feet wide. The approximate acreage for each of the wash Categories was calculated and tallied for the project.

Survey Results

The entire study area is dissected by numerous ephemeral washes ranging in size from small (1 to 4 feet wide), weakly expressed erosional features to large, broad (over to 85 feet wide) drainages (Table BR-4). The active flow channels of the smaller washes are generally devoid of vegetation and typically have a sandy-gravel substrate, although some washes also contained cobble and scattered larger rocks. Most of the larger channels typically contained scattered vegetation including creosote bush and cheesebush, especially those in braided channels that contained slightly elevated areas intermixed with the active flow channels. Mojave wash scrub habitat is limited to the larger washes (typically over 15 feet) with sandy gravel substrate and well-defined banks.

TABLE BR-4
Summary of Ephemeral Washes Identified in the Project Study Area

Wash Category	Project Feature	Number of Washes*	Wash Length (feet)	Wash Acreage
Category 1 (36-85 feet)	Category 1 Total	8	13,559	16.78
Category 2 (21-35 feet)	Category 2 Total	12	12,953	8.22
Category 3 (11-20 feet)	Category 3 Total	94	113,446	40.37
Category 4 (5-10 feet)	Category 4 Total	459	428,083	73.71
Category 5 (1-4 feet)	Category 5 Total	1,400	970,129	55.68
	All Categories (Total)	**1,973**	**1,538,170**	**198.72**

Notes:
*Number of washes is approximate.
**Acreage calculated using Wash Length and the mean width of the Category range (i.e. Category 3 has a mean width of 15.5 feet)
No wetlands were observed within the entire project area.

Approximately 198.72 acres of ephemeral washes were identified and mapped in the project area. These include: 1) 16.78 acres of category 1 washes, 2) 8.22 acres of category 2 washes, 3) 40.37 acres of category 3 washes, 4) 73.71 acres of category 4 washes, and 5) 55.68 acres of category 5 washes. Small- to medium-sized washes are common and widespread throughout the entire project area, while the larger washes (categories 1, 2 and 3) are most abundant in the northern section of Ivanpah 3 as well as the east and west sides of Ivanpah 2. The larger washes tend to dissipate into smaller, more braided channels as they progress downslope. The majority of the drainages terminate prior to reaching Ivanpah Dry Lake with defined erosion features diminishing and becoming broad surface flow only. All of the ephemeral washes identified in the study area typically flow only in response to storm events. No wetlands or perennial water were observed within the project area.

Permitting and Areas of Controversy

The Regional Water Quality Control Board (RWQCB), U.S. Army Corps of Engineers (USACE), and California Department of Fish and Game (CDFG) were contacted regarding the extent of their jurisdiction over the drainage features within the project site, and to determine the permitting requirements for the project. In May 2009, the USACE determined that the drainages onsite are not waters of the United States (U.S.). However, the drainages affected by the Project are waters of the State, as defined by California Water Code (Water Code) section 13050, and are subject to State requirements in accordance with Water Code section 13260 and to the Water Quality Control Plan for the Lahontan Region (Basin Plan).

All actions impacting or potentially impacting these drainages, including dredge and fill activities and construction and industrial activities, will be regulated through these requirements, which will be incorporated in the Energy Commission's certification process. The Applicant will comply with stormwater control measures as outlined in the state water board's General Permit No CAS00002, Waste Discharge Requirements for Discharges of Storm Water Runoff Associated with Construction Activity.

In addition to regulation by the RWQCB, the California Department of Fish and Game (CDFG) has the authority to protect water resources of the state through regulation of modifications to streambeds, under Section 1602 of the Fish and Game Code. The applicant filed a Streambed Alteration Agreement (SAA) with CDFG on June 2, 2009 and the requirements of the SAA are included in the CEC's recommended Conditions of Certification.

To address the potential for storm water and sediment project-related impacts, Condition of Certification SOIL&WATER-5 has been developed by the CEC. This Condition of Certification defines monitoring, inspection, and damage response requirements, as well as standards and procedures for re-considering the proposed storm water management approach if needed in the future.

On May 28, 2009 the USACE issued a formal determination that there are no jurisdictional waters of the United States or adjacent wetlands in the proposed project area and therefore a Clean Water Act Section 404 permit would not be required. The USACE determined that all of the ephemeral washes on the site are non-relatively permanent waters that convey flow only in response to storm events. It was determined that the ephemeral washes do have a physical and hydrological connection to Ivanpah Lake; however, there is no downstream connectivity to a traditional navigable water or significant nexus to any water related interstate or foreign commerce associated with the lake or any of the ephemeral washes.

Staff's recommendations in proposed Condition of Certification BIO-20 requires compensation for impacts to 198 acres of waters of the state. Applicant believes this requirement is not supported by the record. The Applicant's Low Impact Design (LID) means these 198 acres of state water are not lost. Indeed, it is clear that the Staff still considers the washes "water of the state", even after the installation of the heliostats through the Low Impact Design. Put another way, if Staff insists that then 198 acres of washes be treated as "state water" after construction of the project using the Low Impact Design, these state waters are not "lost" and thus there is no loss to mitigate.

Notwithstanding that waters of the state will remain with the implementation of the he Low Impact Design, the applicant will ask the Staff to meet and confer to provide the opportunity to work with CEC and BLM staff to develop to discuss the disagreement on this issue.

References

California Department of Fish and Game (2009) Notification of Lake or Streambed Alteration. Submitted to the Eastern Sierra and Deserts (Region 6) by CH2M HILL on June 2, 2009.

CH2M HILL (2008). Delineation of Waters of the United States for the Ivanpah Solar Energy Project: Eastern San Bernardino County, California. Final Version as Revised September 2008.

California Regional Water Quality Control Board, Lahontan Region (2009). Application for Clean Water Act §401 Water Quality Certification and/or Waste Discharge Requirements for Projects Involving Discharge of Dredged and/or Fill Material to Waters of the U.S. and/or Waters of the State. Submitted by CH2M HILL.

United States Army Corps of Engineers (USACE), 2009. Formal Jurisdictional Determination for the Ivanpah Solar Energy Project, San Bernardino County, California. File No. SPL-2007-00415-SLP. May 28, 2009.

III. Proposed Licensing Conditions

The FSA/DEIS for the project filed by the CEC and BLM recommends that 20 Conditions of Certification be adopted to address Biological Resource issues: BIO-1 through BIO-20. The Applicant proposes the changes to the following conditions, BIO-6, BIO-8, BIO-9, BIO-11, BIO-12, BIO-13, BIO-14, BIO-17, and BIO-18. Subject to our general comments below and the Applicant's Prehearing Conference Statement, the others are acceptable.

Proposed Revisions to BIO-6

As a general comment that applies to several of these conditions, the Applicant is opposed to conditions that require separate approvals of post-certification compliance activities by both BLM and the CPM because they are simply unworkable. If the approval is sequential, it will result in doubling the required approval time for everything. If the approval is concurrent, approvals may be potentially conflicting. As a general rule, consistent with current Commission practice, we have identified the Commission's CPM as the authority to review and approve post-certification compliance submissions or actions of the Applicant. It is also imperative that specific timeframes for approval be included in the Conditions so that the project will not be unnecessarily delayed. See the applicant's pre-hearing conference statement for a more-detailed explanation. In addition, the Applicant is concerned that the requirement to provide WEAP material "in the language best understood by the participants" is impermissibly vague and ambiguous.

BIO-6 The project owner shall develop and implement an Ivanpah SEGS-specific Worker Environmental Awareness Program (WEAP) and shall secure approval for the WEAP from ~~USFWS, CDFG, BLM's Authorized Officer and~~ the CPM. The WEAP shall be administered to all onsite personnel including surveyors, construction engineers, employees, contractors, contractor's employees, supervisors, inspectors, subcontractors, and delivery personnel. The WEAP shall be implemented during site mobilization, ground disturbance, grading, construction, operation, and closure. The WEAP shall:

1. Be developed by or in consultation with the Designated Biologist and consist of an on-site or training center presentation in which supporting written material and electronic media, including photographs of protected species, is made available to all participants. ~~The training presentation shall be made available in the language best understood by the participants;~~

* * *

Proposed Revisions to BIO-8

The Applicant is concerned about subpart 2.c. The language blurs any difference between temporary fencing that may only need to be in place for a few days and permanent fencing.

a. Utility Corridor Fencing. The utility rights-of-way shall be temporarily fenced on each side of the right-of-way prior to ground disturbing activities to prevent desert tortoise entry during construction. Temporary fencing ~~must follow guidelines for permanent fencing and supporting stakes shall be sufficiently spaced to maintain fence integrity~~must be able to prevent tortoise from entering the work area.

Proposed Revisions to BIO-9

Proposed changes to BIO-9 clarifies agency roles.

DESERT TORTOISE TRANSLOCATION PLAN

BIO-9 The project owner shall develop and implement a final Desert Tortoise Relocation/Translocation Plan (Plan) that is consistent with current USFWS approved guidelines, and meets the approval of BLM's Authorized Officer and the CPM, in consultation with CDFG and USFWS~~BLM, USFWS, CDFG and Energy Commission staff~~. The final Plan shall be based on the draft Desert Tortoise Relocation/Translocation Plan prepared by the applicant dated May 2009 and shall include all revisions deemed necessary by BLM's Authorized Officer and the CPM, in consultation with CDFG and USFWS~~BLM, USFWS, CDFG and the Energy Commission staff~~.

Verification: Within 60 days of publication of the Energy Commission Decision the project owner shall provide BLM's Authorized Officer and the CPM with the final version of a Desert Tortoise Relocation/Translocation Plan that has been reviewed ~~and approved~~ by BLM, USFWS, CDFG and ~~Energy Commission staff~~the CPM. BLM's Authorized Officer and the CPM will determine the plan's acceptability within 15 days of receipt of the final plan. All modifications to the approved translocation must be made only after consultation with BLM's Authorized Officer~~,~~ and the CPM, in consultation with ~~,~~ USFWS, and CDFG. ~~The project owner shall notify BLM's Authorized Officer and the CPM no fewer than 5 working days before implementing any BLM- and CPM-approved modifications to the Plan.~~

Proposed Revisions to BIO-11, item 13

The Applicant requests that the disposal of roadkill animals condition be revised to add that roadkill only should be disposed of by biological monitors to ensure proper species identification. Also, due to potential scientific value of the carcass, the Applicant requests that special-status species roadkill be held by the biologists until they receive direction from the California Department of Fish and Game and/or the Fish and Wildlife Service as to its disposition.

13. Dispose of Roadkilled Animals. Road killed animals or other carcasses detected in the project area or on roads near the project area shall ~~be picked up~~ immediately be reported to a biological monitor. The BM shall determine the species of the roadkill and maintain the item until the BM receives direction from the California Department of Fish and Game and/or the Fish and Wildlife Service as to its disposition. ~~upon detection and appropriately disposed of to avoid attracting common ravens and coyotes.~~

Proposed Revisions to BIO-12

RAVEN MANAGEMENT PLAN

BIO-12 The project owner shall implement a Raven Management Plan that is consistent with the most current USFWS-approved raven management guidelines, and which meets the approval of BLM's Authorized Officer and the CPM, in consultation with CDFG and USFWS~~USFWS, CDFG, BLM, and the Energy Commission staff~~. The draft Raven Management Plan submitted by the applicant (CH2M Hill 2008f) shall provide the basis for the final plan, subject to review and revisions by the BLM's Authorized Officer and the CPM, in consultation with CDFG and USFWS~~from USFWS, CDFG, BLM, and the Energy Commission staff~~.

Verification: At least 60 days prior to start of any project-related ground disturbance activities, the project owner shall provide BLM's Authorized Officer, the CPM, USFWS, and CDFG with the final version of a Raven Management Plan that has been reviewed by USFWS, CDFG, BLM, and the Energy Commission staff. The CPM and BLM's Authorized Officer will determine the plan's acceptability within 15 days of receipt of the final plan. All modifications to the approved Raven Management Plan shall be made only after consultation with BLM's Authorized Officer and the CPM, in consultation with CDFG and USFWS~~BLM and Energy Commission staff, USFWS, and CDFG~~. ~~The project owner shall notify BLM's Authorized Officer and the CPM no less than 5 working days before implementing any BLM- and CPM-approved modifications to the Raven Management Plan.~~

Within 9~~3~~0 days after completion of project construction, the project owner shall provide to the CPM for review and approval, a written report identifying which items of the Raven Management Plan have been completed, a summary of all modifications to mitigation measures made during the project's construction phase, and which items are still outstanding.

Proposed Revisions to BIO-13

The Applicant has already made its draft and draft-final review copies of its Weed Management Plan available to all agencies. Despite review times exceeding 6 months, only the BLM has responded with actionable input, and that was incorporated in the current, accepted Weed Management Plan. For this reason, and because the current, accepted Weed Management Plan addresses all six BMPs articulated in BIO-13, the following rewording is appropriate to the first two sentences of BIO-13:

BIO-13 The project owner shall implement a Weed Management Plan that meets the approval of BLM and the ~~Energy Commission staff~~CPM. The ~~draft~~ approved Weed Management Plan submitted by the applicant (CH2M Hill 2008e) shall ~~provide the basis for~~be the final plan, subject to ~~review and revisions~~ from review and approval by BLM and Energy Commission staff and review and comment by~~,~~ USFWS, and CDFG. To be considered, comments shall be received within sixty (60) days of the date that a request for comments is made to the agencies by the project owner. In addition to describing weed eradication and control methods, and a reporting plan for weed management during and after construction, the final Weed Management Plan shall include at least the following Best Management Practices to prevent the spread and propagation of noxious weeds:

* * *

Verification: At least 60 days prior to start of any project-related ground disturbance activities, the project owner shall provide BLM's Authorized Officer and the CPM with the final version of a Weed Management Plan~~ that has been reviewed and approved by BLM, and Energy Commission staff, USFWS, and CDFG~~. BLM's Authorized Officer and the CPM will determine the plan's acceptability within 15 days of receipt of the final plan. All modifications to the approved Weed Control Plan must be made only after consultation with ~~the Energy Commission staff, BLM, USFWS, and CDFG~~BLM's Authorized Officer and the CPM, in consultation with CDFG and USFWS. The project owner shall notify the CPM no less than 5 working days before implementing any BLM- and CPM-approved modifications to the Weed Management Plan.

Proposed Revisions to BIO-14

BI0-14 does not acknowledge that the Applicant has provided for review a Closure, Revegetation and Rehabilitation Plan (CRRP) and, since that time, has received and responded to a number of comments on the plan. These include, but are not limited to:

- Establishing an area to be used as a long-term succulent salvage and storage as well as soil stockpiles

- Implementing additional low impact design (LID) measures such as minimizing ground disturbance and erosional potential through restricting site grading
- Committing to long-term monitoring of revegetation success, and wildlife management within the boundaries of the solar fields
- Committing to soil preparation procedures (compaction or decompaction depending on the circumstances) to facilitate revegetation of temporarily disturbed areas

As currently proposed, BIO-14 requires a number of commitments, mitigation measures, and BMPs that are already incorporated in the Applicant's CRRP, and therefore, is concerned that Applicant's current commitments have not been adequately taken into account in developing BIO-14. Moreover, Applicant cannot commit to any goals in the CRRP that are not realistically attainable, or that have experimental objectives. As it is currently presented BIO-14 makes the following requirements:

> "The Plan shall address all issues discussed in **Biological Resources Appendix-A**: *Revisions to Draft Closure, Revegetation and Rehabilitation Plan"*

Appendix A is, however, entitled "Percentage of Statewide Documented Occurrences for Special Status Plant Species in the ISEGS Project."

BIO-14 also points to Biological Resources Appendix B as proving guidance information on performance standards, weed management, monitoring methods, baseline vegetation surveys, and detailed seed testing methods. However, Appendix B is entitled "Issues to address in the Closure, Revegetation and Rehabilitation Plan."

The content of Appendix B is principally weakly adjudicated review comments regarding the CRRP. They include requirements for unnecessary research projects, unrealistic revegetation goals, and display fundamental misunderstandings regarding the biological goals and ecological basis of the current CRRP which were provided in some detail, including supporting research, in a preceding Technical Basis Document (Attachment DR125-1A in Data Response Set 2B, filed on July 22, 2008).

Therefore, to satisfy both the intent and purpose of BIO-14, the Applicant commits to the implementation of the current revised CRRP subject to a feasibility and practicability review of the items provided in current Biological Resources Appendix B "Issues to address in the Closure, Revegetation and Rehabilitation Plan." The Applicant will notify the BLM and CEC within thirty (30) days from approval of this modified COC of goals and methods it considers to be unrealistic, not attainable, or unnecessary to achieve revegetation and restoration. As a baseline, the goals presented in the current revised CRRP will be the Applicant's minimal performance standard and current guidelines for implementation of COC BIO-14.

Therefore, the recommended revised COC BIO-14 should read, in its entirety, as follows:

BIO-14 The project owner shall develop and implement a revised Closure, Revegetation and Rehabilitation Plan (Plan) in cooperation with BLM and Energy Commission staff~~, USFWS and CDFG~~ to guide site restoration and closure activities, including methods proposed for revegetation of disturbed areas immediately following construction and rehabilitation and revegetation upon closure of the facility. This plan ~~must~~ addresses preconstruction salvage and relocation of succulent vegetation from the site to ~~either~~ an onsite ~~or nearby~~ nursery facility for storage and propagation of material to reclaim disturbed areas. In the case of unexpected closure, the plan ~~should~~ assumes restoration activities w~~e~~ould possibly take place prior to the anticipated lifespan of the plant. The Plan ~~shall address all issues discussed in **Biological Resources Appendix A**: *Revisions to Draft Closure, Revegetation and Rehabilitation Plan*, and~~ shall include but is not limited to the following elements in the revised plan:

* * *

Verification: No more than thirty (30) days from the Energy Commission Decision and BLM Record of Decision, the project owner shall provide BLM's Authorized Officer and the CPM with a draft ~~version~~ final of the ~~revised~~ Closure, Revegetation and Rehabilitation Plan, including written responses to all itemized issues in the current Biological Resources Appendix B Issues to Address in the Closure, Revegetation and Rehabilitation Plan. At least sixty (60) days prior to start of any project-related ground disturbance activities, the project owner shall provide BLM's Authorized Officer and the CPM with the final version of the Closure, Revegetation and Rehabilitation Plan that has been reviewed ~~and approved~~ by BLM, USFWS, CDFG, and ~~the Energy Commission staff~~approved by the CPM. All modifications to the approved ~~Revegetation and Reclamation~~ Plan must be made only after consultation with BLM's Authorized Officer, the CPM, USFWS and CDFG. The project owner shall notify BLM's Authorized Officer and the CPM and no less than 5 working days before implementing any ~~BLM- and~~ CPM-approved modifications to the ~~Closure, Revegetation and Rehabilitation~~ Plan.

Within 30 days after completion of project construction for each phase of development, the project owner shall provide to ~~BLM's Authorized Officer and~~ the CPM for review and approval, a written report identifying which items of the ~~Closure, Revegetation and Rehabilitation~~ Plan have been completed, a summary of all modifications to mitigation measures made during the project's construction phase, and which items are still outstanding.

* * *

Proposed Revisions to BIO-17

Applicant's letter of August 7, 2009, to the CEC, attached to Applicant's Prehearing Conference Statement, sets forth the rationale for these changes. (See Ellison, Schneider & Harris 2009. (tn 52788) Letter to J. Kessler, Energy Commission, from J. Harris, Ellison, Schneider & Harris L.L.P, regarding proposal for mitigation, dated August 7, 2009. Submitted to California Energy Commission Docket Unit on August 7, 2009.)

DESERT TORTOISE COMPENSATORY MITIGATION

BIO-17 Delete BIO-17 in its entirety and replace with the following:

BIO-17-F (Federal ESA) The BLM applies a 1:1 compensation ratio because BLM pursues desert tortoise recovery goals not through parcel by parcel acquisitions and management, but rather through implementation of region-wide management plans and land use planning as described in the NEMO, the California Desert Conservation Act plan, and the Desert Tortoise Recovery Plan (USFWS 1994). The Commission understand that BLM's compensatory mitigation will include (1) a per acre assessment for the 4,062 acres or the area disturbed by the final project footprint, (2) a land acquisition fee, and (3) a BLM management fee.

To mitigate for habitat loss and potential take of desert tortoise under the federal Endangered Species Act (ESA), the project owner shall provide to the BLM's Authorized Officer or other appropriate BLM official, compensatory mitigation at a 1:1 ratio at a per acre rate to be determined by the BLM in its sole and absolute discretion.

Verification: The project owner shall submit a report to the CPM at least 30 days prior to the start of any project-related site disturbance activities confirmation from BLM that the project owner has satisfied BLM desert tortoise compensatory mitigation obligations as administered by the BLM.

BIO-17-S (State California ESA) To fully mitigate for habitat loss and potential take of desert tortoise, the project owner shall provide additional compensatory mitigation at a 2:1 ratio for impacts to 4,062 acres or the area disturbed by the final project footprint (for total compensatory mitigation at a 3:1 ratio) at a per acre rate to be determined by the BLM in its sole and absolute discretion. This additional 2:1 shall be either paid in full or, alternatively, financial assurance can be provided in the form of an irrevocable letter of credit, a pledged savings account, surety bond, or another form of security ("Security") prior to initiating ground-disturbing project activities.

[Option A: BLM-Administered Recovery Funding]

The project owner shall provide this additional 2:1 compensation to the BLM to allow the BLM to further pursue desert tortoise recovery goals not through parcel by parcel acquisitions and management, but rather through implementation of region-wide management plans and land use planning as described in the NEMO, the California Desert Conservation Act plan, and the Desert Tortoise Recovery Plan (USFWS 1994).

Verification: The project owner shall submit a report to the CPM at least 30 days prior to the start of any project-related site disturbance activities confirmation from BLM that the project owner has satisfied this additional 2:1 compensatory mitigation via payment or Security.

[Option B: Commission-Administered Recovery Funding]

The project owner shall provide this additional 2:1 compensation to the CPM in trust as a contribution toward to further pursue desert tortoise recovery goals not through parcel by parcel acquisitions and management, but rather through implementation of region-wide management plans and land use planning as described in the NEMO, the California Desert Conservation Act plan, and the Desert Tortoise Recovery Plan (USFWS 1994).

Verification: The project owner shall submit a report to the CPM at least 30 days prior to the start of any project-related site disturbance activities confirmation from BLM that the project owner has satisfied this additional 2:1 compensatory mitigation via payment or Security.

Proposed Revisions to BIO-18

The Applicant strongly disagrees with BIO-18 and proposes it be replaced with the following.

BIO-18 Prior to start of construction, the project owner shall submit a Special-Status Plant Avoidance and Protection Plan to BLM Authorized Officer for review and comment and to the CPM for review and approval. This plan will identify those areas on the project site that, to the greatest extent practicable considering engineering and construction constraints, can be avoided during project construction and operations. Since there are no federal or state listed threatened or endangered plants on the project site, for the purposes of this Condition, "Special-Status" plants include the following plants designated as "Special Status" by the California Native Plant Society: Mojave milkweed, Rusby's desert-mallow, desert pincushion, nine-awned pappus grass, and Parish's club-cholla. Rusby's desert-mallow is also listed as "Sensitive" by the BLM.

Verification: At least sixty (60) days before the commencement of construction, the project owner shall submit a Draft Special Status Plant Avoidance and Protection Plan to the BLM's Authorized Officer for review and comment and to the CPM for review and approval. The BLM and CPM shall provide comments on the Draft Special Status Plant Avoidance and Protection Plan within fifteen (15) days of receipt of the Draft.

The project owner will offer to meet and confer with the BLM's Authorized Officer and CPM before submission of the Draft Special Status Plant Avoidance and Protection Plan to discuss the Draft. The Draft Special Status Plant Avoidance and Protection Plan shall include the following:

a) Special status plants that are feasible to protect will be designated on project engineering drawings and construction plans as Environmentally Sensitive Plant Areas (ESPAs).

b) Prior to construction, qualified botanists familiar with these Special Status plants will identify localities to be protected on the ground. The limits of the locality will be demarked in the field with temporary flags and/or staking. The approximate "core" of each Special Status plant locality (the area that has the highest density of plants) will marked in the field.

c) Fencing (e.g., such as temporary construction fencing, netting, or mesh) will be used to clearly identify the ESPA to be avoided by construction personnel. The fencing requirements for each ESPA (e.g., dimensions of protective fencing around the plants) will also be determined in the field following re-location of each ESPA to be protected. The exact fence dimensions would need to be field-fitted based on the size of the population and the particular engineering constraints of that area. It is expected that very small localities of ESPA's composed of an individual plant will require a smaller protective fence than a larger population. It will not be possible to fence and protect some of the largest localities that span several feet in diameter; however, it may be possible to fence the "core" area that has the highest density of plants within each locality. Fence size will be constrained by the size of area needed to access and install the heliostat array and other project elements, and the area needed for operations and maintenance. A variety of heavy equipment will be used to complete project construction, and there needs to be sufficient room between the fenced ESPA locality and the power tower, heliostat, and other project elements to maintain safe construction and operation, consistent with OSHA requirements. In addition, the width of the heliostat arrays vary. The distance between heliostat rows is greater farther from the power towers and there is more room to work around fenced ESPA localities in these areas. In parts of the project area where heliostat rows are wider, it may not be as constrained during construction and operation; therefore, the localities farther from the power towers may be more suitable for protection.

d) Fencing will be installed around or above the ESPAs that are to be protected. Fencing or netting installation will be supervised by a qualified botanist familiar with these Special Status plants and will be installed according to locality size data collected during preconstruction surveys and the botanists' field flagging. As-built drawings that show the location of fenced ESPAs will be prepared along with a report describing the number of Special Status plant localities protected, by species.

e) All ESPAs will be avoided during construction. ESPA fencing will be monitored during construction and repaired as necessary.

f) ESPAs will be monitored for three years following the completion of construction of each individual project (i.e., ISEGS 1, 2 and 3) to determine the presence of each Special Status plant locality protected and if each locality is reproducing.

g) Monitoring will be conducted by a qualified botanist familiar with the flora of the Mojave desert and monitoring results will be submitted in accordance with Condition of Certification COMPLIANCE-7 for three years following completion of construction of each Ivanpah project.

BIO-19 Deleted.

It is our understanding based on our professional expertise in preparing environmental reviews that mitigation is not required in the absence of a significant unmitigated impact.

BIO-20 Deleted.

It is our understanding based on our professional expertise in preparing environmental reviews that mitigation is not required in the absence of a significant unmitigated impact.

IV. Correlation to FSA and Hearing Topics:

- Biological Resources

Cultural Resources

I. Introduction

A. **Name:** Clint Helton and W. Geoffrey Spaulding, Ph.D.

B. **Qualifications:** Mr. Helton's and Dr. Spaulding's qualifications are as noted in their resumes contained in Appendix A.

C. **Prior Filings:** In addition to the statements herein, this testimony includes by reference the following documents submitted in this proceeding:

- Application for Certification, Volumes 1 & 2, including confidential appendixes. [Exhibit 1]
- Data Adequacy Supplement A [Exhibit 2]
- Data Adequacy Supplement B [Exhibit 3]
- Comments to the PSA [Exhibit 57]
- Applicant's Response to CEC Staff Requests, Data Response Set 1A, dated January 14, 2008, Responses to Data Requests 33 through 42 [Exhibit 4].
- Applicant's Response to CEC Staff Requests, Data Response Set 1B, dated February 11, 2008, Responses to Data Request 40 [Exhibit 5].
- Applicant's Response to CEC Staff Requests, Data Response Set 1E, dated, July 22, 2008, Responses to Data Request 37 [Exhibit 8].
- Applicant's Response to CEC Staff Requests, Data Response Set 1J, dated December 8, 2008, Responses to Data Request 41 [Exhibit 13].
- Applicant's Response to CEC Staff Requests, Data Response Set 2A, dated June 10, 2008, Responses to Data Request 126 through 129 [Exhibit 20].
- Applicant's Response to CEC Staff Requests, Data Response Set 2B, dated July 22, 2008, Responses to Data Request 126 through 129 [Exhibit 21].
- Applicant's Response to CEC Staff Requests, Data Response Set 2E, dated September 19, 2008, Responses to Data Request 126 through 129 [Exhibit 24].
- Applicant's Response to CEC Staff Requests, Data Response Set 2F, dated October 2, 2008, Responses to Data Request 126 through 129 [Exhibit 25].
- Applicant's Response to CEC Staff Requests, Supplemental Data Response Set 1A, dated August 12, 2008, Responses to CR-1 [Exhibit 32].
- Applicant's Response to CEC Staff Requests, Supplemental Data Response Set 1B, dated August 22, 2008, Responses to CR-2 [Exhibit 33].

To the best of our knowledge, all of the facts contained in this testimony (including all referenced documents) are true and correct. To the extent this testimony contains opinions, such opinions are our own. We make these statements, and render these opinions freely and under oath for the purpose of constituting sworn testimony in this proceeding.

II. Summary of Testimony

A. Affected Environment

The proposed Ivanpah SEGS facility site is on BLM-managed land within the BLM's California Desert District in far eastern San Bernardino County. The border with Nevada lies 4.5 miles to the northwest and the site lies in the northwestern quarter of the Ivanpah Valley. The project site is located in the northwestern quarter of the Ivanpah Valley, which is a largely uninhabited valley in the central Mojave Desert.

The Ivanpah SEGS site and linear facilities were subject to 100 percent (Class III, or complete) archeological resources inventory. The cultural resources study conducted for Ivanpah SEGS included many elements: archival research, a geoarchaeology study, two reconnaissance surveys (one of which included the use of a helicopter to identify potential resources on the surrounding Clark Mountain Range foothills that overlook the project site), two intensive surface pedestrian surveys, and consultation with public sector cultural resource managers, cultural resource management consultants, and archaeological scholars.

A record search conducted at the Central California Information Center of the California Historical Resources Information System (CHRIS) and a search of the Native American Heritage Commission (NAHC) Sacred Lands file was conducted and failed to indicate the presence of Native American cultural resources in the immediate project area.

The NAHC provided a list of Native American contacts that may have knowledge of cultural resources in the area. Letters were sent by CH2M HILL to initiate correspondence with the list of Native American groups and individuals provided by the NAHC. The BLM also initiated government-to-government consultation with those Native American groups that are federally recognized. No responses have been received from any of the letters sent to Native American groups or individuals by either CH2M HILL or the BLM. Local historical societies were also contacted, with no response.

The CHRIS records search found that 21 investigations, 20 pedestrian surveys, and one ethnographic study, had been wholly or partially conducted in the record search area. The search revealed that one site, the Hoover Dam-to-San Bernardino Transmission Line (CA-SBR-10315H), is located in the project area.

As a result of intensive pedestrian survey, CH2M HILL located and documented two new cultural resources in the proposed project area (CA-SBR-12574H and CA-SBR-12575H) and six cultural resources isolates in primary depositional contexts.

CH2M HILL also conducted a geoarchaeology study that concluded that the surface and subsurface prehistoric archaeological potential of the proposed project area is negligible.

CH2M HILL conducted archival research, contacted Native American groups and historic societies, and conducted exhaustive field investigations of the project site, as well as areas outside it. As a result of all these efforts, only one significant historic property, the Hoover

Dam-to-San Bernardino Transmission Line (CA-SBR-10315H), was detected within the project area.

B. Construction Impacts

No NRHP- or CRHR-eligible prehistoric or historical archaeological resources are anticipated to be affected by project construction. Moreover, based on intensive pedestrian inventory and a geoarchaeological study performed by CH2M HILL, the surface and subsurface prehistoric archaeological potential of the proposed project area is considered very low. It is highly improbable that construction-related ground disturbance of the project would directly impact surface or subsurface archaeological resources.

C. Operational Impacts

There should be no operational impacts to cultural or historic resources.

D. Summary of the Cumulative Impacts

One NRHP-eligible and CRHR-listed built-environment resource, the Hoover Dam to San Bernardino transmission line (CA-SBR-10315H) is on the project site. The effects of the proposed project on the subject transmission line have been found to be cumulative in character, rather than the direct result of the construction, operation, maintenance, closure, and decommissioning of the project. With the adoption and implementation of staff's proposed Conditions of Certification CUL-8 and CUL-9, staff can conclude that the cumulative effect of the proposed project on the one presently known NRHP-eligible and CRHR-listed resource, the Hoover Dam-to-San Bernardino transmission line (CA-SBR-10315H), would be rendered less than cumulatively considerable.

E. Mitigation

The Ivanpah SEGS intends to implement measures recommended in the AFC to mitigate potential project impacts to cultural resources:

- Designation of a Cultural Resources Specialist (CRS) to address any unanticipated Discoveries or changes to the project that require additional analysis.
- Construction worker sensitivity training

Though significant archaeological and historical sites were not found during project field surveys conducted by CH2M HILL, and the project area has been shown to possess very low cultural resources sensitivity, it is remotely possible that subsurface construction could encounter buried archaeological remains. A program of construction worker sensitivity training will be implemented and a CRMMP prepared so that if resources are found, the plans will be in place to address the find.

III. Proposed Licensing Conditions

The Cultural Resources section of the FSA/DEIS for the project filed by the CEC and BLM recommends that 10 Conditions of Certification be adopted to address cultural resource issues: Conditions CUL-1through CUL-10. As a general comment, the Applicant is opposed to conditions that require separate approvals of post-certification compliance activities by both BLM and the CPM because they are simply unworkable. If the approval is sequential, it will result in doubling the required approval time for everything. If the approval is concurrent, approvals may be potentially conflicting. As a general rule, consistent with current Commission practice, we have identified the Commission's CPM as the authority to review and approve post-certification compliance submissions or actions of the Applicant. It is also imperative that specific timeframes for approval be included in the Conditions so that the project will not be unnecessarily delayed. See the applicant's pre-hearing conference statement for a more-detailed explanation. Otherwise, these conditions are acceptable.

IV. Correlation to FSA and Hearing Topics:

- Cultural Resources.

Geologic Resources

I. Introduction

A. Name: Tom Lae

B. Qualifications: Mr. Lae's qualifications are as noted in his resume contained in Appendix A.

C. Prior Filings: In addition to the statements herein, this testimony includes by reference the following documents submitted in this proceeding:

- Application for Certification, Volumes 1 & 2. [Exhibit 1]
- Comments to the PSA [Exhibit 57]

To the best of my knowledge, all of the facts contained in this Section of the Applicant's testimony (including all referenced documents) are true and correct. To the extent this testimony contains opinions, such opinions are my own based upon my professional judgment. I make these statements, and render these opinions freely and under oath for the purpose of constituting sworn testimony in this proceeding.

II. Summary of Testimony

A. Affected Environment

The proposed Ivanpah Solar Electric Generating System (Ivanpah SEGS) site is a 4,062-acre parcel within southern California's Mojave Desert, west of Ivanpah Dry Lake in San Bernardino County, California. The Ivanpah Valley is an elongated north-south trending alluvial valley located near the California–Nevada border. The proposed generating facility site is relatively flat (approximate elevation ranges between 3,000 and 2,800 feet). This area is underlain by Quaternary age alluvial sediments and is not within a highly active seismic region.

The most significant geologic hazard at the Ivanpah SEGS site is seismic ground shaking. An earthquake event along the Pahrump-Stateline Fault could produce peak ground gravity (g) acceleration of up to 0.39g in the vicinity of the site, according to the California Department of Transportation (Caltrans) Seismic Hazard Map.

No geologic resources of recreational or scientific value were identified in the vicinity of the project site.

B. Construction Impacts

Construction of the Ivanpah SEGS will require minor grading and excavation; thereby, minimizing alteration of the terrain of the project site. Impacts to the geologic conditions involve dust generation, changes in drainage, cuts, and fills. Since the site is generally level, site grading is not expected to adversely impact the geologic environment. The generating facility and all of the associated linear facilities will be designed and constructed in accordance with the requirements of all applicable federal, state, regional, and local laws, ordinances, regulations, and standards.

C. Operational Impacts

The project will be designed and constructed in accordance with the requirements of all applicable federal, state, regional and local laws, ordinances, regulations, and standards. This will minimize any operational impacts to a level of insignificance.

D. Summary of the Cumulative Impacts

The construction and operation of the Ivanpah SEGS will not produce any significant negative cumulative impacts to geologic resources.

E. Mitigation

Ivanpah SEGS and linear facilities will be constructed in accordance with California Building Code and consistent with the standards adopted by the County of San Bernardino Building Department, minimizing the exposure of people to risks associated with seismic events.

The design and construction of the Ivanpah SEGS and linear facilities will include measures that will limit impacts to less than significant levels. With the implementation of the proposed project mitigation measures and the Condition of Certification, the project will comply with all applicable LORS.

III. Proposed Licensing Conditions

The Final Staff Assessment/Draft Environmental Impact Statement (FSA/DEIS) for the project recommends one geologic Condition of Certification (GEO-1). This condition requires a written Soils Engineering Report that specifically addresses the potential for liquefaction, settlement, dynamic compaction, subsidence, and the presence of expansive clays at the site. Design and construction of the proposed facility in accordance with the requirements of GEO-1 will ensure that the facility will be in compliance with the applicable federal, state, and local laws, ordinances, regulations, and standards (LORS). The Applicant has reviewed the Staff's proposed GEO-1and finds it acceptable.

IV. Correlation to FSA and Hearing Topics:

- Geology, Paleontology and Minerals

Hazardous Materials Handling

I. Introduction

A. Name: Sarah Madams

B. Qualifications: Ms. Madams' qualifications are as noted in her resume contained in Appendix A.

C. Prior Filings: In addition to the statements herein, this testimony includes by reference the following documents submitted in this proceeding:

- Application for Certification, Volumes 1 & 2 [Exhibit 1]
- Comments to the PSA [Exhibit 57]

To the best of my knowledge, all of the facts contained in this Section of the Applicant's testimony (including all referenced documents) are true and correct. To the extent this testimony contains opinions, such opinions are my own based upon my professional judgment. I make these statements, and render these opinions freely and under oath for the purpose of constituting sworn testimony in this proceeding.

II. Summary of Testimony

The project site will be located in southern California's Mojave Desert, about 3.1 miles from the Nevada border, to the west of Ivanpah Dry Lake. The project will be located in San Bernardino County, California, on federal land managed by the Bureau of Land Management (BLM). The proposed project site is located in unincorporated San Bernardino County.

There are no sensitive receptors (such as schools, day-care facilities, convalescent centers, or hospitals) in the vicinity of the project site. The nearest sensitive receptors are the Sandy Valley Elementary and Middle School located approximately 17 miles north of the project site at the corner of Pearl and Hopi Road in Sandy Valley, Nevada, and the Good Springs Elementary School located approximately 18 miles northeast of the project site at 385 W. San Pedro in Goodsprings, Nevada. In California, the nearest sensitive receptor is Baker High School which is located approximately 40 miles southwest of the project at 72100 School House Lane in Baker, California.

Hazardous materials to be used at Ivanpah SEGS during construction and operation were evaluated for hazardous characteristics. Some of these materials will be stored at the project site continuously. Others will be brought onsite for the initial startup and maintenance. Some materials will be used only during startup. Hazardous materials will not be stored or used in the gas supply line, water supply line, or electric transmission line corridors during operation of the plant. Storage locations are described in Table 5.5-2 of the AFC.

A. Construction Impacts

During construction of the project and linears, regulated substances, as defined in California's Health and Safety Code, Section 25531, will not be used.

Hazardous materials to be used during construction of the project and its associated linear facilities will include gasoline, diesel fuel, motor oil, hydraulic fluid, solvents, cleaners, sealants, welding flux, various lubricants, paint, and paint thinner. There are no feasible alternatives to motor fuels and oils for operating construction equipment. The types of paint required are dictated by the types of equipment and structures that must be coated and by the manufacturers' requirements for coating.

The quantities of hazardous materials that will be onsite during construction are small and similar to the quantities used during operation. Construction personnel will be trained to handle the materials properly. The most likely possible incidents will involve the potential for fuels, oil, and grease dripping from construction equipment. The small quantities of fuel, oil, and grease that might drip from construction equipment will have relatively low toxicity and will be biodegradable. Therefore, the expected environmental impact is minimal.

Small fuel spills may also occur during onsite refueling. The potential environmental effects from fueling operations are expected to be limited to small areas of contaminated soil. If a fuel spill occurs on soil, the contaminated soil will be placed into barrels or trucks for offsite disposal as a hazardous waste.

The quantities of hazardous materials that will be handled during construction are relatively small. Personnel working on the project during the construction phase will be trained in handling of and the dangers associated with hazardous materials. Therefore, the potential for environmental effects is expected to be small.

B. Operational Impacts

During the Ivanpah SEGS operation, one regulated substance will be stored onsite. Sulfuric acid, an extremely hazardous substance, is a very corrosive chemical that can cause severe harm to humans if ingested, inhaled, or contacted. However, sulfuric acid has a very low vapor pressure and will not readily volatilize upon release. Therefore, the potential for harm to humans offsite is minimal. Sulfuric acid is identified as a regulated substance under the CalARP program, but only if it is concentrated with greater than 100 pounds of sulfur trioxide, if it meets the definition of oleum, or if it is stored in a container with flammable hydrocarbons. The sulfuric acid that will be used at the Ivanpah SEGS facility does not contain more than 100 pounds of sulfur trioxide or meet the definition of oleum. In addition, it will not be stored in a container with flammable hydrocarbons. Therefore, sulfuric acid is not subject to the RMP requirements under CalARP.

If a spill involves hazardous materials equal to or greater than the specific reportable quantity all federal, state, and local reporting requirements will be followed.

C. Cumulative Effects

The only past, present, or reasonably foreseeable future projects in the vicinity are the Primm Valley Golf Club located about 0.5 mile east of the project site and Interstate15 (I-15) located about 0.8 mile southeast. Ivanpah SEGS would not store any hazardous material that could migrate offsite. Therefore, hazardous materials at the project site would not combine with hazardous materials at the Golf Club or from vehicles traveling I-15 to create a cumulative impact.

D. Mitigation

As outlined in the AFC, potential impacts during construction and operational phases will be mitigated through extensive implementation of engineered controls, training, best management practices, and the development of plans and procedures. With the implementation of the proposed project mitigation measures and the Conditions of Certification, the project will comply with all applicable federal, state, and local laws, ordinances, regulations, and standards (LORS).

All hazardous materials stored onsite during Ivanpah SEGS operation will be handled and stored in accordance with applicable codes and regulations. All containers used to store hazardous materials will be inspected regularly for signs of leaking or failure. Incompatible materials will be stored in separate storage and containment areas. Areas susceptible to potential leaks and/or spills will be paved and bermed. Containment areas may drain to a collection area, such as an oil/water separator or a waste collection tank. Piping and tanks will be protected from potential traffic hazards by concrete or pipe-type traffic bollards and barriers.

Hazardous materials will be delivered periodically to Ivanpah SEGS. Transportation will comply with the applicable regulations for transporting hazardous materials, including the US Department of Transportation, USEPA, California Department of Toxic Substances Control, California Highway Patrol, and California State Fire Marshal.

A worker safety plan, in compliance with applicable regulations, will be prepared and implemented. It will include training for contractors and operations personnel. Training programs will include safe operating procedures, the operation and maintenance of hazardous materials systems, proper use of personal protective equipment, fire safety, and emergency communication and response procedures. All plant personnel will be trained in emergency procedures, including plant evacuation and fire prevention. In addition, designated personnel will be trained as members of a plant hazardous material response team; team members will receive the first responder and hazardous material technical training to be developed in the HMBP (Section 5.5.6.4 of the AFC). For emergency spills, San Bernardino County Fire Department has a formally trained Hazardous Materials Response Team to provide assistance during a spill cleanup.

Although the Ivanpah SEGS will store one regulated substance, sulfuric acid, the type that will be used at the Ivanpah SEGS facility does not contain more than 100 pounds of sulfur trioxide or meet the definition of oleum. In addition, it will not be stored in a container with flammable hydrocarbons. Therefore, sulfuric acid is not subject to the RMP requirements under CalARP, and an RMP will not be prepared for the project. An extensive monitoring program will not be required because environmental effects during the construction and operation phases of the facility are expected to be minimal. However, sufficient monitoring will be performed during the construction and operation phases to ensure that the proposed mitigation measures are implemented and that they are effective in mitigating any potential environmental effects.

III. Proposed Licensing Conditions

The FSA/DEIS for the project filed by the CEC and BLM recommends that 6 Conditions of Certification be adopted to address hazardous materials management issues: HAZ-1 through HAZ-6. The Applicant respectfully requests that only the CPM be required to approve.

Proposed Revisions to HAZ-1 through 6

The Applicant is opposed to conditions that require separate approvals of post-certification compliance activities by both BLM and the CPM because they are simply unworkable. If the approval is sequential, it will result in doubling the required approval time for everything. If the approval is concurrent, approvals may be potentially conflicting. As a general rule, consistent with current Commission practice, we have identified the Commission's CPM as the authority to review and approve post-certification compliance submissions or actions of the Applicant. It is also imperative that specific timeframes for approval be included in the Conditions so that the project will not be unnecessarily delayed. See the applicant's pre-hearing conference statement for a more-detailed explanation.

Proposed Revisions to HAZ-4 and 5

The project owner needs flexibility in determining what is appropriate and may not want all of these measures. We agree that we have an obligation to "discuss" the relative value of each but not be obligated to implement ALL of them. Regarding HAZ-5, since the power block will be within the security fencing of the heliostat field, there is no need for it to be fenced separately.

HAZ-4 At least thirty (30) days prior to commencing construction, a site-specific Construction Site Security Plan for the construction phase shall be prepared and made available to ~~BLM's Authorized Officer and~~ the CPM for review and approval. The Construction Security Plan shall ~~include the following~~discuss the following measures, indicate which ones the project owner plans to implement, and describe how these measures will be implemented:

1. Perimeter security consisting of fencing enclosing the construction area;

2. Security guards;

3. Site access control consisting of a check-in procedure or tag system for construction personnel and visitors;

4. Written standard procedures for employees, contractors and vendors when encountering suspicious objects or packages on-site or off-site;

5. Protocol for contacting law enforcement~~,~~ and the CPM ~~BLM's Authorized Officer and the CPM~~ in the event of ~~suspicious activity or emergency~~ emergency or conduct endangering the facility, its employees, or contractors; and

6. Evacuation procedures.

Verification: At least thirty (30) days prior to commencing construction, the project owner shall notify ~~BLM's Authorized Officer and~~ the CPM that a site-specific Construction Security Plan is available for review and approval.

HAZ-5 The project owner shall prepare a site-specific Operation Security Plan for the operational phase, which ~~and~~ shall be made available to ~~BLM's Authorized Officer and~~ the CPM for review and approval. The project owner shall implement site security measures addressing physical site security and hazardous materials storage. The level of security to be implemented shall not be less than that described below (as per NERC 2002).

The Operations Security Plan shall ~~include the following~~discuss the following measures, indicate which ones the project owner plans to implement, and describe how these measures will be implemented:

1. Permanent full perimeter fence or wall, at least eight feet high around the ~~Power Block and~~ Solar Field;

2. Main entrance security gate, either hand operable or motorized;

3. Evacuation procedures;

4. Protocol for contacting law enforcement~~, BLM's Authorized Officer and the CPM~~ and the CPM in the event of ~~suspicious activity or~~ emergency or conduct endangering the facility, its employees, or contractors; and;

5. Written standard procedures for employees, contractors and vendors when encountering suspicious objects or packages on-site or off-site;

6. a. A statement (refer to sample, attachment "A") signed by the project owner certifying that background investigations have been conducted on all project personnel. Background investigations shall be restricted to ascertain the accuracy of employee identity and employment history, and shall be conducted in accordance with state and federal law regarding security and privacy;

   b. A statement(s) (refer to sample, attachment "B") signed by the contractor or authorized representative(s) for any permanent contractors or other technical contractors (as determined ~~by BLM's Authorized Officer and~~ the CPM after consultation with the project owner) that are present at any time on the site to repair, maintain, investigate, or conduct any other technical duties involving critical components (as determined by ~~BLM's Authorized Officer and~~ the CPM after consultation with the project owner) certifying that background investigations have been conducted on contractor personnel that visit the project site. Background investigations shall be restricted to ascertain the accuracy of employee identity and employment history, and shall be conducted in accordance with state and federal law regarding security and privacy

7. Site access controls for employees, contractors, vendors, and visitors;

8. Closed Circuit TV (CCTV) monitoring system, recordable, and viewable in the power plant control room and security station (if separate from the control room) capable of viewing, at a minimum, the main entrance gate; and

9. Additional measures to ensure adequate perimeter security consisting of either:

   a. Security guard present 24 hours per day, seven days per week, **OR**

   b. Power plant personnel on-site 24 hours per day, seven days per week and **all** of the following:

      1) The CCTV monitoring system required in number 8 above shall include cameras that are able to pan, tilt, and zoom (PTZ), have low-light capability, are recordable, and are able to view 100% of the perimeter fence, the outside entrance to the control room, and the front gate from a monitor in the power plant control room; **AND**

      2) Perimeter breach detectors **or** on-site motion detectors.

The project owner shall fully implement the security plans and obtain ~~BLM's Authorized Officer and~~ CPM approval of any substantive modifications to the security plans. BLM's Authorized Officer and the CPM may authorize modifications to these measures, or may require additional measures, such as protective barriers for critical power plant components (e.g., transformers, gas lines, compressors, etc.) become necessary ~~depending on circumstances unique to the facility or in~~ as a result of ~~response to~~ changes to ~~industry-related standards,~~ security concerns, or guidance provided by the ~~additional guidance provided by the~~ U.S. Department of Homeland Security, the U.S. Department of Energy, or the North American Electrical Reliability Council, the project owner will ~~after~~ consult~~ation~~ with appropriate law enforcement agencies to discuss appropriate modifications to the Operations Security Plan. Such modifications shall be submitted to the CPM, who will review and approve such modifications as described above. ~~and the applicant.~~

Verification: At least 30 days prior to ~~the initial receipt of hazardous materials on-site~~commercial operations, the project owner shall notify ~~BLM's Authorized Officer and~~ the CPM that a site-specific Operations Site Security Plan is available for review and approval. The CPM shall review and approve the Operations Site Security Plan within thirty (30) days of submission. In the Annual Compliance Report, the project owner shall include a statement that all current project employee and appropriate contractor background investigations have been performed, and updated certification statements are appended to the Operations Security Plan. In the Annual Compliance Report, the project owner shall include a statement that the Operations Security Plan includes all current hazardous materials transport vendor certifications for security plans and employee background investigations.

HAZ-6 The ~~holder (~~project owner~~)~~ shall ~~comply with all applicable Federal laws and regulations existing or hereafter enacted or promulgated~~ provide to the BLM's Authorized Officer and the CPM a copy of any report required or requested by any Federal or State governmental entity as a result of a reportable release or spill of any toxic substances.~~. In any event, the holder(s) shall comply with the Toxic Substances Control Act of 1976, as amended (15 U.S.C. 2601, et seq.) with regard to any toxic substances that are used, generated by or stored on the right-of-way or on facilities authorized under this right-of-way grant. (See 40 CFR, Part 702-799 and especially, provisions on polychlorinated biphenyls, 40 CFR 761.1-761.193.) Additionally, any release of toxic substances (leaks, spills, etc.) in excess of the reportable quantity established by 40 CFR, Part 117 shall be reported as required by the Comprehensive Environmental Response, Compensation and Liability Act of 1980, Section 102b~~

Verification: A copy of any report required or requested by any Federal ~~agency~~ or State governmental entity as a result of a reportable release or spill of any toxic substances shall be furnished to BLM's Authorized Officer and the CPM concurrent with the filing of the report~~s~~ ~~to~~ with the ~~involved~~ Federal ~~agency~~ or State governmental entity.

IV. Correlation to FSA and Hearing Topics:

- Hazardous Materials

Land Use

(Including Livestock Grazing, Wild Horses and Burrows, and Recreation)

I. Introduction

A. **Name:** Jennifer Scholl

B. **Qualifications:** Ms. Scholl's qualifications are as noted in her resume contained in Appendix A.

C. **Prior Filings:** In addition to the statements herein, this testimony includes by reference the following documents submitted in this proceeding:

- Application for Certification, Volume 1 & 2 [Exhibit 1]
- Data Adequacy Supplement A [Exhibit 2]
- Comments to the PSA [Exhibit 57]
- Applicant's Response to CEC Staff Requests, Data Response Set 1A, dated January 14, 2008, Responses to Data Requests 43 through 52 [Exhibit 4].
- Applicant's Response to CEC Staff Requests, Data Response Set 1B, dated February 11, 2008, Responses to Data Requests 44 through 49 [Exhibit 5].
- Applicant's Response to CEC Staff Requests, Data Response Set 1D, dated May 9, 2008, Responses to Data Requests 44 through 49 [Exhibit 7].

To the best of my knowledge, all of the facts contained in this Section of the Applicant's testimony (including all referenced documents) are true and correct. To the extent this testimony contains opinions, such opinions are my own based upon my professional judgment. I make these statements, and render these opinions freely and under oath for the purpose of constituting sworn testimony in this proceeding.

II. Summary of Testimony

The Ivanpah SEGS power plant site and associated linear project features are located within unincorporated San Bernardino County on federal land administered by the U.S. Department of the Interior, BLM. Therefore, the applicant has applied for a BLM Right-of-Way (ROW) grant and a Certification from the CEC. For the purposes of environmental review and permitting, the BLM is the lead federal agency for compliance with the National Environmental Policy Act (NEPA) and the CEC is the lead state agency for compliance with the California Environmental Quality Act (CEQA). The project was processed in accordance with e Memorandum of Understanding (MOU) Between the BLM, California Desert District, and the CEC Staff fully executed on August 8, 2007 which outlines the process for coordinating the review and analysis of solar energy projects subject to both BLM and CEC jurisdiction.

A. Affected Environment

The Ivanpah SEGS site is located within areas in the California Desert Conservation Area Plan (CDCA) that are designated Multiple-Use Class L (Limited Use) and Multiple-Use Class M (Moderate Use). The Energy Production and Utility Corridors Element of the CDCA Plan also states that the BLM focuses on the same factors affecting public lands and their resources as those used by the CEC. These factors include: (1) consistency with the CDCA Plan, including the designation of proposed planning corridors; (2) protection of air quality; (3) impact on adjacent wilderness and sensitive resources; (4) visual quality; (5) fuel sources and delivery systems; (6) cooling-water source(s); (7) waste disposal; (8) seismic hazards; and (9) regional equity. The Proposed Northern and Eastern Mojave (NEMO) Desert Management Plan (July 2002) amends the BLM CDCA Plan as discussed below.

County of San Bernardino

Pursuant to an MOU between BLM and the County of San Bernardino, the County's General Plan, would be the county planning document that would be applicable to the Ivanpah SEGS and the associated linear features but for the Federal lands designation and the CEC's the exclusive permitting authority. Implementation of the General Plan would occur through classification and regulation of land uses and structures in the County Development Code. The Ivanpah SEGS and linear components (transmission, natural gas, and sewer lines) are all located in an area designated in the CDCA Plan as Multiple-Use Class L (Limited Use) and Multiple-Use Class M (Moderate Use) and in the San Bernardino County General Plan and Development Code as RC (Resource Conservation).

General Description of Study Area

Land uses in the vicinity of the project area are largely BLM-managed open space. Existing utility corridors are located throughout the BLM property and between Ivanpah 1 and 2. The nearest recreational land use is the Primm Valley Golf Club, Desert Course, located approximately 0.5 miles east of the Ivanpah 1 site boundary. There are no schools, day-care facilities, convalescent centers, or hospitals within, or in the immediate vicinity of, the project study area. There are no current agricultural uses within the proposed Ivanpah SEGS site or lands mapped as Important Farmlands within the project study area; however, the project study area is part of an existing BLM Grazing Lease. The NEMO Plan area is also a popular area that provides diverse recreational and scenic opportunities for off-highway vehicle use. Ivanpah Dry Lake is located approximately 1.6 miles east of the project site and is open to non-motorized vehicles and is a popular destination for activities such as kite buggying, land sailing, long-distance archery, and kite demonstrations.

B. Environmental Analysis

The Ivanpah SEGS project was evaluated against CEQA Guidelines Appendix G, CEQA Checklist to evaluate the potential land use impacts associated with implementation of the project. For each of the appropriate checklist criteria, it was determined that implementation of the Ivanpah SEGS project would not result in any land use impacts. Specifically it was determined that the Ivanpah SEGS does not:

1. Physically divide an established community because the power plant project site and linear features would be located on generally undeveloped federal property in unincorporated San Bernardino County and are not located within an established community.

2. Conflict with applicable land use plans, policies, or regulations adopted for the purpose of avoiding or mitigating an environmental effect. All of the project components are located on land that is designated Multiple-Use Class L and Multiple-Use Class M by the BLM which allows for solar electrical generation facilities and linear project components, pursuant to an amendment to the CDCA Plan to account for implementation of the project. ISEGS is located on land designated RC by the County of San Bernardino which also allows for electrical power generation facilities. A solar energy generation system (electrical power generation) would be an allowed use for RC designated land with a conditional use permit (if the County had permit jurisdiction, however, the CEC licensing process supersedes this permit requirement).

3. Lie within critical habitat for the desert tortoise as identified in the Desert Tortoise Recovery Plan and therefore, does not conflict with any applicable HCP or natural community conservation plans.

4. Fall within the category of lands designated for prime farmland, unique farmland, or farmland of statewide importance.

5. Have any agriculture uses or Williamson Act properties present within the project study area. However, it will be necessary for the project area that is contained within the existing BLM Clark Mountain Allotment Grazing Lease to be removed from the grazing lease prior to the start of project construction. These grazing lands are not considered to qualify as important farmlands.

C. Cumulative Impacts

Ivanpah SEGS is consistent with the applicable plans and policies and, therefore, would not result in significant land use, recreation, or agricultural impacts. No farmland is present in the study area and existing agricultural uses are minimal, so the project would not directly or cumulatively affect farmland. The project site does not lie within critical habitat for the desert tortoise as identified in the Desert Tortoise Recovery Plan, and therefore, would not result in a cumulative conflict with this Plan. Further it is expected that the reasonably foreseeable projects considered in the cumulative analysis would also not contribute to a significant impacts on land use, recreation, or agricultural impacts because each of these projects will receive development approvals that could not be issued without a determination that these projects are consistent with applicable plans and policies, including development, farmland, and habitat conservation policies.

III. Proposed Licensing Conditions

The FSA/DEIS for the project filed by the CEC and BLM recommends that two Conditions of Certification be adopted to address land use issues: LAND-1 through LAND-2. In addition, the FSA/DEIS recommends one condition (REC-1) for Recreation. While the Applicant does not dispute the need to complete the BLM ROW process, this action is considered an entitlement and not a mitigation measure. Regarding, Recreation and REC-1, the project is not "located in the coastal zone or any other area with recreational, scenic, or historic value." Further, Section 25529 focuses on access "an area be established for public use." Since the focus is on public access, Section 25529 does not require construction of any facility, in general, and certainly does not require construction of a multimillion-dollar facility. Further, Section 25529 contemplates acquisition of lands ("Lands shall be acquired…") and dedication of such to the State or a non-profit. The project lands are federal lands. As such, they cannot be "acquired" and dedicated to the state agency or any non-profit. REC-1 should be deleted.

Proposed Revisions to LAND-1

LAND-1, as proposed in the FSA/DEIS restates a requirement of BLM for obtaining a ROW Grant that is necessary for the project. Since this is exclusively a federal permitting process, the Applicant respectfully suggests that the project owner's obligation should be to keep the CPM informed of the status of these federal activities. It is also important to avoid confusion regarding the BLM's exclusive authority in these federal issues. Accordingly, the project owner suggests the following changes to LAND-1:

LAND-1 The project owner shall obtain a Right-of-Way Grant (ROW Grant) and final approved Plan of Development from the Bureau of Land Management (BLM) prior to the start of construction of the project. ~~Among the conditions for obtaining the ROW grant, the applicant shall provide the following:~~

~~A. Prior to issuance of any right of way grant, the project owner shall submit a final Plan(s) of Development that describes in detail the construction, operation, maintenance, and termination of the right-of-way and its associated improvements and/or facilities. The project owner shall construct, operate, and maintain the facilities, improvements, and structures within this right-of-way in strict conformity with the final approved Plan of Development. The degree and scope of these plans will vary depending upon (1) the complexity of the right-of-way or its associated improvements and/or facilities, (2) the anticipated conflicts that require mitigation, and (3) additional technical information required by BLM's Authorized Officer and the CPM. The plans will be reviewed, and if appropriate, modified by the project owner until acceptable, and approved by BLM's Authorized Officer and the CPM. An approved Plan of Development shall be made a part of the right-of-way grant. Any relocation, additional construction, or use that is not in accord with the approved Plan(s) of Development, shall not be initiated without the prior written approval of BLM's Authorized Officer and the CPM.~~

~~B.~~ A bond, acceptable to BLM's Authorized Officer, shall be furnished by the project owner prior to the issuance of a Notice to Proceed with construction or at such earlier date as may be specified by BLM's Authorized Officer. The amount of this bond shall be determined by BLM's Authorized Officer. ~~This bond must be maintained in effect until removal of improvements and restoration of the right-of-way have been accepted by BLM's Authorized Officer and the CPM.~~

Verification: At least 30 days prior to the start of construction and prior to any Notice to Proceed with construction issued by BLM's Authorized Officer ~~and the CPM~~, the project owner shall provide ~~BLM's Authorized Officer and~~ the CPM with documentation of the following:

A. BLM's ROW Grant and final approved Plan of Development;

B. The bond satisfactory to BLM's Authorized Officer.~~;~~

~~C. Certification that the project owner acknowledges that the ISEGS development and all related construction, operation, maintenance and closure activities are to be conducted in conformance with the approved Plan of Development and within the approved ROW boundaries for the life of the project.~~

Proposed Revisions to LAND-2

The requirement for a 20-foot setback is more than what is required in order to maintain the exterior of the perimeter fence. Requiring the additional setback unnecessarily reduces the size of the heliostat field. If temporary use of an area outside the ROW boundary is needed, existing statutes require application for a temporary use permit; hence, the requirement does not need to be restated. The 8 to 12 feet in the revised language represents the space needed for utility vehicle access to repair the fence or remove any accumulated debris.

LAND-2 The ~~applicant's Project Description and associated construction plans shall be revised to~~project owner shall allow a minimum 8- to 12~~20~~-foot ~~buffer~~ setback between (1) the security and tortoise exclusion fence, and (2) the proposed ROW boundary. Once the fencing is constructed, all inspection, monitoring, and maintenance activities required outside of the fencing will occur on lands included within this ~~buffer~~ setback area ~~and ROW boundaries. Should project activities requiring the use of an area larger than the buffer be required (such as installation of new drainage structures one acre or more in size), the project owner shall make application to BLM for a Temporary Use Permit (TUP) or additional ROW Grant, and to the Energy Commission for a license amendment prior to conducting any activities. Authorization of a TUP or additional ROW Grant may require additional environmental evaluation pursuant to the National Environmental Policy Act and the California Environmental Quality Act.~~

Verification: At least thirty (30)~~60~~ days prior to the start of construction, the project owner shall provide BLM's Authorized Officer and the CPM with a revised project description and construction plans specifying the inclusion of the ~~buffer zone~~setback area within the ROW boundaries. The project owner shall also provide BLM's Authorized Officer and the CPM with certification acknowledging that the ISEGS development and all related construction, operation, maintenance and closure activities are to be conducted within the ROW boundaries for the life of the project.

IV. Correlation to FSA and Hearing Topics:

- Land Use, Livestock Grazing, Wild Horses and Burrows, and Recreation

Noise and Vibration

I. Introduction

A. Name: Mark Bastasch and Todd Stewart

B. Qualifications: Mr. Bastasch's and Mr. Stewart's qualifications are as noted in their resumes contained in Appendix A.

C. Prior Filings: In addition to the statements herein, this testimony includes by reference the following documents submitted in this proceeding:

- Application for Certification, Volume 1[Exhibit 1]
- Comments to the PSA [Exhibit 57]

To the best of our knowledge, all of the facts contained in this testimony (including all referenced documents) are true and correct. To the extent this testimony contains opinions, such opinions are our own. We make these statements, and render these opinions freely and under oath for the purpose of constituting sworn testimony in this proceeding.

II. Summary of Testimony

A. Affected Environment

The proposed Ivanpah Solar Electric Generating System site is a 4,062-acre parcel within southern California's Mojave Desert, west of Ivanpah Dry Lake in San Bernardino County, California. The closest community is Primm, Nevada, with a population of 436, located approximately 4.5 miles northeast of the project area. The Primm Valley Golf Club is located east of Ivanpah 2. The Ivanpah Dry Lake is located east of the project site and is bisected by I-15. The project vicinity consists of BLM-managed open space. The nearest human occupancy is the Primm Valley Golf Club. Ambient noise measurements were not surveyed as part of this analysis given the community of Primm, Nevada, 4.5 miles distant, is too far from Ivanpah SEGS to be significantly impacted by project noise. The Primm Valley Golf Club is considered a less noise-sensitive land use.

B. Construction Impacts

Construction of Ivanpah SEGS is expected to be similar to other power plants in terms of schedule, equipment used, and other types of activities. The noise level will vary during the construction period, depending upon the construction phase. Construction noise is not anticipated to be noticeable in Primm, with the potential exception of pile driving, which (if required) is not anticipated to exceed current noise exposure levels.

C. Operational Impacts

Given the solar nature of this project, activity at night will be limited and primarily maintenance-related and would not represent significant noise sources. The power plant will operate an average of about 10 hours a day, 7 days a week throughout the year, with the exception of a scheduled shutdown in late December for maintenance. The solar field and power generation equipment will be started up each morning after sunrise and insolation build-up, and shut down in the evening when insolation drops below the level required to keep the steam turbine on line. Nighttime activities include mirror washing, water pumping and water treatment. Operational noise from the Ivanpah SEGS is predicted not to exceed 30 dBA in Primm, Nevada and to be less than the County's residential daytime standard of 55 dBA at the golf club.

D. Summary of the Cumulative Impacts

Given the mitigation for the project and the lack of past, present or reasonable foreseeable projects in the vicinity, no significant noise or vibration cumulative impacts would occur.

E. Mitigation

The Ivanpah SEGS and linear facilities will be constructed in accordance with the proposed mitigation measures and Conditions of Certification. With the implementation of the proposed project mitigation measures and Conditions of Certification, operation and construction of the project will comply with all applicable LORS and would produce no CEQA or NEPA significant adverse noise impacts on people within the affected area, directly, indirectly, or cumulatively.

III. Proposed Licensing Conditions

The FSA/DEIS for the project filed by the CEC and BLM recommends that 7 Conditions of Certification be adopted to address noise and vibration issues, NOISE-1 through NOISE-7. With the exception of minor changes to NOISE-6 and NOISE-7 described below, the CEC's proposed Conditions of Certification are acceptable. Given the remote nature of the project site and lack of sensitive receptors, the following modifications are proposed to provide the Applicant flexibility.

As a general comment, that applies to the noise conditions, the Applicant is opposed to conditions that require separate approvals of post-certification compliance activities by both BLM and the CPM because they are simply unworkable. If the approval is sequential, it will result in doubling the required approval time for everything. If the approval is concurrent, approvals may be potentially conflicting. As a general rule, consistent with current Commission practice, we have identified the Commission's CPM as the authority to review and approve post-certification compliance submissions or actions of the Applicant. It is also imperative that specific timeframes for approval be included in the Conditions so that the project will not be unnecessarily delayed. See the applicant's pre-hearing conference statement for a more-detailed explanation.

Proposed Revisions to NOISE-4

The text of NOISE-4 has been revised to add clarity.

NOISE-4 The project design and implementation shall include appropriate noise mitigation measures adequate to ensure that operation of the project will not cause noise complaints from residents of Primm, Nevada, or from the operator of the Primm Valley Golf Course. If legitimate project-related noise complaints are received from residents of Primm, the project owner shall perform a noise survey to demonstrate that noise levels due to plant operation do not exceed an average of 45 dBA L_{eq} measured at the nearest residence of the community of Primm, Nevada. If legitimate project-related noise complaints are received from the operator of the Primm Valley Golf Course, the project owner shall perform a noise survey to demonstrate that noise levels due to plant operation do not exceed an average of 55 dBA L_{eq} measured at the nearest boundary of the golf course. No new project ~~ure-tone~~ components creating pure-tone noises will be added to~~may be caused by~~ the project unless they are balanced by other plant features. No single piece of equipment shall be allowed to stand out as a source of noise that draws legitimate complaints.

A. The measurement of power plant noise for the purposes of demonstrating compliance with this condition of certification may alternatively be made at a location, acceptable to ~~BLM's Authorized Officer and~~ the CPM, closer to the plant (e.g., 400 feet from the plant boundary) and this measured level then mathematically extrapolated to determine the plant noise contribution at the affected location. The character of the plant noise shall be evaluated at the affected residential locations to determine the presence of pure tones or other dominant sources of plant noise.

Verification: The survey shall take place within 30 days of the receipt of the noise complaint, unless the complaint has been resolved to the complaining party's satisfaction. Within 15 days after completing the survey, the project owner shall submit a summary report of the survey to ~~BLM's Authorized Officer and~~ the CPM. Included in the survey report will be a description of ~~any~~ additional mitigation measures (if any) necessary to achieve compliance with the above-listed noise limit and a schedule, subject to ~~BLM's Authorized Officer and~~ CPM approval, for implementing these measures. When these measures are in place, the project owner shall repeat the noise survey.

Within 15 days of completion of the new survey, the project owner shall submit to ~~BLM's Authorized Officer and~~ the CPM a summary report of the new noise survey, performed as described above and showing compliance with this condition.

Proposed Revisions to NOISE-6

In the Preliminary Staff Analysis, the staff made the following statement:

> Construction of an industrial facility such as a power plant is typically noisier than permissible under usual noise ordinances. In order to allow the construction of new facilities, construction noise during certain hours of the day is commonly exempt from enforcement by local ordinances. The San Bernardino County Development Code exempts all construction noise from numerical noise limits between 7:00 a.m. and 7:00 p.m. Monday through Saturday (see NOISE Table 2, above). Even though this LORS does not apply to construction on federal land, the applicant commits to complying with this restriction (BSE 2007a, AFC § 5.7.7.3). (PSA, page 5.6-6)

Applicant agrees with Staff's analysis that the San Bernardino County LORS do not apply. However, the Staff has misquoted the Applicant's commitment in the AFC where the Applicant stated:

> Noisy construction work (that causes offsite annoyance as evidenced by the filing of a legitimate noise complaint) shall be restricted to the 7:00 a.m. to 7:00 p.m. time period. Haul trucks shall be operated in accordance with posted speed limits. Truck engine exhaust brake use shall be limited to emergencies. (AFC, page 5.7-13)

Hence, the Applicant proposes the following changes to NOISE-6 so that it will conform to the commitment made in the AFC.

NOISE-6 Heavy equipment operation and noisy construction work that causes offsite annoyance as evidenced by the filing of a legitimate noise complaint) shall be restricted to the 7:00 a.m. to 7:00 p.m. time period.~~relating to any project features shall be restricted to the times of day delineated below:~~

~~Weekdays and Saturdays 7:00 a.m. to 7:00 p.m. No noisy construction work shall be performed on Sundays or federal holidays. Haul trucks and other engine-powered equipment shall be equipped with mufflers that meet all applicable regulations~~Haul trucks shall be operated in accordance with posted speed limits. Truck engine exhaust brake use shall be limited to emergencies.

Verification: Prior to ground disturbance, the project owner shall transmit to ~~BLM's Authorized Officer and~~ the CPM a statement acknowledging that the above restrictions will be observed throughout the construction of the project.

Proposed Revisions to NOISE-7

Additional text is proposed to provide the project owner flexibility.

NOISE-7 If a high-pressure steam blow is employed, the project owner shall equip steam blow piping with a temporary silencer or take other effective measures that quiet the noise of steam blows to no greater than 60 dBA measured at the Primm Valley Golf Club and no greater than 55 dBA measured at any affected residential locations in Primm, NV. The project owner shall conduct high-pressure steam blows only during the hours of 7:00 a.m. to 7:00 p.m.

If a low-pressure continuous steam blow is employed, the project owner shall limit the noise of steam blows to no greater than 45 dBA measured at any affected residential location in Primm, NV. In lieu of specifying the level of silencing above, the project owner may alternatively submit an analysis to the CPM that documents that during either high or low pressure steam blows, steam blow noise levels would not exceed 60 dBA at the golf club (daytime), or 55 dBA (daytime)/45 dBA (nighttime) at the nearest residential location in Primm.

Verification: At least fifteen (15) days prior to the first high pressure steam blow, the project owner shall submit to ~~BLM's Authorized Officer and~~ the CPM drawings or other information describing the temporary steam blow silencer or other noise attenuating measures to be taken, the noise levels expected and a description of the steam blow schedule.

At least fifteen (15) days prior to any low-pressure continuous steam blow, the project owner shall submit to ~~BLM's Authorized Officer and~~ the CPM drawings or other information describing the process, including the noise levels expected and the projected time schedule for execution of the process.

IV. Correlation to FSA and Hearing Topics:

- Noise and Vibration

Paleontological Resources

I. Introduction

A. Name: W. Geoffrey Spaulding, Ph.D.

B. Qualifications: Dr. Spaulding's qualifications are as noted in his resume contained in Appendix A.

C. Prior Filings: In addition to the statements herein, this testimony includes by reference the following documents submitted in this proceeding:

- Application for Certification, Volumes 1 & 2 [Exhibit 1]
- Comments to the PSA [Exhibit 57]

To the best of my knowledge, all of the facts contained in this Section of the Applicant's testimony (including all referenced documents) are true and correct. To the extent this testimony contains opinions, such opinions are my own based upon my professional judgment. I make these statements, and render these opinions freely and under oath for the purpose of constituting sworn testimony in this proceeding.

II. Summary of Testimony

The Paleontological analysis establishes that no direct impacts to paleontological resources would occur from construction or operation of the proposed Ivanpah SEGS or associated gas pipeline. Impacts to paleontological resources would not occur from construction-related excavations or other activities that would disturb low sensitivity Quaternary alluvium, which underlies the project site. Quaternary alluvium does not yield fossils other than the occasional Paleozoic invertebrate in a clast of carbonate rock from the surrounding mountains. Such remains are out of stratigraphic context and of minimal scientific significance.

The only area of moderate to high paleontological sensitivity in the project vicinity is the Ridge 1059 area immediately to the west. Because the configuration of the Ivanpah SEGS project area avoids the ridge, no direct impacts will occur to the potentially fossiliferous limestone of ridge itself, or to the Quaternary-age packrat middens in rock shelters within the ridge. Indirect impacts are unlikely from this project, since the nature of the paleontological resources on the ridge would not attract the attention of the casual collector or vandal. Macrofossils are rare or absent in the limestone itself, and the ancient packrat middens are non-descript, obscurely placed, and of a composition that would not attract the casual collector or vandal.

III. Proposed Licensing Conditions

The FSA/EIS for the project filed by the BLM and the CEC recommends that 7 Conditions of Certification be adopted to address paleontological issues: PAL-1 through PAL-7. These would be acceptable should paleontologically sensitive sediment be encountered in the project area, or paleontological resources found during excavations. However, as noted above, and in the AFC, there is no evidence for paleontological sensitivity in the project area. However, the Applicant appreciates Staff concerns and agrees to retain a project paleontologist, and provide paleontological resources awareness training to construction personnel, as well as to the implementation of other measures should paleontologically sensitive sediment or resources be encountered. No paleontologically sensitive sediments are currently known in the project area and, therefore, no paleontological resources monitoring is anticipated. Hence, a PRMMP will be developed for review and approval. Consequently, the Paleontological conditions are acceptable.

IV. Correlation to FSA and Hearing Topics:

- Geology and Paleontology

Public Health

I. Introduction

A. Names: Steve Hill and Gary Rubenstein

B. Qualifications: Mr. Hill's and Mr. Rubenstein's qualifications are as noted in their resume contained in Appendix A.

C. Prior Filings: In addition to the statements herein, this testimony includes by reference the following documents submitted in this proceeding.

- Application for Certification, Volume 1 [Exhibit 1]

Documents Prepared by Others

- CARB (California Air Resources Board). 2006 Almanac of Emissions and Air Quality
- CARB (California Air Resources Board). 2005. Consolidated Table of OEHHA/ARB-Approved Risk Assessment Health Values, April 25, 2005. Available at: http://www.arb.ca.gov/toxics/healthval/healthval.htm. Last updated June 7, 2005. Accessed March 19, 2006.
- CARB (California Air Resources Board). HARP Model, Version 1.3.
- National Institute of Environmental Health Sciences. Environmental Health Institute report concludes evidence is 'weak' that EMFs cause cancer. Press release. National Institute of Environmental Health Sciences, National Institutes of Health, 1999.
- OEHHA (California Office of Environmental Health Hazard Assessment). 2003. Air Toxics Hot Spots Program Risk Assessment Guidelines, Guideline, The Air Toxics Hot Spots Program Guidance Manual for Preparation of Health Risk Assessments. CalEPA. August 2003.

To the best of our knowledge, all of the facts contained in this testimony (including all referenced documents) are true and correct. To the extent this testimony contains opinions, such opinions are our own. We make these statements, and render these opinions freely and under oath for the purpose of constituting sworn testimony in this proceeding.

II. Summary of Testimony

The risk assessment for the Project was conducted using the methodology and values for health risks developed by the California Office of Environmental Health Hazard Assessment (OEHHA). Emissions of non-criteria pollutants from Ivanpah SEGS were estimated using emission factors developed by the U.S. Environmental Protection Agency (USEPA). Air dispersion modeling was conducted following USEPA and CARB guidance for modeling. Health risks potentially associated with the estimated concentrations of pollutants in air were characterized in terms of potential lifetime cancer risk (for carcinogenic substances), or comparison with reference exposure levels (RELs) for non-cancer health effects (for non carcinogenic substances), using dose-response factors published by OEHHA.

Risks due to construction activities were estimated, as well as risks due to ongoing operation of the Project.

No significant public health effects are expected during construction. Construction-related emissions are temporary and localized, resulting in no long term significant impacts to the public. Strict construction practices that incorporate safety and compliance with applicable LORS will be followed. In addition, measures to reduce impacts from construction air emissions will be implemented as described in the AFC.

No significant public health effects are expected during operation. The non-criteria pollutants emitted from Ivanpah SEGS include certain volatile organic compounds (VOCs) and polycyclic aromatic hydrocarbons (PAHs) from the combustion of natural gas and Diesel exhaust particulate matter (DPM) from combustion of Diesel fuel in the emergency engines.

The nearest residence to Ivanpah SEGS is in Primm, Nevada, 5 miles northeast of the site. No daycare, hospital, park, preschool, or school receptors were found within 6 miles.

Beneficial aspects of Ivanpah SEGS regarding protection of public health include the following:

- Use the sun to generate electricity, and limiting the size and operation of combustion devices at the facility.
- Use of clean-burning natural gas fuel.
- Low-sulfur content of the natural gas, which reduces sulfate fine particulate generation.

There are no ambient monitors measuring toxic air contaminants in San Bernardino County. However, air quality and health risk data presented for the upwind South Coast Air Basin in CARB's *2006 Almanac of Emissions and Air Quality* show that over the period 1990 through 2005, the average concentrations for the top ten toxic air contaminants (TACs) have been substantially reduced, and the associated health risks for the air basin are showing a steady downward trend as well.

III. Proposed Licensing Conditions

The FSA for the project filed by the CEC does not recommend any Conditions of Certification to address Public Health.

IV. Correlation to FSA and Hearing Topics:

- Public Health

Socioeconomics

(Including Environmental Justice)

I. Introduction

A. Name: Fatuma Yusuf

B. Qualifications: Ms. Yusuf's qualifications are as noted in her resume contained in Appendix A.

C. Prior Filings: In addition to the statements herein, this testimony includes by reference the following documents submitted in this proceeding:

- Application for Certification, Volumes 1& 2 [Exhibit 1]
- Data Adequacy Supplement A [Exhibit 2]
- Comments to the PSA [Exhibit 57]
- Applicant's Response to CEC Staff Requests, Supplemental Data Response Set 1A, dated August 12, 2008, Response to Data Request 1 [Exhibit 32].

To the best of my knowledge, all of the facts contained in this Section of the Applicant's testimony (including all referenced documents) are true and correct. To the extent this testimony contains opinions, such opinions are my own based upon my professional judgment. I make these statements, and render these opinions freely and under oath for the purpose of constituting sworn testimony in this proceeding.

A. Affected Environment

The Ivanpah Solar Electric Generating System (Ivanpah SEGS) will be located in southern California's Mojave Desert, near the Nevada border, to the west of Ivanpah Dry Lake. The project will be located in San Bernardino County, California, on federal land managed by the Bureau of Land Management (BLM). It is proposed to be constructed in three phases: two 100-megawatt (MW) phases (known as Ivanpah 1 and 2) and a 200-MW phase (Ivanpah 3). The phasing is planned so that Ivanpah 1 (the southernmost site) will be constructed first, followed by Ivanpah 2 (the middle site), then Ivanpah 3 (the 200-MW plant on the north), though the order of construction may change.

B. Construction Impacts

Overall construction period for all three phases will be 48 months. Total construction personnel requirements will be approximately 6,654 person-months for Ivanpah 1; 6,584 person-months for Ivanpah 2; and 9,496 person-months for Ivanpah 3. When considering the overlap of all phases, the workforce will peak at 959 workers in month 32.

Available skilled labor in Riverside-San Bernardino-Ontario MSA in California and Las Vegas-Paradise MSA was evaluated by surveying local labor unions and contacting the California Employment Development Department and the Nevada Department of Employment, Training, and Rehabilitation. All sources show that the workforce in the area will be adequate to fulfill Ivanpah SEGS's labor requirements for construction. It is expected that most of the construction workforce will be drawn from the local area and/or will commute daily from within the MSAs to reach the job site. As a result, the construction of the Ivanpah SEGS will not create any significant adverse impacts to the local school system since there will likely be very few new students, if any, entering the local school districts. The construction of the proposed project will not cause significant demands on public services or facilities.

Ivanpah SEGS's initial capital cost is estimated to be about $1.1 billion. The estimated value of materials and supplies that will be purchased locally during construction is $77 million. The total local sales tax expected to be generated during construction is $6 million. Ivanpah SEGS will provide about $197 million in construction payroll, at an average salary of $50 per hour (including benefits). Since 95 percent of the construction workforce is assumed to reside in Clark County, about $187.2 million of the $197 million in construction payroll is assumed to stay in Clark County while the remaining five percent, or about $9.8 million, is assumed to stay in San Bernardino County over the 4-year construction period.

In addition to the direct impacts of the project, construction activity will result in secondary beneficial economic impacts (indirect and induced impacts) within San Bernardino and Clark counties. The estimated indirect and induced impacts result from the $41 million in annual local construction expenditures as well as the $137.9 million (disposable portion of the $197 million in annual spending – here assumed to be 70 percent) in spending by local construction workers.

C. Operational Impacts

Ivanpah SEGS is expected to employ up to 90 full-time employees: 35 with Ivanpah 1, 20 with Ivanpah 2, and 35 with Ivanpah 3. The entire permanent workforce is expected to commute from San Bernardino or Clark counties. The Ivanpah SEGS's operation will generate a small benefit by employing 90 full-time employees at an average annual salary of $60,000, resulting in an annual payroll of about $5.4 million. In addition to the payroll, there will be an annual operations and maintenance budget of $4 million, of which approximately $540,000 will be spent locally, within San Bernardino or Clark counties.

The operation of the proposed project would result in secondary beneficial economic impacts (indirect and induced impacts) that would occur within San Bernardino and Clark counties. These indirect and induced impacts represent permanent increases in the county's economic variables. The estimated indirect and induced impacts would result from annual $5.4 million in operations payroll as well as the $540,000 million in annual operations and maintenance (O&M).

There will be no significant impacts due to plant operations, since the entire permanent workforce is expected to commute from within San Bernardino and Clark counties. Ivanpah SEGS is expected to pay approximately $2.2 million per year in property taxes and about $2,090 in annual sales tax revenues to San Bernardino County.

Ivanpah SEGS will be in compliance with Guidances and the Executive Order 12898, Federal Actions to Address Environmental Justice in Minority and Low Income Populations (1994), because local minority and low-income populations will not be exposed to disproportionately high and adverse impacts from the project (AFC Section 5.10.6).

D. Cumulative Impacts

Because the majority of both construction and operations personnel will reside primarily in the Clark County, Nevada and live within commuting distance, no adverse effect to local schools or housing is anticipated. No adverse cumulative socioeconomic impacts are anticipated from either the construction or operation of Ivanpah SEGS. Instead, the local community will enjoy a beneficial (but not significant) impact from short-term construction and longer-term operations employment.

Despite the potential for construction schedule overlap with the Southern Nevada Supplemental Airport, no adverse cumulative socioeconomic effects are anticipated from either the construction or operation of Ivanpah SEGS.

E. Mitigation

The project has no significant socioeconomic or environmental justice impacts, so no mitigation measures are proposed.

II. Proposed Licensing Conditions

The FSA/DEIS for the project filed by the CEC and BLM does not recommend Conditions of Certification to address socioeconomic resource issues. We concur with this assessment.

III. Correlation to FSA and Hearing Topics:

- Socioeconomics and Environmental Justice

Soils

I. Introduction

A. Names: Steve Long, Kathy Rose, Tim Durbin, Mark Kubik, and Tom Reagan

B. Qualifications: The panel's qualifications are as noted in their resumes contained in Appendix A.

C. Prior Filings: In addition to the statements herein, this testimony includes by reference the following documents submitted in this proceeding:

- Application for Certification, Volume 1 & Volume 2 [Exhibit 1]
- Data Adequacy Supplement A [Exhibit 2]
- Comments to the PSA [Exhibit 57]
- Applicant's Response to CEC Staff Requests, Data Response Set 1A, dated January 14, 2008, Responses to Data Requests 53 through 80 [Exhibit 4]
- Applicant's Response to CEC Staff Requests, Data Response Set 1B, dated February 11, 2008, Responses to Data Requests 53 through 55, 57 through 60, 63 and 68 [Exhibit 5].
- Applicant's Response to CEC Staff Requests, Data Response Set 1D, dated May 9, 2008, Responses to Data Requests 53 through 60, 63, 66 through 68, 75 and 76 [Exhibit 7].
- Applicant's Response to CEC Staff Requests, Data Response Set 1E, dated July 22, 2008, Responses to Data Requests 57 through 58 [Exhibit 8].
- Applicant's Response to CEC Staff Requests, Data Response Set 1N, dated August 5, 2009, Responses to Data Requests 64, 65 [Exhibit 17].
- Applicant's Response to CEC Staff Requests, Data Response Set 1P, dated September 9, 2009, Responses to Data Requests 64 [Exhibit 19].
- Applicant's Response to CEC Staff Requests, Data Response Set 2A, dated June 10, 2008, Responses to Data Requests 133 through 145 [Exhibit 20].
- Applicant's Response to CEC Staff Requests, Data Response Set 2B, dated July 22, 2008, Responses to Data Requests 137, 139, 140 and 145[Exhibit 21].
- Applicant's Response to CEC Staff Requests, Data Response Set 2C, dated August 6, 2008, Responses to Data Requests 140B, 140E and 145 [Exhibit 22].
- Applicant's Response to CEC Staff Requests, Data Response Set 2H, dated May 13, 2009, Responses to Data Requests 140 [Exhibit 27].

- Applicant's Response to CEC Staff Requests, Data Response Set 2I, dated May 18, 2009, Responses to Data Requests 139 [Exhibit 28]
- Applicant's Response to CEC Staff Requests, Data Response Set 2J, dated June 17, 2009, Responses to Data Requests 139 [Exhibit 29]
- Applicant's Response to CEC Staff Requests, Supplemental Data Response Set 1A, dated August 12, 2008, Responses to Data Requests S&W-1 thought S&W-4 [Exhibit 32].
- Applicant's Response to CEC Staff Requests, Supplemental Data Response Set 2B, dated May 13, 2009, Responses to Data Requests Appendix 5.15-A2 [Exhibit 39]
- Applicant's Response to CEC Staff Requests, Supplemental Data Response Set 2F, dated June 5, 2009, Responses to Data Requests S&W-5 [Exhibit 43].

To the best of our knowledge, all of the facts contained in this testimony (including all referenced documents) are true and correct. To the extent this testimony contains opinions, such opinions are our own. We make these statements, and render these opinions freely and under oath for the purpose of constituting sworn testimony in this proceeding.

II. Summary of Testimony

A. Affected Environment

Project Site

The approximately 4,062-acre project site is located on BLM-managed land within the Ivanpah Valley, on the middle part of the bajada at the base of the Clark Mountains that lie to the west, in the Mojave Desert. The Ivanpah Valley is a closed basin and surface waters in the project area drain to and evaporate on Ivanpah Lake, a desert playa located about 1.6 miles to the east of the project site. The project site is characterized by relatively flat topography and desert scrub vegetation. Waterways in or near the project site include approximately 198.72 acres of unnamed ephemeral washes that dissect the project site and range in size from weakly expressed erosional features to large, broad drainages. No springs, seasonal or perennial creeks, or wetlands are located on or near the project site. Ephemeral washes are not under federal jurisdiction as Waters of the U.S.; however, all washes are considered to be Waters of the State.

Soil and Agricultural Resources

Information on types and distribution of soils within the project area was derived from the published NRCS soil survey report, "*Soil Survey of the Mojave Desert Area, Northeast Part, California*." Table 5.11-2 of the AFC identifies the four soil map units that occur in the area, along with their physical and chemical properties. Two different soil types would be directly affected by the project: Arizo loamy sand and Popups sandy loam. Arizo soils have loamy sand textures in surface soils and subsurface soils have gravelly sand textures. Popups soils have sandy loam surface soils, with stratified subsurface horizons having gravelly sandy clay loam and gravelly coarse sandy loam textures. Primary soil types are characterized by negligible to medium runoff potential, and with implementation of best management practices (BMPs) soil loss from water erosion is expected to be low. These soil types have high wind erodibility, and BMPs will need to be implemented during construction to reduce soil loss via wind. Soils have inherent limitations to support revegetation, and restoration of the site following decommissioning will need to include soil rehabilitation measures that are identified in the project's *Closure, Revegetation, and Rehabilitation Plan.*

The project site is not used for crop production; however, grazing does occur in the area under a grazing lease with the BLM. The project area is not designated as Important Farmland. Further discussion regarding agriculture is in the land use section.

B. Construction Impacts

Each of the three proposed solar plants will consist of heliostat fields surrounding a power block, which is supplied with the necessary utilities through a utility corridor. Construction of each project phase will result in temporary land disturbances, with site rehabilitation and revegetation in temporary-disturbance areas occurring as soon as practical upon completion of construction. For the purposes of this testimony, temporary disturbance is associated with the construction laydown area, parking, and equipment laydown/washing area. Permanent impacts would result from construction of facilities to be used during the 50-year lease period (e.g., structures and access roads), and site rehabilitation and revegetation of these areas will occur after closure and decommissioning. Taken together, the three project phases will result in permanent disturbance from facilities and linear features totaling about 3,231 acres; and temporary disturbance of about 831 acres of land.

To reduce environmental impacts, the project incorporates a low impact development (LID) approach that includes:

- Cutting vegetation to a height that will not interfere with construction and operation of the heliostat fields but not clearing or grading
- Restricting clearing and grading activities to areas where foundations, drainage facilities, and all-weather roads must be placed
- Taking advantage of the natural permeability of the alluvium at the site by minimizing compaction and decompacting soils where necessary
- Implementing a revegetation and rehabilitation program to accelerate the return of areas that have been temporarily disturbed to a vegetated state
- Implementing a stormwater control design that promotes sheet flow and greater infiltration, rather than channelization and concentration of stormwaters

Clearing and grading activities may include cuts to embankments to allow equipment access within washes; surface rock and boulder harvesting; limited vegetation removal; and light earth movement in limited areas to allow for vehicle and equipment access. Light grading for equipment access and boulder clearing, including rock harvesting, is anticipated in a 170-acre area in Ivanpah 3, where there may be up to 135,000 cubic yards (yd^3) of material graded and rock harvested. More extensive grading will be required in the power block areas, receiving towers, substation and administration/maintenance building areas. In these areas, earthwork cuts and fills will be approximately balanced, and the total quantity of cut is estimated at 245,000 cubic yards.

While the AFC estimated that 412,600 cubic yards of mulch might be generated from removal of vegetation, this figure would be substantially reduced following project modifications to incorporate low impact design and development strategies. Clearing and grubbing, where shrubs with their root systems are removed, will be limited to permanent access roads, power blocks, and in common areas where needed for equipment and materials access. Elsewhere, root systems will remain in place to anchor the soil and reduce erosion. Most of the natural drainage features will be maintained and any grading required will be designed to promote sheet flow where possible. Areas disturbed by grading and other ground disturbance will be protected from erosion by implementation of appropriate best management practices (BMPs) that are identified in the project's Stormwater Pollution Prevention Plan (SWPPP). A stormwater diversion channel will be constructed to direct storm flows around the substation and power blocks to protect those structures, and channel outlets will be designed to facilitate sheet flow.

The project design avoids impacts to Waters of the State wherever feasible, and minimizes impacts where avoidance was not possible. Fill impacts to Waters of the State are discussed in the Biological Resources section.

C. Operational Impacts

During project operations, impacts to soils could occur that are related to vehicle traffic, heliostat mirror washing and breakage, chemical spills, and other operations and maintenance activities. Routine vehicle traffic during operations would be limited to existing roads, and during the 50-year life of the project soil compaction in dirt access roads may occur to a depth of about 4 inches. Operations will require biweekly heliostat washing; the amount of wash water produced will not be great enough to result in runoff and soil erosion. Furthermore, the wash water will be of high quality and will not result in substantial salt or pollutant loading to the soil. During maintenance washing, vehicles will be moving less than 5 mph, and therefore, dust generation should not be a concern. Mirror breakage is expected at about 0.1 percent per year, and broken mirrors are anticipated to be changed out about once per year. A SWPPP will be finalized identifying BMPs that will be implemented to achieve substantive compliance with the State's Stormwater General Permit for Industrial Activities. A Drainage, Erosion and Sediment Control Plan (DESCP) will also be finalized and implemented to reduce or eliminate soil loss due to erosion during construction and operations. Implementation of the SWPPP and DESCP will ensure that any impacts to soils from project operations are minimized or avoided.

If stormwater peak flows substantially increase during project operations as the result of increase in impervious surfaces, removal of vegetation, and alteration of flow paths, then soil loss due to erosion and downstream sedimentation could likewise increase. Stormwater modeling, however, suggests that hydrologic changes will be minor and sedimentation is not expected to vary substantially from existing conditions.

D. Cumulative Impacts

Without the implementation of a SWPPP, DESCP, and other proposed mitigation, construction activities associated with the Ivanpah SEGS project could have the potential to increase cumulative wind and water erosion. However, implementation of the SWPPP during construction activities and the DESCP would ensure that the project would not contribute significantly to cumulative erosion and potential sedimentation impacts to the Ivanpah playa.

E. Mitigation

Erosion control measures will be required during construction to maintain water quality, protect property, and prevent accelerated soil erosion and/or dust generation. Construction and post-construction BMPs and stormwater monitoring protocols are identified in the draft Construction SWPPP and draft DESCP for the project. BMPs include erosion and sediment controls, tracking controls, stormwater diversion channels, wind erosion controls, and non-stormwater management. Post-construction BMPs include site revegetation following temporary construction impacts, and other measures to permanently stabilize soils. Additional waste discharge requirements have been proposed by the RWQCB and CEC to ensure adequate protection of beneficial uses of Waters of the State. With implementation of the Construction and Industrial SWPPPs, DESCP, and other waste discharge requirements, impacts to soil resources are less than significant and no further mitigation is required.

III. Proposed Licensing Conditions

The FSA/DEIS for the project filed by the CEC and BLM recommends that eight Conditions of Certification be adopted to address soil, and water resource issues, SOIL&WATER-1 through -8. Conditions of Certification SOIL&WATER-1 andSOIL&WATER-2 specifically relate to soil resources, and are addressed below. SOIL&WATER -3 through SOIL&WATER-8 are discussed in other testimony on Water Resources. Comments on Soil and Water Resources Appendixes B and C are contained in the Water Resources testimony.

Proposed Revisions to Soil&Water-1 and 2

As a general comment, the Applicant is opposed to conditions that require separate approvals of post-certification compliance activities by both BLM and the CPM because they are simply unworkable. If the approval is sequential, it will result in doubling the required approval time for everything. If the approval is concurrent, approvals may be potentially conflicting. As a general rule, consistent with current Commission practice, we have identified the Commission's CPM as the authority to review and approve post-certification compliance submissions or actions of the Applicant. It is also imperative that specific timeframes for approval be included in the Conditions so that the project will not be unnecessarily delayed. See the applicant's pre-hearing conference statement for a more-detailed explanation.

SOIL&WATER-1: Prior to site mobilization, the project owner shall obtain ~~both BLM's Authorized Officer and~~ the CPM's approval for a site specific DESCP that ensures protection of water quality and soil resources of the project site and all linear facilities for both the construction and operation phases of the project. This plan shall address appropriate methods and actions, both temporary and permanent, for the protection of water quality and soil resources, demonstrate no increase in off-site flooding potential, and identify all monitoring and maintenance activities. The project owner shall complete all ~~necessary~~ engineering plans, reports, and documents necessary for ~~both BLM's Authorized Officer and~~ the ~~CMP~~ CPM to conduct a review of the proposed project and provide a written evaluation as to whether the proposed grading, drainage improvements, and flood management activities comply with all requirements presented herein. The CMP will review and make a decision on all plans within thirty (30) days of those plans being submittedThe plan shall be consistent with the grading and drainage plan as required by Condition of Certification **CIVIL-1** and shall contain the following elements:

* * *

Soil Wind and Water Erosion Control: The plan shall address exposed soil treatments to be used during construction and operation of the proposed project for both road and non-road surfaces including specifically identifying all chemical based dust palliatives, soil bonding, and weighting agents appropriate for use at the proposed project site that would not cause adverse effects to vegetation; BMPs shall include measures designed to prevent wind and water erosion including application of chemical dust palliatives after rough grading to limit water use. All dust palliatives, soil binders, and weighting agents shall be approved by ~~both~~ the ~~BLM's Authorized Officer and the~~ CPM prior to us~~e~~e. The CPM shall make a determination of compliance with this certification within ten (10) days of submission of the plan.

Project Schedule: The DESCP shall identify on the topographic site map the location of the site-specific BMPs to be employed during each phase of construction (initial grading, project element construction, and final grading/stabilization). ~~Separate~~ BMP implementation schedules shall be provided for each project element for each phase of construction.

* * *

Agency Comments: The DESCP shall include copies of recommendations~~, conditions, and provisions~~ from the County of San Bernardino, California Department of Fish and Game (CDFG), and Lahontan Regional Water Quality Control Board (RWQCB).

Monitoring Plan: Monitoring activities shall include routine measurement of the volume of accumulated sediment in the onsite drainage ditches, and storm water diversions and the requirements specified in Appendix B, C, and D.[1]

<u>Verification</u>: The DESCP shall be consistent with the grading and drainage plan as required by Condition of Certification **CIVIL-1**, and relevant portions of the DESCP shall ~~clearly show approval by~~be submitted to the chief building official (CBO) for review and comment. In addition, the project owner shall do all of the following:

a. No later than ninety (90) days prior to start of site mobilization, the project owner shall submit a copy of the DESCP to the County of San Bernardino~~,~~ and the RWQCB~~, the BLM's authorized officer, and CMP~~ for review and comment. ~~Both BLM's Authorized Officer and t~~The CPM shall consider comments received from San Bernardino County and RWQCB.

1 Note: comments on Appendixes B, C, and D are in the Water Resources testimony

b. During construction, the project owner shall provide an analysis in the monthly compliance report on the effectiveness of the drainage-, erosion- and sediment-control measures and the results of monitoring and maintenance activities.

c. Once operational, the project owner shall provide in the annual compliance report information on the results of storm water BMP monitoring and maintenance activities.

~~d. Provide BLM's Authorized Officer and the CPM with two (2) copies each of all monitoring or other reports required for compliance with San Bernardino County, CDFG, and RWQCB.~~

Proposed Revisions to Soil&Water-2

Effective July 1, 2010, all construction-related discharges to Waters of the U.S. are required to obtain coverage under the Construction General Permit Order 2009-0009-DWQ adopted by the SWRCB on September 2, 2009. The draft Construction SWPPP will need to be modified, as necessary, to meet the substantive requirements of the revised General Permit, although coverage under the General Permit is not required since receiving waters are not deemed to be Waters of the U.S.

Because there will be no stormwater or non-stormwater discharges to Waters of the U.S. and, therefore, coverage under either the Construction General Permit or Industrial General Permit is not required, NOIs will not be filed with the SWRCB to obtain coverage under either permit and no NOIs will be submitted to the BLM or CPM. This is consistent with the "*Facts, Requirements, Surface Water Monitoring and Reporting Program, Ivanpah Solar Electric Generating Systems Project*," provided to the CEC by the RWQCB on October 12, 2009, and Appendix C part I.A. (Storm Water Discharges) on page 6.9-75 of the FSA/DEIS, which states:

> . . . The applicant shall comply with all requirements (*with the exception of purely administrative requirements, e.g., filing a Notice of Intent*) contained in State Water Resources Control Board's (SWRCB) Waste Discharge Requirements For Discharges of Storm Water Runoff Associated With Construction Activity, General Permit No. CAS00002; Waste Discharge Requirements For Discharges of Storm Water Associated With Industrial Activities, General Permit No. CAS00001; and all subsequent revisions and amendments. [emphasis added]

SOIL&WATER-2: The project owner shall comply with the requirements specified in Appendix B, C, and D for dredge and fill, wastewater, and storm water discharges associated with construction and industrial activity. The project owner shall develop, obtain ~~both BLM's Authorized Officer and~~ the CPM's approval of, and implement a construction Storm Water Pollution Prevention Plan (SWPPP) for the construction of the project and an Industrial SWPPP for operation of the project. The CPM will provide a final decision within thirty (30) days of submitting the SWPPP.

Verification: At least sixty (60) days prior to construction, the project owner shall submit to ~~both BLM's Authorized Officer and~~ the CPM a copy of the construction SWPPP for construction of the project for review and approval.

~~Verification~~: At least sixty (60) days prior to commercial operation, the project owner shall submit to ~~both BLM's Authorized Officer and~~ the CPM a copy of the Industrial SWPPP for operation of the project for review and approval prior to commercial operation. The project owner shall retain a copy on site. The project owner shall submit copies to ~~both BLM's Authorized Officer and~~ the CPM of all correspondence between the project owner and the RWQCB regarding the WDRs for discharge of storm water associated with construction and industrial activity within ten (10) days of its receipt or submittal. Copies of correspondence shall include the Notice of Intent sent by the project owner to the SWRCB.

IV. Correlation to FSA and Hearing Topics:

- Soil and Water Resources

Traffic and Transportation

I. Introduction

A. Name: Loren Bloomberg and Yoel Gilon

B. Qualifications: Mr. Bloomberg's and Mr. Gilon's qualifications are as noted in their resumes contained in Appendix A.

C. Prior Filings: In addition to the statements herein, this testimony includes by reference the following documents submitted in this proceeding:

- Application for Certification, Volume 1 [Exhibit 1]
- Data Adequacy Supplement A [Exhibit 2]
- Data Adequacy Supplement B [Exhibit 3]
- Comments to the PSA [Exhibit 57]
- Applicant's Response to CEC Staff Requests, Data Response Set 1A, dated January 14, 2008, Responses to Data Requests 81 through 90 [Exhibit 4].
- Applicant's Response to CEC Staff Requests, Data Response Set 1B, dated February 11, 2008, Responses to Data Requests 82 through 84 [Exhibit 5].
- Applicant's Response to CEC Staff Requests, Supplemental Data Response Set 1A, dated August 12, 2008, Response to Data Request TT-1 [Exhibit 32].
- Applicant's Response to CEC Staff Requests, Supplemental Data Response Set 1C, dated September 12, 2008, Response to Data Request TT-1 [Exhibit 34]
- Applicant's Response to CEC Staff Requests, Supplemental Data Response Set 1F, dated December 8, 2008, Response to Data Request TT-1 [Exhibit 37]

To the best of our knowledge, all of the facts contained in this testimony (including all referenced documents) are true and correct. To the extent this testimony contains opinions, such opinions are our own. We make these statements, and render these opinions freely and under oath for the purpose of constituting sworn testimony in this proceeding.

II. Summary of Testimony

A. Affected Environment

The site is located in a rural area of the Mojave Desert in San Bernardino County, 3.1 miles southwest of the California/Nevada border. This area is served by Interstate 15 (I-15) and local streets. Regional access to the site is provided from the south via I-15 and Highway 164 (Joshua Tree Highway, becoming Nipton Road at the California/Nevada border), which traverse through the region in a north-south and the east-west direction, respectively. To the north (south of Las Vegas), I-215 and Highway 604 are the closest major facilities that feed into I-15. Local roadways in the project vicinity include Yates Well Road and Colosseum Road (an unpaved roadway).

Traffic operations on the local streets and ramps are generally level of service (LOS) A due to the low traffic volumes. However, there are high traffic volumes on I-15 due to regional traffic between Southern California and Las Vegas (and beyond). Recurring congestion occurs on Friday evenings on northbound I-15 between San Bernardino and Las Vegas.

B. Construction Impacts

To analyze the worst-case scenario, traffic impacts associated with peak period construction traffic were considered. The analysis in the AFC concluded that the project was expected to generate approximately 243 daily round-trips, assuming that 60 percent of the workers would arrive by bus transport (15-passenger bus). This calculation was based on the assumption of 959 on-site workers in the peak month, plus additional vehicles for truck deliveries, for heliostat construction, power block construction, grading, and other construction activities.

Since the AFC was prepared, however, the construction market has changed significantly, along with the overall economy. Given the high unemployment and base of available workforce in Southern California, the Applicant expects to draw most of the workforce from the Inland Empire areas of Riverside and San Bernardino Counties, instead of Las Vegas. A recent article in the *Press-Enterprise* (http://www.pe.com/localnews/inland/stories/PE_News_Local_S_ivanpah12.4522948.html) is a good summary of the availability of this workforce.

With these assumptions, the travel patterns associated with construction of the project will be different. With most of the workforce coming from Southern California, it is expected that construction workers will likely stay in Primm or Las Vegas during the weekdays (staying in hotels, apartments, RVs, etc.). Then, they will go home for the weekend.

Therefore, the owner no longer expects that providing buses will be necessary. Carpooling will still be encouraged, as workers drive their own vehicles to the work site each day. However, with this scenario, many workers will drive alone on Friday mornings so that they can return to Southern California after work on Friday.

The assumptions for the workforce travel patterns are as follows:

- On Mondays, 50% of the workers will have traveled the night before from Southern California (or stayed the weekend) and will be commuting from Nevada (i.e., southbound (SB) on I-15). The other 50% would travel directly from Southern California on Monday morning. All would return to temporary housing in Nevada. The average occupancy for all vehicles would be 2.0.
- On Tuesdays, Wednesdays, and Thursdays, all workers would commute from temporary (or permanent housing in Nevada). A slightly higher occupancy rate (2.5 persons per vehicle) is expected.
- On Fridays, all workers would commute from Nevada, but at a lower average occupancy rate (1.5) since carpooling would be more difficult with varied travel patterns on Friday night. Some workers might choose to drive alone back to their Southern California homes,

while others would choose to carpool back to Nevada for the weekend. It is assumed that 75 percent of the workers would return south on I-15 to Southern California.

For truck deliveries, the following assumptions were part of the analysis:

- On Mondays to Thursdays, 55 truck deliveries will be made daily.
- On Fridays, truck deliveries will only occur in the morning (before noon), so only half as many are expected (28 trucks per day).
- The truck deliveries will be spread evenly throughout the day, so 10 percent will occur in the peak hours.
- Up to 100 concrete trucks per day will be needed occasionally, but not on Fridays.
- Except for concrete trucks (which will all come from Nevada), half of all trucks will come from California, and half with come from Nevada.

Given the different travel patterns on different days, it is important to understand the traffic volumes on I-15 on different days. The original analysis in the AFC indicated that operations on I-15 were the most critical part of the system.

Wednesdays through Friday were considered. Tuesdays will have the same construction traffic patterns as Wednesdays, and similar baseline traffic. Monday morning traffic impacts will be less, because construction traffic will be spread from two directions. Monday evening traffic impacts will be the same as Tuesday and Wednesday evenings.

Figures T&T-1 and T&T-2 are graphs of the traffic volumes (per day) on NB and SB I-15. Except for November (heavily influenced by Thanksgiving travel), Fridays are much higher than Thursdays, which are somewhat higher than Wednesdays. Although Thursdays and Wednesdays will have the same construction traffic, the data indicate that these two days need to be analyzed separately since baseline traffic volumes are different.

FIGURE T&T-1
Average NB Traffic Volume on I-15, Wednesday to Friday

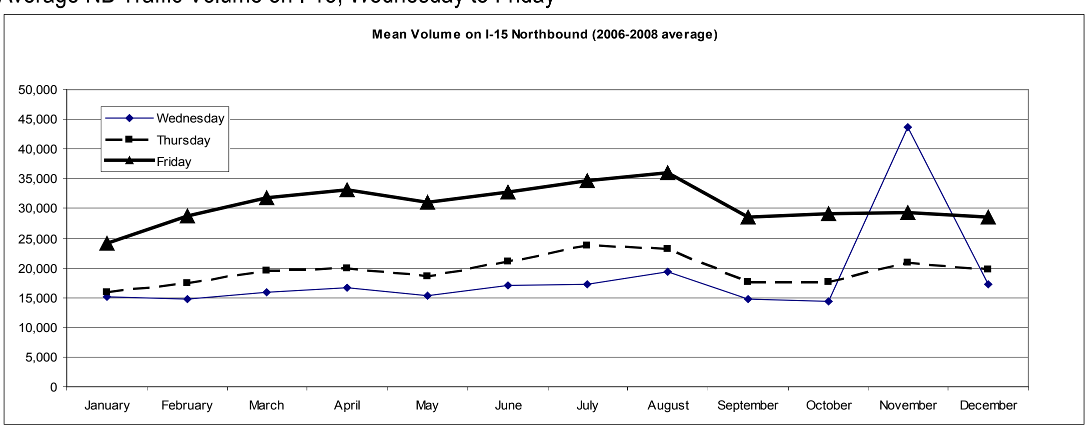
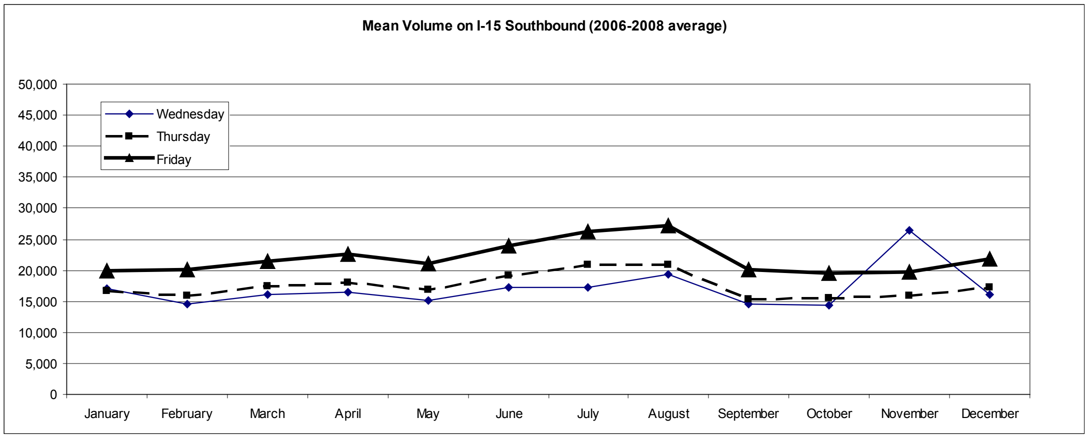


FIGURE T&T-2
Average SB Traffic Volume on I-15, Wednesday to Friday



To better understand the impacts on I-15, a focused analysis of the total daily traffic was conducted on I-15. The volume/capacity (v/c) ratios were calculated separately for NB and SB I-15 for the three days, considering the construction traffic described above.

Additional traffic volumes associated with the project on Wednesdays and Thursdays are 440 vehicles/day (for both NB and SB I-15). On Fridays, the construction traffic volumes are 174 vehicles/day NB and 1136 vehicles/day SB. Table T&T-1 is a summary of the results for v/c ratios, using a capacity of 36,000 vehicles/day in each direction (from the FSA).

TABLE T&T-1
I-15 Construction Traffic Analysis

Segment	Capacity	Scenario	Wednesday		Thursday		Friday	
			Volume	V/C	Volume	V/C	Volume	V/C
NB I-15	36,000 vehicles/ day	Existing	18,444	0.51	19,601	0.54	30,667	0.85
		Construction	18,884	0.52	20,041	0.56	30,841	0.86
SB I-15		Existing	17,028	0.47	17,477	0.49	22,001	0.61
		Construction	17,468	0.49	17,915	0.50	23,137	0.64

The construction analysis in this table reflects the typical day. When concrete truck deliveries occur (during concrete placement), the number of daily trucks will increase by up to 100 vehicles/day. In most cases, this will not change the V/C ratios (rounded to the nearest 0.01).

When evaluated on a daily basis, there is sufficient capacity on I-15 during construction. Construction traffic will add one to three percent to the daily volumes on I-15. The highest construction traffic volumes are in the reverse (SB) direction. The updated assumptions for the workforce result in less of an impact, because more of the traffic will be SB on Friday nights.

While the traffic volumes will be lower on NB I-15 on Friday nights, there is still the possibility of significant impacts without mitigation, because NB I-15 is congested for several hours on Fridays.

The intersection analysis was also revisited. Monday is the worst-case scenario, where more conflicts occur at Yates Wells Road and I-15 SB ramps.

The results are presented in Table T&T-2. All intersections operate at a satisfactory level. Note that although the Yates Well Road/I-15 NB ramps operate at LOS C for the minor approach, only three vehicles will be affected. The majority of the vehicles will experience LOS A.

TABLE T&T-2
Intersection LOS – Monday (Construction Scenario)

Intersection	AM Peak Period (Delay in seconds – LOS)	PM Peak Period (Delay in seconds – LOS)
Colosseum Road/Yates Well Road	12.3 – B	9.6 – A
I-15 SB ramps/Yates Well Road	12.0 – B	8.4 – A
I-15 NB ramps/Yates Well Road	9.8 – A	19.8 – C

In summary, an operational analysis of the intersections in the project vicinity revealed that the construction of the Ivanpah SEGS will not result in significant changes in LOS on the local roads or the freeway. However, NB I-15 already operates at LOS F on Fridays. The project cannot cause further degradation (i.e., there is no LOS category below F), but the project will result in the addition of 174 daily vehicles to NB I-15. While the number of vehicles is minor as compared to the total number of vehicles on I-15 (and lower than the original estimate), the project will add to the congestion. As discussed below, the implementation of a Traffic ControL Plan (TCP) would address workers trips on Friday afternoons, to minimize impacts to I-15. With the implementation of appropriate TCP measures, the impact on I-15 traffic can be reduced to a less-than-significant level.

C. Operational Impacts

The operational workforce for all three phases is projected to be 90 people—at least 60 of which will work a night shift. The operational workforce is substantially less than the construction workforce, so the traffic impact assessment was conducted only for the construction workforce.

D. Glare and Reflectivity

Staff has proposed two Conditions of Certification relating to impacts of glare and reflectivity on receptors including hikers, motorists and aircraft pilots and passengers. In TRANS-3, Staff proposes that Applicant prepare a Heliostat Positioning Plan (HPP) and a Heliostat Operations Plan (HOP). From a practical standpoint these two plans would include the same information since heliostat positioning is the primary element of heliostat operation. Applicant disagrees with Staff on the health and safety issues put forth due to off target and concentrated heliostat reflectance. Each heliostat has a unique physical location coded into the heliostat operation and positioning program. Each heliostat is also individually programmed with the location of the solar receiver and calculates the location of the sun with great precision as it tracks across the sky. The positioning and movements of each of the heliostats is planned, coordinated and managed by a central computer that ensures safe operation of the heliostat field, not only in terms of the solar flux reflected onto the SRSG, but also in terms of controlling where beams are reflected at those times when any particular heliostat is not targeting the SRSG. Each heliostat is equipped with a heliostat controller (HC) that specifically incorporates the functionality of independently positioning the heliostat to aim its reflected beam to a defined (x,y,z) location. Among other built-in safety features, the HC will have a programmed border limitation such that aiming points are checked to ensure that they do not fall outside the boundaries of the solar field, and within the 1350 feet maximal height in the sky.

Since heliostats are individually controlled based on their unique location and instant position, yet centrally directed, the potential for heliostats to collectively refocus on a location that would impact hikers, motorists or aircraft pilots and passengers is non-existent. Applicant nonetheless agrees to prepare an HPP that will explain the operation of the heliostats including operating and positioning methodology, and alarms that are provided to plant operators in the event that a heliostat malfunctions.

Staff is also proposing that Applicant undertake a program to measure luminance intensity at various locations both on and off the project site for every solar receiver on all four sides of each receiver. Staff further proposes that if the luminance intensity exceeds a "standard" of 89 cd/m2 at the property line, that power output of the facility be reduced to comply with the "standard". Staff in its own testimony admits that there are no regulations governing reflectance of solar receivers (FSA/DEIS Section 6.10-13) while citing maximum exposure limits for momentary exposure of 10 kw/m2 and continuous exposure limits of 1 kw/m2. Applicant will address each of the limits separately.

Continuous Exposure Limit: This limit is typical luminance intensity during a sunny day in the desert. Additionally, for persons in high mountain regions, the continuous luminance intensity from the sun is typically 1.2 kw/m2. It is not practical to manage operation of the facility to be less than ambient exposure.

Momentary Exposure Limit: Staff states in its testimony that the momentary exposure limit for heliostat reflectance poses a risk for retinal injury only if an individual elects to stare at a heliostat reflecting directly into his/her eyes and not looking away. Additionally, Staff states that the only observers that could be close enough to exceed the momentary exposure limit would be low flying aircraft that hypothetically could be at an elevation less than 1,000 meters above the site, or from an observer on foot. Applicant will not argue the hypothetical concurrent possibility of a heliostat malfunction and an aircraft flying over the site at that low altitude or the opportunity for hikers to observe a heliostat directly from the property line, but also recognizes that the probability of either of these occurrences is extremely low.

Staff states that the level of reflected light intensity from a heliostat would be a maximum of 3.125 kw/m2 at a distance of 500 meters, based on Applicant's previously submitted data. In fact Applicant's latest analysis, based on some 1,600 heliostats in service for more than a year at Applicant's pilot plant in Israel, shows that the level of reflected light intensity would be at or above the continuous exposure limit of 1 kW/m2 only at a distance of 400 meters or less from the heliostat, and only under specific circumstances combining very clear-sky insolation and a narrow range of sun angles, and a specific corresponding range of heliostat elevation angle as well. At more than 400 meters, whether on the ground or in the sky, the reflected light will be below the continuous exposure limit of 1 kW/m2. In any case, the potential for a heliostat to be positioned such that sunlight is reflected onto a motorist, hiker, or aircraft pilot or passenger is remote. A hiker would have to be less than 400 meters from the edge of the

> plant precisely at the moment, statistically rare, that a heliostat is 'stuck' in the hiker's direction. A motorist traveling on I-15 past the field when such a malfunction occurred, would pass through the beam in about 0.2 seconds if travelling 40 mph.
>
> It's important to note, therefore that the distance that a person is from the heliostat has a dramatic impact on relative intensity. All heliostats at the ISEGS facility are generally directed towards the solar receivers atop the towers in the center of the solar field. Therefore, the most probable occurrence for a ground-borne observer, whether afoot or in a car, would be from a heliostat on the opposite side of the solar field from the point of observation. The minimum radius of the solar field is approximately 1,000 meters, which means that the closest heliostat that the theoretical ground-borne observer could be impacted by is approximately 1,100 to 1,200 meters away. At this range, the illuminance from the heliostat would be 80% to 90% less than the continuous exposure limit if that person could even see the heliostat on the opposite side of the solar field. The most likely probability is that the ground-borne observer would be unable to see the offending heliostat since there would be many rows of interceding heliostats. Notwithstanding that the expected illuminance would be substantially less than either of the Staff's limits, the reasonable action that would occur if a heliostat was directed at the theoretical observer, and is human nature, is to look away and not stare at a bright point of light. Therefore the potential for retinal injury from the heliostat, where the expected luminance intensity would be less, or in most cases considerably less, than 1 kw/m2 is not possible. (FSA/DEIS Section 6.10-14 Table 7)

Staff is also proposing that Applicant undertake a program to measure luminance intensity at various locations both on and off the project site for every solar receiver on all four sides of each receiver. Staff further proposes that if the luminance intensity exceeds a "standard" of 89 cd/m2 at the property line, that power output of the facility be reduced to comply with the "standard". Staff in its own testimony admits that there are no regulations governing reflectance of solar receivers (FSA/DEIS Section 6.10-13). In fact, the Illuminating Energy Society of North America (IESNA Lighting Handbook, 9th Edition, Dec. 2000, Chapter 22) recommends a <u>minimum</u> luminance of 89 cd/m2 for externally lighted roadway signs in urban areas to ensure their readability <u>at night</u>. Such a lighting level can barely be discerned during daylight hours, let alone pose a hazard or distraction. Staff also states in its testimony that there is no safety issue when applying the MPE limit discussed above to the glow of the solar receivers. However, Staff elects to recommend an onerous, expensive, and ineffective Condition of Certification (TRANS-4). Staff's argument is that motorists on I-15 would be surprised to see stationary towers to the north and west of I-15 glowing in the daytime. This argument is specious in that once the plant is constructed, the glow from the solar towers would quickly become a common item of observation. The recommended COC (TRANS-4) sets a limit of only 89 cd/m2 at the property line. Exceeding this limit would require the Applicant to reduce power so that the illuminance would stay below this "limit". Note that the continuous exposure limit that the staff refers to above is 1kw/m2. When simply applying the figures from Staff's testimony in the FSA, Section 6-10 Table 7, the expected illuminance at a distance of 1,000 meters (still inside the property line) is 0.0007 kw/m2. This is 0.07% of the continuous exposure limit discussed above. Applicant therefore believes that Condition of Certification TRANS-4 is unneeded, and should be deleted.

E. Cumulative Impacts

Projects that are reasonably foreseeable include the Desert Xpress Rail Line, Caltrans improvements to I-15, the Southern Nevada Supplemental Airport (Ivanpah Valley Airport), and the FirstSolar photovoltaic project. Cumulative traffic impacts could occur if construction of these projects overlapped causing a combined impact to Friday afternoon traffic on I-15. The construction schedules are not set for all of these planned projects, but it is likely that there will be some overlap with Ivanpah SEGS. These impacts would be mitigated by a combination of the proposed Ivanpah SEGS mitigation measures and the mitigations proposed for those specific projects.

F. Mitigation

The implementation of a Transportation Control Plan (TCP) would address workers trips on Friday afternoons, to minimize impacts to I-15. The specific TCP elements should be identified once the specifics of the selected Construction Contractor's schedule are known, but should include provisions for staggering shifts and worker departure times, buses for workers, and provisions for monitoring. With the implementation of appropriate TCP measures, the impact on I-15 traffic can be reduced to a less-than-significant level.

III. Proposed Licensing Conditions

The FSA/DEIS for the project filed by the CEC and BLM recommends that six Conditions of Certification be adopted to address traffic and transportation issues: TRANS-1 through TRANS-6. TRANS-5 is acceptable. The Applicant proposes the following changes to the other conditions.

Proposed Revisions to TRANS-1

For TRANS-1, the requirement to develop a Traffic Control Plan (TCP) or TMP is generally acceptable. However, TCPs and TMPs are similar documents, so it is recommended that a single document be required.

Also, based on the analysis described above, buses are not necessary to maintain the current LOS. On Fridays, the proposed mitigation to limit egress from the site to 12 or fewer vehicles every 3 minutes between 12 noon and 10 PM on Fridays is acceptable.

The language for TRANS-1 should be modified as follows:

TRANS-1 Prior to start of construction of the ISEGS, the project owner shall prepare and implement a Traffic Control Plan (TCP) for ISEGS construction and operation traffic. The TCP shall address~~ing~~ the movement of workers, vehicles, and materials, including arrival and departure schedules, and designated workforce and delivery routes. The plan shall include:

- ~~requiring at least 60% of construction workers to arrive to the site by bus transport (15 people per bus);~~
- ~~limiting~~ truck deliveries to the project site ~~to no more than 12 truck trips per day~~on Fridays will be limited to mornings only ~~and~~so that they occur before 12:00 noon ~~, limiting truck deliveries to the project site to Mondays through Thursdays only~~;

- A work schedule and end-of shift departure plan will be implemented to limit Friday departures from the site, traveling north to Las Vegas, to 12 or fewer vehicles every 3 minutes between 12:00 noon and 10:00 pm.

* * *

The project owner shall consult with the County of San Bernardino and the Caltrans District 8 office in the preparation and implementation of the Traffic Control Plan and shall submit the proposed Traffic Control Plan to the County of San Bernardino and the Caltrans District 8 office in sufficient time for review and comment and to ~~BLM's Authorized Officer and~~ the Energy Commission Compliance Project Manager (CPM) for review and approval prior to the proposed start of construction and implementation of the plan. The CPM shall review and approve within thirty (30) days of receipt. The project owner shall provide a copy of any written comments from the County of San Bernardino and the Caltrans District 8 office and any changes to the Traffic Control Plan to ~~BLM's Authorized Officer and~~ the CPM prior to the proposed start of construction.

Verification: At least 90 calendar days prior to the start of construction, including any grading or site remediation on the power plant site or its associated easements, the project owner shall submit the proposed traffic control plan to the County of San Bernardino and the Caltrans District 8 office for review and comment and to ~~BLM's Authorized Officer and~~ the CPM for review and approval. The project owner shall also provide ~~BLM's Authorized Officer and~~ the CPM with a copy of the transmittal letter to the County of San Bernardino and the Caltrans District 8 office requesting review and comment.

At least 30 calendar days prior to the start of construction, the project owner shall provide copies of any comment letters received from either the County of San Bernardino and the Caltrans District 8 office, along with any changes to the proposed traffic control plan to ~~BLM's Authorized Officer and~~ the CPM for review and approval.

Proposed Revisions to TRANS-2

BLM or Caltrans would have no need for copies of the video taped roadways. Hence, the Applicant proposes the following changes to TRANS-2 Verification:

TRANS-2

Verification: At least 30 days prior to the start of mobilization, the project owner shall photograph or videotape all affected public roads, easements, and right-of-way segment(s) and/or intersections and shall provide ~~BLM's Authorized Officer,~~ the CPM~~, the affected local jurisdiction(s) and Caltrans (if applicable)~~ with a copy of these images.

Within 60 calendar days after completion of construction, the project owner shall meet with ~~BLM's Authorized Officer and~~ the CPM~~, the County of San Bernardino and Caltrans District 8~~ to identify sections of public right-of-way to be repaired. At that time, the project owner shall establish a schedule to complete the repairs and to receive approval for the action(s). Following completion of any public right-of-way repairs, the project owner shall provide a letter signed by the County of San Bernardino and Caltrans District 8 stating their satisfaction with the repairs to ~~BLM's Authorized Officer and~~ the CPM.

Proposed Revisions to TRANS-3

Staff proposes a Heliostat Positioning Plan (HPP) and a Heliostat Operations Plan (HOP). As a point of practicality, these plans would be one and the same since heliostat positioning is the primary point of heliostat operation. Therefore, Applicant proposes creation of a single plan - the Heliostat Positioning Plan. The proposed criteria to be avoided and mitigated through the HPP listed in under item 3, subparagraph b, are similar to the exposure a person receives while outdoors on a sunny day. In mountain environments, exposure from the sun commonly exceeds even 1.2kW/m^2. It is unreasonable and excessively costly to prepare a monitoring and mitigation plan that uses as the basis for mitigation, the background exposure that any person is subject on a clear sunny day.

The remaining changes are to organize the HPP such that it identifies how the heliostats are programmed, how they move, and potential malfunctions that could result in stray solar reflections impacting observers off site. Finally, the HPP will provide requirements and procedures to follow in the unlikely event of a legitimate complaint that must be investigated.

TRANS-3 The project owner shall prepare a Heliostat Positioning Plan that would accomplish the following:

~~1. Identify potential sensitive receptors including observers in aircraft, motorists on I-15, hikers in the Clark Mountains and other hikers and motorists who could access locations closer to the project;~~

1.~~2~~. Identify the heliostat movements and positions (including reasonably possible malfunctions) that could result in exposure to potential observers to reflected solar radiation from heliostats including observers in aircraft, motorists and hikers in the Clark Mountains;

2~~3~~. ~~Prepare a Heliostat Operating Plan that~~Describe within the HPP how programmed heliostat operation would avoid potential for human health and safety hazards at locations of sensitive receptors described in paragraph 1 above in this C~~O~~ondition, including the how programming will eliminate the potential for momentary ~~and continuous~~ solar radiation exposure at ~~to occur~~ greater than the thresholds ~~of significance~~ of ~~:~~

~~a.~~ MPE for momentary exposure (for a period of 0.25 second or less) is 10 kw/m^2

~~b. MPE for continuous exposure (for a period greater than 0.25 second) is 1 kw/m^2~~

3~~4~~. ~~Prepare a monitoring plan that would: a) verify that the Heliostat Operating Plan would avoid potential for human health and safety hazards at locations of sensitive receptors, and b)~~Within the HPP provide requirements and procedures to document, investigate and resolve legitimate complaints regarding glare.

~~5~~4. The monitoring plan should be coordinated with the FAA, U.S. Department of the Navy, CalTrans, and Clark County Department of Aviation in relation to the proposed Southern Nevada Supplemental Airport and be updated on an annual basis for the first 5 years, and at 2-year intervals thereafter for the life of the project.

Verification: Within 90 days before commercial operation of any of the three ISEGS power plants, the project owner shall submit the Heliostat Positioning Plan to the CPM for review and approval. The project owner shall also submit the plan to CalTrans, FAA, and the Clark County Department of Aviation for review and comment and forward any comments received to the CPM.

Proposed Revisions to TRANS-4

Staff is proposing an extremely onerous condition including monitoring and mitigation that would be nearly impossible to implement and manage and would likely render solar power tower technology as unfeasible. There is no practical method for measuring the level of brightness from the solar power towers such that the proposed mitigation would be meaningful. Staff's recommended mitigation for exceeding the dubious proposed standard of 89 cd/m^2, would be to reduce plant output. The Staff's testimony states that the solar receiver brightness at I-15 even under the highest expected intensity, would be only 38 cd/m^2., which is the same brightness experienced from viewing a 100-watt light bulb from a distance of 35 meters (115 feet). Staff had the Applicant's intensity numbers independently verified and Staff states that it does not disagree with Applicant's assessment. Based on this factor alone, the proposed COC should be eliminated.

As a practical matter there is no way of being able to determine compliance with the standard on a day-to-day basis. People and animals all over the world, and for thousands of years, understand not to stare at the sun or bright lights because it results in eye discomfort and pain.

Staff states that the primary risk is for "surprised" motorists traveling on I-15 due to the high rate of speed. Staff states that while motorists expect to see the sun either rising or setting on the horizon, they would not expect reflections from 459-foot solar towers in the desert. This argument is specious in that motorists are not likely to be surprised by the steady glow of a family of stationary towers located at least 1.5 miles to the north and west of the freeway. Especially since the towers would be in the same place for decades. Motorists have mitigation features already in place in there autos including tinted glass and sun visors. In addition, on sunny days, when the solar tower's glow would be at its highest level, many motorists would be wearing sunglasses as a personal preference to further attenuate brightness.

Therefore, based on Staff's own assessment, the impracticality and ineffectualness of the mitigation proposal, the common sense measures already in existence, and human nature not to stare at something that would cause them discomfort, Applicant proposes deletion of Condition of Certification **TRANS-4**

Proposed Revisions to TRANS-5

This condition is acceptable.

Proposed Revisions to TRANS-6

The Applicant believes that it would be more appropriate to move some of the condition language to the verification section.

TRANS-6 Prior to start-up and testing activities of the plant and all related facilities, the project owner shall coordinate with the FAA to notify all pilots using the airspace in the vicinity of the ISEGS of potential air hazards from turbulence. ~~These activities would include, but not be limited to: 1) issuing a notice to airmen (NOTAM of the identified air hazard, 2) updating all applicable FAA-approved airspace charts to indicate that plume hazards could exist up to an altitude of 1,350 feet above the ground surface, and 3) requesting FAA to require pilots to avoid direct overflight of the ISEGS site at or below this altitude during daylight hours.~~

Verification: At least 60 days prior to start of project operation, the project owner shall submit to ~~BLM's Authorized Officer and~~ the CPM for review a letter from the FAA showing compliance with these measures. These notification activities would include, but not be limited to: 1) issuing a notice to airmen (NOTAM of the identified air hazard, 2) updating all applicable FAA-approved airspace charts to indicate that plume hazards could exist up to an altitude of 1,350 feet above the ground surface, and 3) requesting FAA to require pilots to avoid direct overflight of the ISEGS site at or below this altitude during daylight hours.

IV. Correlation to FSA and Hearing Topics:

- Traffic and Transportation

Visual Resources

I. Introduction

A. **Name:** Wendy Haydon, Thomas Priestley and Yoel Gilon

B. **Qualifications:** The panel's qualifications are as noted in their resumes contained in Appendix A.

C. **Prior Filings:** In addition to the statements herein, this testimony includes by reference the following documents submitted in this proceeding:

- Application for Certification, Volumes 1 & 2 [Exhibit 1]
- Data Adequacy, Supplement A, dated October 5, 2007, Section 5.13 Visual Resources [Exhibit 2].
- Preliminary Staff Assessment Comments, Set 1, Visual Resources #128 through #152 [Exhibit 57].
- Applicant's Response to CEC Staff Requests, Data Response Set 1A, dated January 14, 2008, Responses to Data Requests 97 through 110 [Exhibit 4].
- Applicant's Response to CEC Staff Requests, Data Response Set 1B, dated February 11, 2008, Responses to Data Request 100 [Exhibit 5].
- Applicant's Response to CEC Staff Requests, Data Response Set 1D, dated May 9, 2008, Responses to Data Requests 97 and 102 [Exhibit 7].
- Applicant's Response to CEC Staff Requests, Data Response Set 2A, dated June 10, 2008, Responses to Data Requests 146 through 151 [Exhibit 20].
- Applicant's Response to CEC Staff Requests, Data Response Set 2B, dated July 22, 2008, Responses to Data Request 148 [Exhibit 21].
- Applicant's Response to CEC Staff Requests, Data Response Set 2C, dated August 6, 2008, Responses to Data Requests 147 through 148 [Exhibit 22].
- Applicant's Response to CEC Staff Requests, Supplemental Data Response, Set 2H, dated June 9, 2009, Responses to Data Requests VR-1 [Exhibit 45].
- Applicant's Response to CEC Staff Requests, Supplemental Data Response, Set 3A, dated July 23, 2009. Responses to Data Requests VR-2 through VR-6 [Exhibit 48].
- Applicant's Response to CEC Staff Requests, Supplemental Data Response, Set 4, dated August 20, 2009. Responses to Data Requests VR-8 through VR-12 [Exhibit 49].

To the best of our knowledge, all of the facts contained in this testimony (including all referenced documents) are true and correct. To the extent this testimony contains opinions, such opinions are our own. We make these statements, and render these opinions freely and under oath for the purpose of constituting sworn testimony in this proceeding.

II. Summary of Testimony

Affected Environment

Description of the Visual Setting

The project is proposed to be developed in unincorporated San Bernardino County in the Mojave Desert approximately 0.8 mile to the west of I-15 at its closest point (southeast corner of Ivanpah 1), and approximately 3.1 miles south of the California/Nevada border (closest point between Ivanpah 3 and the State Line).

The physical setting in which the project would be located consists of an area that is vegetated with grasses and low-lying scrub bushes. The elevation of the property ranges from 3,525 feet at the northwest corner, sloping to 2,800 feet elevation at the southeast corner of the property. Overhead electric transmission lines are located in the project vicinity, crossing the project site. One transmission line corridor with three transmission lines is oriented in a southwest-northeast direction, passing between Ivanpah 1 and Ivanpah 2.

The Ivanpah Dry Lake is situated to the east of the three project sites, and is bisected by I-15. The dry lake covers an area of approximately 35 square miles. It is a popular place for kite buggying, land sailing, long-distance archery, and kite demonstrations. The North American Buggy eXpo, a week-long event, recently occurred in April 2007 at the lake. The lake area is open to non-motorized vehicle access only; it is closed to motorized vehicles without a permit. The gate to the lake area at the Yates Well Road exit off I-15 indicates that the area is closed.

Located to the northeast are casinos in Primm, Nevada on the east and west sides of I-15, apartments for casino employees located behind (east of) the casinos on the east side of I-15 (described below), and a power plant (Reliant's Bighorn Generating Station) is located on the east side of I-15. To the east of Ivanpah 1, on the east side of I-15 and the Yates Well Road exit, is a residence (described below) and additional buildings that appear to be abandoned along with a communications tower. Paralleling I-15 on its east side are railroad tracks.

The Mescal and Ivanpah ranges are located to the west of the project area, and the Clark Mountain Range forms the valley's northwestern border. The nearest topographical feature is a metamorphic outcrop located east of Ivanpah 2 and Ivanpah 3. In addition, Ridge 1059 is a quartzite outcrop located to the west of Ivanpah 3. The New York Mountains, Providence Mountains, and the Granite Mountains are located to the southeast, south, and southwest of the project area. The Mojave National Preserve is located to the southwest of the project area.

The Primm Valley Golf Club is located approximately 0.5 mile east of the Ivanpah 1 property boundary, and is approximately 1.5 miles east of the Ivanpah 2 plant boundary. The Golf Club is located on an approximately 500-acre parcel of land, and consists of two golf courses: the Desert Course and the Lakes Course. Each course has 18 holes, and is approximately 150 acres. Holes 1 and 2 in the southwest portion of the Desert Course and Holes 6, 7, and 8 in the northwest portion of the Desert Course provide views of the project sites. Hole 8 is the highest in elevation. Holes 10 through 18 of the Desert Course are lower in elevation and are located to the east of Holes 1 through 9, so views of the project are not available from there. The Lakes Course, located to the east of the Desert Course and at a lower elevation than the Desert Course, does not provide views of the project.

The nearest residence to the project sites is one trailer located on the east side of I-15 at the Yates Well Road exit. This residence is located approximately 1.4 miles east of the Ivanpah 1 project facility boundary. In addition, there is a casino employee apartment complex located approximately 5 miles northeast of the project. It is located to the east of the hotels/casinos that are situated on the east side of I-15 in Primm, Nevada. The casino employee apartment complex has no views of the project due to the presence of the casinos that are between the apartments and the project. There may be views of the project from some hotel rooms in the hotels/casinos in Primm; however, due to the 5-mile distance between the hotels and the project sites, views of the sites would be a small part of the overall view. In addition, persons staying at hotels are considered transient and are not considered to be sensitive viewers.

Description of the Project

The Applicant proposes to develop a solar energy project called the Ivanpah Solar Electric Generating System (Ivanpah SEGS). It will be located in southern California's Mojave Desert, near the Nevada border, to the west of Ivanpah Dry Lake. The project will be located in San Bernardino County, California, on federal land managed by the Bureau of Land Management (BLM). It is proposed to be constructed in three phases: two 100-megawatt (MW) phases (known as Ivanpah 1 and 2) and a 200-MW phase (Ivanpah 3). The expected phasing is that Ivanpah 1 (the southernmost site) will be constructed first, followed by Ivanpah 2 (the middle site), then Ivanpah 3 (the 200-MW plant on the north), although the construction order may change. Each 100-MW site requires about 917 acres (or 1.4 square miles); the 200-MW site will be about 1,837 acres (or about 2.9 square miles). The total area required for all three phases, including the Administration/Operations and Maintenance building and substation, is approximately 3,671 acres (or about 5.7 square miles).

Impacts of the Project

Impacts of the project from the ten KOPs are summarized below.

We agree with Staff that with implementation of VIS-1 and VIS-2 visual impacts on views from KOPs 1 and 2 (views of the project from Primm Valley Golf Course) will be less than significant.

We do not agree with Staff that impacts on views from the I-5 corridor (views from KOPs 3, 4, and 5) would be significant. It is important to note that existing views across the project site from I-15 are not pristine in that this area is crossed by roads and a major electric transmission line, and that the Primm Valley Golf Course, which contrasts with the surrounding landscape is located within the foreground of views from a an approximately one mile stretch of the Interstate, and is visible in the middleground as travelers approach it from the east and west. It is true that the proposed solar power plant will be readily visible in views from I-15, but contrary to an assertion made in the FSA/DEIS, it will be seen in the middleground of the view, not the foreground. We agree with Staff that the Project would not obstruct views toward the Clark Mountains in the background because of the low height of the mirror fields and the relatively large distances between the vertical towers. We disagree with Staff's assertion that "glare" from the receiver units atop the solar towers would dominate or interfere with views from I-15 toward the Clark Mountains. In the FSA/DEIS Transportation analysis, the Staff testimony is that the brightness of the solar receiving units as seen from I-15 would be 38 cd/m^2 (see page 6.10-19). This level of brightness is equivalent to the brightness of a 100-watt light bulb seen at a distance of 35 meters (115 feet). This level of brightness does not fit the definition of glare, which properly speaking, refers to levels of brightness that cause discomfort or interfere with vision. In addition, it does not appear to be reasonable to assume that small points of light of this intensity seen at distances ranging from one to over 4 miles from the Interstate would dominate the views of the mountains or interfere with the views toward them. It is true that, although the project will be located in the middleground where it will be integrated into the larger landscape, the presence of the project in this view will represent an incremental change, increasing the intensity of human development in the corridor seen from the Interstate. However, we are in agreement with Staff's assessment that the proposed facility will:

> "…exhibit strong visual unity and simplicity, attributes that are generally associated with positive visual quality. This condition is in contrast to scenes of visual disorder and disunity that are generally equated with low visual quality or 'visual blight.' For example, a mining operation or manufacturing facility might present scenes of strong visual disorder and thus, low visual quality or 'blight.' The proposed project, in comparison, would exhibit moderate visual quality and would likely appear more acceptable than many other forms of intensive urban or industrial development. Thus, Staff notes that within an urban frame of reference, not all viewers would find the project disagreeable or unattractive; indeed, many viewers could find the project interesting to view due to its novelty. Overall, it would exhibit moderate visual quality and preserve scenic (though strongly altered) views."

Our assessment is that the overall impact on the views of travelers on I-15 will be less than significant. As Staff indicates, the level of sensitivity of views from I-15 is at most moderate. In addition, the time of viewer exposure is limited (only 4.8 minutes of elapsed time from the Nipton Road offramp to the Primm Valley Golf Club, when traveling at Interstate speeds), and there are no parking lots or vista point viewing areas in the area along this stretch of I-15 that permit travelers to stop to enjoy the scenery. Of that 4.8-minute view of the project, a background view toward the project is afforded for 2.2 minutes, and a middleground view is provided for the remaining 2.6 minutes. A foreground view of the project is not provided when driving on I-15 because the project sites are located more than 0.5 mile from I-15. In terms of impacts, views toward the Clark Mountains in the background will remain unobstructed, and the solar installation will appear as an orderly and attractive addition to the landscape. In addition, many viewers are likely to find the solar power plant to be a point of interest, with positive connotations as an expression of a concrete step toward energy independence and a shift toward production of energy in a way that is renewable and has low levels of overall environmental impact.

For the reasons stated above related to KOPs 3, 4, and 5, we disagree with Staff's assertion that the solar receivers atop the towers would create nuisance glare and interfere with views toward the mountains in the distance

We agree with Staff that the Project's visual impacts on views from KOP 6 (east side of Ivanpah Lake), KOP 7(west side of Ivanpah Lake), and KOP 8 (Whiskey Pete's in Primm, NV) would be less than significant.

Staff has found that with implementation of VIS-1, visual impacts on views from KOP 9 (views of the project from the road to Umberci Mine) and KOP 10 (views from the Benson Mine vicinity); would be significant and unavoidable. The conclusions that Staff has reached about the impacts on these views require some discussion. In characterizing the existing visual conditions and visual sensitivity of the views in these areas, Staff states that "…the existing intact natural landscape is considered one of the primary attractions for visitors to these mountains." However, Staff fails to point out that both locations include the sites of past mining activity, where there are roads, excavations, and derelict structures in the immediate foreground of the views that visitors experience, and that in fact, these remnants of the old mines may be part of what attracts visitors to these areas. Unfortunately, the Staff analysis in the FSA/DEIS does not place the views from KOPs 9 and 10 in their larger context. It provides no indication of the role of these particular views in the overall experience of the Stateline Wilderness and the Mojave National Preserve. It is reasonable to assume that the KOP 9 and 10 views represent views from just a portion of these areas, and that in most of these areas, the project area is either not visible due to topographic conditions, or is visible only in the distant background. More specifically, the view from KOP 9 may overstate the prominence of the project as it would appear from the Umberci Mine. This is because the KOP 9 photo was taken from the road to the mine and not from the mine location itself due to potential safety hazards (the presence of recreational gun shooting in that location, coupled with the high temperatures that were experienced in the area during the site visit). If the viewer experience is similar to that of those who attempted to take the photo, scenic views of the project site may not be possible from the mine, and are not likely to be a significant purpose of most visitors at this location. In addition, the views from these areas to the project site is already the view toward the Ivanpah Valley, which has a developed character in that it is traversed by a major Interstate highway, a railroad, a transmission line and gas line, and includes a large golf course and a complex of casinos. The legislation that established the Stateline Wilderness and the Mojave National Preserve contain no statements of purpose or specific provisions that provide for protection of the views from these areas toward the Ivanpah Valley below. We agree with Staff's assessment that, although the Project would change the views toward the Ivanpah Valley by adding a solar facility to the view, the Project would not obstruct the scenic views toward the valley.

Cumulative Impacts

The FSA/DEIS uses a much larger cumulative impact study area (e.g., the entire Mojave Desert and the California Desert Conservation Area, with 66 solar project applications and 63 wind project applications) than the Applicant (who considered the 5 projects planned to be developed in the project viewshed). It is the Applicant's position that it is improper to assess cumulative visual impacts outside of the project viewshed.

Compliance with Applicable LORS

The project would comply with applicable LORS. The FSA/DEIS asserts that the project would not comply with San Bernardino County Conservation Element Goal D/CO 1 and San Bernardino County Open Space Element Goal OS 5 and Policy OS 5.2. However, San Bernardino County Conservation Element Goal D/CO 1 calls for preservation of scenic vistas in the County; and the Ivanpah Valley (a BLM designated VRM Class III area) is not a scenic vista when the visual resources evaluation of the project was conducted by the Applicant.

A county-designated scenic route (a portion of I-15) exists in the project vicinity. The project would be visible against the mountain backdrop, but the view of the mountains to the west of I-15 will not be significantly degraded by the proposed project's presence, and the project will not detract from the view of the mountains. Therefore, the project is compatible with San Bernardino County Open Space Element Goal OS 5 and Policy OS 5.2.

Mitigation

With the imposition of the mitigation measures contained in Conditions of Certification VIS-1 through VIS-3 (as modified below) the project's visual impacts will be mitigated to a level that is less than significant.

III. Proposed Licensing Conditions

The FSA/DEIS for the project filed by the CEC and BLM recommends four Conditions of Certification be adopted to address visual resource issues. These conditions, VIS-1 to VIS-4, described on pages 6.12-43 to 6.12-46 of the FSA/DEIS, address surface treatments of project facilities, landscape screening, revegetation (details described in Condition of Certification BIO-14), and temporary and permanent exterior lighting.

In addition, the Visual Resources section of the FSA/DEIS mentions Conditions of Certification for visual resource-related impacts that are prescribed in detail in the Air Quality, Soil and Water, and Traffic and Transportation sections of the document (Conditions of Certification AQ-SC3, AQ-SC4, AQ-SC7, SOIL&WATER-1, TRANS-3, TRANS-4, and TRANS-5).

The Applicant has reviewed the Conditions of Certification (VIS-1 to VIS-4) set forth in the FSA/DEIS and find them acceptable in concept. We propose deleting VIS-3 (Revegetation of Disturbed Soil Areas) because it is simply a reference to BIO-14, which will be completed as part of the biological resource conditions. The Applicant proposes the following edits to the remaining three visual resource conditions for clarity, consistency, simplification, and to reduce redundancy. Comments on conditions AQ-SC3, AQ-SC4, AQ-SC7, SOIL&WATER-1, TRANS-3, TRANS-4, and TRANS-5 are provided in their respective discipline areas.

Proposed Revisions to VIS-1

As a general comment, the Applicant is opposed to conditions that require separate approvals of post-certification compliance activities by both BLM and the CPM because they are simply unworkable. If the approval is sequential, it will result in doubling the required approval time for everything. If the approval is concurrent, approvals may be potentially conflicting. As a general rule, consistent with current Commission practice, we have identified the Commission's CPM as the authority to review and approve post-certification compliance submissions or actions of the Applicant. It is also imperative that specific timeframes for approval be included in the Conditions so that the project will not be unnecessarily delayed. See the applicant's pre-hearing conference statement for a more-detailed explanation. In addition, changes proposed by the Applicant provide flexibility by moving requirements from the condition language to the verification section.

VIS-1: The project owner shall treat the surfaces of all project structures and buildings visible to the public, other than surfaces that are intended to direct or reflect sunlight, such that a) their colors minimize visual intrusion and contrast by blending with the existing tan and brown color of the surrounding landscape; and b) their colors and finishes do not create excessive glare~~; and c) their colors and finishes are consistent with local policies and ordinances~~. The transmission line conductors shall be nonspecular and non-reflective, and the insulators shall be non-reflective and non-refractive.

The project owner shall submit for CPM review and approval, a specific Surface Treatment Plan that will satisfy these requirements. ~~The treatment plan shall include:~~

~~A. A description of the overall rationale for the proposed surface treatment, including the selection of the proposed color(s) and finishes;~~

~~B. A list of each major project structure, building, tank, pipe, and wall; the transmission line towers and/or poles; and fencing, specifying the color(s) and finish proposed for each. Colors must be identified by vendor, name, and number; or according to a universal designation system;~~

~~C. One set of color brochures or color chips showing each proposed color and finish;~~

~~D. A specific schedule for completion of the treatment; and~~

~~E. A procedure to ensure proper treatment maintenance for the life of the project.~~

~~The project owner shall not specify to the vendors the treatment of any buildings or structures treated during manufacture, or perform the final treatment on any buildings or structures treated in the field, until the project owner receives notification of approval of the treatment plan by BLM's Authorized Officer and the CPM. Subsequent modifications to the treatment plan are prohibited without BLM's Authorized Officer and CPM approval.~~

Verification: At least 90 days prior to specifying to the vendor the colors and finishes for each set of ~~the first~~ structures or buildings that are surface treated during manufacture, the project owner shall submit the proposed treatment plan to ~~BLM's Authorized Officer and~~ the CPM for review and approval and simultaneously to San Bernardino County for review and comment. If ~~BLM's Authorized Officer and~~ the CPM, in consultation with BLM's Authorized Officer and the County, determine that the plan requires revision, the CPM shall request such revisions within thirty (30) days of receipt of such plan and the project owner shall provide to ~~BLM's Authorized Officer and~~ the CPM a plan with the specified revision(s) for review and approval by ~~BLM's Authorized Officer and~~ the CPM before any treatment is applied. The review of any subsequent revisions shall be completed by the CPM within fifteen (15) days of receipt of the revisions. Any modifications to the treatment plan must be submitted to ~~BLM's Authorized Officer and~~ the CPM for review and approval.

The treatment plan shall include:

A. A description of the overall rationale for the proposed surface treatment, including the selection of the proposed color(s) and finishes;

B. A list of each major project structure, building, tank, pipe, and wall; the transmission line towers and/or poles; specifying the color(s) and finish proposed for each. Colors must be identified by vendor, name, and number; or according to a universal designation system;

C. One set of color brochures or color chips showing each proposed color and finish;

D. A specific schedule for completion of the treatment;

E. Security fencing shall be standard low-reflectivity galvanized steel; and

F. A procedure to ensure proper treatment maintenance for the life of the project. The project owner shall not specify to the vendors the treatment of any buildings or structures treated during manufacture, or perform the final treatment on any buildings or structures treated in the field, until the project owner receives notification of approval of the treatment plan by the CPM.

Prior to the start of commercial operation, the project owner shall notify BLM's Authorized Officer and the CPM that surface treatment of all listed structures and buildings has been completed.~~ and they are ready for inspection and shall submit to each one set of electronic color photographs from the same key observation points identified in (d) above.~~ The project owner shall provide a status report regarding surface treatment maintenance in the Annual Compliance Report. The report shall specify a): the condition of the surfaces of all structures and buildings at the end of the reporting year; b) maintenance activities that occurred during the reporting year; and c) the schedule of maintenance activities for the next year.

Proposed Revisions to VIS-2

This condition assumes that the landscape screening is something that is desired by the Golf Club. If so, the Applicant will provide it, but the Golf Club needs to be responsible for maintaining the landscaping as part of its on-going maintenance of the overall golf courses. In addition, changes proposed by the Applicant provide flexibility by moving requirements from the condition language to the verification section.

VIS-2: ~~At the request of, and in consultation with BLM's Authorized Officer, the CPM and~~ If requested in writing by the golf course owner~~, the project owner~~ within ninety (90)- days of the effective date of this decision, the project owner shall prepare a perimeter landscape screening plan to reduce the visibility of the proposed ISEGS project as seen from the golf course. The purpose~~intent~~ of the plan shall be to provide screening of the power project, particularly the mirror fields from the tees and greens of the golf course, while retaining as much of the scenic portion of the overall views of Ivanpah Valley and Clark Mountains as feasible. The perimeter landscape screening plan ~~design approach~~ shall be developed in~~with prior~~ consultation with the golf course owner. The perimeter landscape screening plan shall be ~~, and~~ implemented by the Project Owner only at the golf course owner's written request. ~~The project owner shall submit to BLM's Authorized Officer and the CPM for review and approval and simultaneously to the golf course owner for review and comment a preliminary conceptual landscaping plan whose objective is to provide an attractive visual screen to views of the ISEGS project mirror fields. Upon approval by BLM's Authorized Officer and the CPM and golf course owner, the project owner shall submit to BLM's Authorized Officer and the CPM for review and approval and simultaneously to the golf course owner for review and comment a landscaping plan whose proper implementation will satisfy these requirements. The plan shall include:~~

~~A. A detailed landscape, grading, and irrigation plan, at a reasonable scale. The plan shall demonstrate how the requirements stated above shall be met. The plan shall provide a detailed installation schedule demonstrating installation of as much of the landscaping as early in the construction process as is feasible in coordination with project construction.~~

~~B. A list (prepared by a qualified professional arborist familiar with local growing conditions) of proposed species, specifying installation sizes, growth rates, expected time to maturity, expected size at five years and at maturity, spacing, number, availability, and a discussion of the suitability of the plants for the site conditions and mitigation objectives, with the objective of providing the widest possible range of species from which to choose;~~

~~C. Maintenance procedures, including any needed irrigation and a plan for routine annual or semi-annual debris removal for the life of the project;~~

~~D. A procedure for monitoring for and replacement of unsuccessful plantings for the life of the project; and~~

~~E. One set each for BLM's Authorized Officer and the CPM of 11"x17" color photosimulations of the proposed landscaping at five years and twenty years after planting, as viewed from adjoining segments of I-15.~~

The plan shall not be implemented unless the golf course owners requests implementation in writing and until the project owner receives ~~final~~approval from ~~BLM's Authorized Officer and~~ the CPM.

Verification: The landscaping plan shall be submitted to ~~BLM's Authorized Officer and~~ the CPM ~~for review and approval~~ and ~~simultaneously to~~ the golf course owner ~~for review and comment~~ at least ninety (90) days prior to installation of the landscaping. If ~~BLM's Authorized Officer and~~ the CPM or the golf course owner request ~~determine that~~ revision of the plan ~~requires revision~~, the project owner shall provide to ~~BLM's Authorized Officer and~~ the CPM and ~~simultaneously to~~ the golf course owner a revised plan~~ for review and approval by BLM's Authorized Officer and the CPM~~.

The plan shall include:

A. A detailed landscape, grading, and irrigation plan, at a reasonable scale. The plan shall demonstrate how the requirements stated above shall be met. The plan shall provide a detailed installation schedule demonstrating installation of as much of the landscaping as early in the construction process as is feasible in coordination with project construction.

B. A list (prepared by a qualified professional arborist familiar with local growing conditions) of proposed species, specifying installation sizes, growth rates, expected time to maturity, expected size at five years and at maturity, spacing, number, availability, and a discussion of the suitability of the plants for the site conditions and mitigation objectives, with the objective of providing the widest possible range of species from which to choose;

C. Maintenance procedures, to be implemented by the golf course owner, including any needed irrigation and a plan for routine annual or semi-annual debris removal for the life of the project;

D. A procedure for monitoring for and replacement of unsuccessful plantings for the life of the project; and

E. One set each for BLM's Authorized Officer and the CPM of 11"x17" color photosimulations of the proposed landscaping at five years and twenty years after planting, as viewed from adjoining segments of I-15.

The plan shall not be implemented until the project owner receives final approval from the CPM.

The planting must occur during the first optimal planting season following site mobilization or following review and approval of the plan by the CPM and the golf course owner. The project owner shall simultaneously notify ~~BLM's Authorized Officer and~~ the CPM and the golf course owner within seven days after completing installation of the landscaping.~~, that the landscaping is ready for inspection.~~

~~The project owner shall report landscape maintenance activities, including replacement of dead or dying vegetation, for the previous year of operation in each Annual Compliance Report.~~

Proposed Revisions to VIS-3

We propose deleting VIS-3 (Revegetation of Disturbed Soil Areas) because it is simply a reference to BIO-14, which will be completed as part of the biological resource conditions.

Proposed Revisions to VIS-4 (renumbered to VIS-3)

Changes proposed by the Applicant provide flexibility by moving requirements from the condition language to the verification section.

VIS-3~~4~~: To the extent feasible, consistent with safety and security considerations, the project owner shall design and install all permanent exterior lighting and all temporary construction lighting such that a) lamps and reflectors are not visible from beyond the project site~~, including any off-site security buffer areas~~; b) lighting does not cause excessive reflected glare; c) direct lighting does not illuminate the nighttime sky, except for required FAA aircraft safety lighting; and d) illumination of the project and its immediate vicinity is minimized~~, and e) the plan complies with local policies and ordinances~~. The project owner shall submit to ~~BLM's Authorized Officer and~~ the CPM for review and approval and simultaneously to the County of San Bernardino for review and comment a lighting mitigation plan. ~~that includes the following:~~

~~A. Location and direction of light fixtures shall take the lighting mitigation requirements into account;~~

~~B. Lighting design shall consider setbacks of project features from the site boundary to aid in satisfying the lighting mitigation requirements;~~

~~C. Lighting shall incorporate fixture hoods/shielding, with light directed downward or toward the area to be illuminated;~~

~~D. Light fixtures that are visible from beyond the project boundary shall have cutoff angles that are sufficient to prevent lamps and reflectors from being visible beyond the project boundary, except where necessary for security;~~

~~E. All lighting shall be of minimum necessary brightness consistent with operational safety and security; and~~

~~F. Lights in high illumination areas not occupied on a continuous basis (such as maintenance platforms) shall have (in addition to hoods) switches, timer switches, or motion detectors so that the lights operate only when the area is occupied.~~

Verification: At least ninety (90) days prior to ordering any permanent exterior lighting or temporary construction lighting, the project owner shall contact ~~BLM's Authorized Officer and~~ the CPM to discuss the documentation required in the lighting mitigation plan. At least sixety (60) days prior to ordering any permanent exterior lighting, the project owner shall submit to ~~BLM's Authorized Officer and~~ the CPM for review and approval and simultaneously to the County of San Bernardino for review and comment a lighting mitigation plan. If ~~BLM's Authorized Officer and~~ the CPM determines that the plan requires revision, the CPM shall notify the project owner within 30 days of receipt of the plan, and the project owner shall provide to ~~BLM's Authorized Officer and~~ the CPM a revised plan for review and approval ~~by BLM's Authorized Officer and the CPM~~.

The lighting plan shall include the following:

A. Location and direction of light fixtures shall take the lighting mitigation requirements into account;

B. Lighting design shall consider setbacks of project features from the site boundary to aid in satisfying the lighting mitigation requirements;

C. Lighting shall incorporate fixture hoods/shielding, with light directed downward or toward the area to be illuminated;

D. Light fixtures that are visible from beyond the project boundary shall have cutoff angles that are sufficient to prevent lamps and reflectors from being visible beyond the project boundary, except where necessary for security;

E. All lighting shall be of minimum necessary brightness consistent with operational safety and security; and

F. Lights in high illumination areas not occupied on a continuous basis (such as maintenance platforms) shall have (in addition to hoods) switches, timer switches, or motion detectors so that the lights operate only when the area is occupied.

The project owner shall not install~~order~~ any exterior lighting until receiving ~~BLM Authorized Officer and~~ CPM approval of the lighting mitigation plan.

Prior to commercial operation, the project owner shall notify ~~BLM's Authorized Officer and~~ the CPM that the lighting has been completed and is ready for inspection. If after inspection, ~~BLM's Authorized Officer and~~ the CPM notifies~~y~~ the project owner that modifications to the lighting are needed for the installed lighting to conform to the approved lighting mitigation plan, within thirty (30) days of receiving that notification, the project owner shall implement the modifications and notify ~~BLM's Authorized Officer and~~ the CPM that the modifications have been completed and are ready for re-inspection.

Within five (5) business days~~48 hours~~ of receiving a ~~lighting~~ complaint regarding lighting, the project owner shall provide ~~BLM's Authorized Officer and~~ the CPM with a complaint resolution form report, as specified in the Compliance General Conditions including either a response to the complaint or a proposal to resolve the complaint, and, if necessary, a schedule for implementation. If action is required to resolve the complaint, t~~T~~he project owner shall notify ~~BLM's Authorized Officer and~~ the CPM within 48 hours after the complaint is resolved.~~after completing implementation of the proposal.~~ A copy of the complaint resolution form report shall be submitted to ~~BLM's Authorized Officer and~~ the CPM within thirty (30) days of resolution of the complaint.

IV. Correlation to FSA and Hearing Topics:

- Visual Resources.

Waste Management

I. Introduction

A. **Name:** Sarah Madams

B. **Qualifications:** Ms. Madams' qualifications are as noted in her resume contained in Appendix A.

C. **Prior Filings:** In addition to the statements herein, this testimony includes by reference the following documents submitted in this proceeding:

- Application for Certification, Volumes 1 & 2 [Exhibit 1]
- Data Adequacy Supplement A [Exhibit 3]
- Comments to the PSA [Exhibit 57]
- Applicant's Response to CEC Staff Requests, Data Response Set 1A, dated January 14, 2008, Responses to Data Requests 111 through 116 [Exhibit 4].
- Applicant's Response to CEC Staff Requests, Data Response Set 1K, dated May 27, 2009, Responses to Data Request 111i [Exhibit 14].
- Applicant's Response to CEC Staff Requests, Data Response Set 1O, dated August 13, 2009, Responses to Data Request 111i [Exhibit 18].

To the best of my knowledge, all of the facts contained in this Section of the Applicant's testimony (including all referenced documents) are true and correct. To the extent this testimony contains opinions, such opinions are my own based upon my professional judgment. I make these statements, and render these opinions freely and under oath for the purpose of constituting sworn testimony in this proceeding.

II. Summary of Testimony

A. Affected Environment

The project site will be located in southern California's Mojave Desert, near the Nevada border, to the west of Ivanpah Dry Lake. The project will be located in San Bernardino County, California, on federal land managed by the Bureau of Land Management (BLM). The proposed project site is located in unincorporated San Bernardino County (Figure 2.1-1 of the AFC).

A Phase I Environmental Site Assessment (ESA) was conducted by CH2M HILL in accordance with the ASTM Standard E 1527-05, Standard Practice for Environmental Site Assessments. According to this report, the property is currently owned by the federal government and is managed by the BLM. The ESA report, dated August 2007, concluded that no past or present commercial or industrial activities have occurred at the site based on review of historical topographic maps, aerial photos, and a site reconnaissance.

The property is located in a remote desert location and has been undisturbed. No industrial or commercial activities are currently being performed onsite. The nearest land use, the Primm Valley Golf Club is located approximately 0.5 mile to the northeast of the Ivanpah 1 project site. The golf course has the potential to store minor quantities of hazardous materials. Due to the location of the golf course, however, no contamination from its activities are anticipated to impact the project site.

B. Construction Impacts

Both hazardous and non-hazardous waste will be generated during the construction and operating phases of the facility. During construction, the primary waste generated will be solid nonhazardous waste. Nonhazardous wastewater will be generated, including sanitary wastewater, equipment washwater, stormwater runoff, and wastewater from pressure testing the gas supply line. Most of the hazardous waste generated during construction will consist of liquid waste, such as flushing and cleaning fluids, passivating fluid (to prepare pipes for use), and solvents. Some hazardous solid waste, such as welding materials and dried paint, may also be generated. Small quantities of solvents, paints, and welding materials will also be generated. The construction contractor will be considered the generator of hazardous waste and will be responsible for proper handling of the waste in compliance with all applicable federal, state, and local laws and regulations including licensing, training of personnel, accumulation limits and times, and reporting and record keeping.

C. Operational Impacts

During Ivanpah SEGS facility operation, the primary waste generated will be nonhazardous solid waste. The majority of nonhazardous waste will be sanitary sewer sludge, from the small sewage treatment unit, that will be shipped offsite to landfill and water treatment filters (granular activated carbon [GAC] vessels, mixed bed vessels, and the de-ionization trailer from the onsite water treatment unit.

The Ivanpah SEGS facility will also produce maintenance and generating facility wastes, typical of power generation operations. These will include rags, broken and rusted metal and machine parts, defective or broken electrical materials, empty containers, the typical refuse generated by workers and small office operations, and other miscellaneous solid wastes.

General facility drainage will consist of plant raw water use such as area washdown, equipment leakage, and drainage from facility equipment areas. If cleaning chemicals are not used, water from these areas will be collected in a system of drains, hub drains, sumps, and piping and routed to the oil/water separator, and then to the waste collection tank. From there, the water will flow through a filter system and be sent back to the raw water storage tank for additional treatment prior to use at the facility. The sanitary wastewater collection treatment systems will collect sanitary wastewater from sinks, toilets, and other sanitary facilities, pass it through package treatment plants with the liquid waste being used for landscape irrigation.

Wastes that will be generated at the facility are summarized in Table 5.14-3 of the AFC. Hazardous waste generated at Ivanpah SEGS will be stored at that facility for less than 90 days. The hazardous waste will then be transported by a licensed hazardous waste transporter to a TSD facility.

For ultimate disposal, California has the three hazardous waste (Class I) landfills described below. The closest commercial hazardous waste disposal facility is the Clean Harbors' Buttonwillow Landfill in Kern County.

Clean Harbors' Buttonwillow Landfill in Kern County

This landfill is permitted at 14.3 million cubic yards and has approximately 9.2 million cubic yards of remaining space as of February 2006. At the current deposit rate, the landfill is permitted to accept waste until 2040. Buttonwillow has been permitted to accept all hazardous wastes except flammables, PCBs with a concentration greater than 50 parts per million, medical waste, explosives, and radioactive waste with radioactivity greater than 1,800 picocuries.

Clean Harbors' Westmorland Landfill in Imperial County

This facility is not currently open and accepting waste because the Buttonwillow facility can accommodate the current hazardous waste generation rate. The facility is, however, available in reserve and could be reopened if necessary. Even if opened, the landfill's conditional use permit prohibits the acceptance of some types of waste, including radioactive (except geothermal) waste, flammables, biological hazard waste (medical), PCB, dioxins, air- and water-reactive wastes, and strong oxidizers.

Waste Management's Kettleman Hills Landfill in Kings County

This facility accepts Class I and II waste. The facility has several landfill units, including the B-18 landfill unit. The B-18 Landfill is permitted for and will accept all hazardous wastes except radioactive, medical, and unexploded ordinance; this landfill has permitted capacity of 10 million cubic yards with a remaining capacity of approximately 2.6 million cubic yards as of June 2007. The life expectance remaining for Landfill B-18 is about 3 years; however, expansion of the facility is anticipated. Expansion of the facility would extend the closure date to 2036.

Additional Commercial Hazardous Waste Treatment and Recycling Facilities

In addition to hazardous waste landfills, there are numerous offsite commercial liquid hazardous waste treatment and recycling facilities in California. Some of the closest facilities include Safety Kleen Corp., Clean Harbors, Industrial Service Oil Co., Inc., and Pacific Resource Recovery Services in Los Angeles, Rho-Chem Corp. in Inglewood, Onyx Environmental in Azusa, Filter Recycling in Rialto, Advanced Environmental in Fontana, and Demenno Kerdoon in Compton.

D. Cumulative Impacts

The Ivanpah SEGS facility will generate nonhazardous solid waste that will add to the total waste generated in San Bernardino County, California and Clark County, Nevada. However, there is adequate recycling and landfill capacity in both California and Nevada to recycle and dispose of the waste generated by the Ivanpah SEGS project. It is estimated that the plant will generate approximately 280 tons of solid waste during construction and about 240 tons a year from operations (including approximately 4 tons of hazardous waste). Compared to the total amount of solid waste landfilled in San Bernardino County in the year 2006 of 1,862,461 tons and Clark County landfill capacity of 1,360,000,000 tons, the Ivanpah SEGS project's contribution will represent less than 1 percent of total county waste disposal (CIWMB, 2007 and Simpson, 2007). Therefore, the impact of the project on solid waste recycling and disposal capacity is not significant. The increased demand on solid waste recycling and disposal capacity by the Ivanpah SEGS would not result in significant cumulative waste management impacts.

Hazardous waste generated during operation of Ivanpah SEGS will consist of waste oil, filters, GAC units, mixed bed vessels, the de-ionization trailer, and fluids used to clean the boilers and steam turbines. The waste oil, GAC units, mixed bed vessels, and de-ionization trailers will be recycled or disposed of off site. Hazardous waste treatment and disposal capacity in California and Nevada is more than adequate. Therefore, the effect of Ivanpah SEGS on hazardous waste recycling, treatment, and disposal capability is not significant. The increased demand on hazardous waste recycling, treatment, and disposal capability by the Ivanpah SEGS would not result in significant cumulative waste management impacts.

E. Mitigation

The handling and management of waste generated by Ivanpah SEGS will follow the hierarchical approach of source reduction, recycling, treatment, and disposal. The first priority will be to reduce the quantity of waste generated through pollution prevention methods (e.g., high-efficiency cleaning methods). The next level of waste management will involve the reuse or recycle of wastes (e.g., used oil recycling). For wastes that cannot be recycled, treatment will be used, if possible, to make the waste non-hazardous (e.g., neutralization). Finally, offsite disposal will be used to dispose of residual wastes that cannot be reused, recycled, or treated.

Because the environmental impacts caused by wastes generated during construction and operation of the facility are expected to be insignificant, extensive monitoring programs will not be required. Generated waste, both nonhazardous and hazardous, will be monitored during project construction and operation in accordance with the monitoring and reporting requirements mandated by the regulatory permits to be obtained for construction and operation.

III. Proposed Licensing Conditions

The FSA/DEIS for the project filed by the CEC and BLM recommends that 7 Conditions of Certification be adopted to address hazardous materials management issues, WASTE-1 through WASTE-7. The Applicant suggests the following changes to these conditions.

Proposed Revisions to WASTE-1 through WASTE-7

As a general comment, the Applicant is opposed to conditions that require separate approvals of post-certification compliance activities by both BLM and the CPM because they are simply unworkable. If the approval is sequential, it will result in doubling the required approval time for everything. If the approval is concurrent, approvals may be potentially conflicting. As a general rule, consistent with current Commission practice, we have identified the Commission's CPM as the authority to review and approve post-certification compliance submissions or actions of the Applicant. It is also imperative that specific timeframes for approval be included in the Conditions so that the project will not be unnecessarily delayed. See the applicant's pre-hearing conference statement for a more-detailed explanation. This comment applies to all of the WASTE conditions. Those with other comments are presented below.

WASTE-3 The project owner shall prepare a Construction Waste Management Plan for all wastes generated during construction of the facility and shall submit the plan to ~~BLM's Authorized Officer and~~ the CPM for review and approval. The plan shall contain, at a minimum, the following:

- a description of all construction waste streams, including projections of frequency, amounts generated, and hazard classifications; and
- management methods to be used for each waste stream, including temporary on-site storage, housekeeping and best management practices to be employed, treatment methods and companies providing treatment services, waste testing methods to assure correct classification, methods of transportation, disposal requirements and sites, and recycling and waste minimization/source reduction plans.

Verification: The project owner shall submit the Construction Waste Management Plan to ~~BLM's Authorized Officer and~~ the CPM for approval no less than 30 days prior to the initiation of construction activities at the site. The CPM shall approve or identify any material deficiencies in the Construction Waste Management Plan within fifteen (15)~~30~~ days following receipt of the Plan.

WASTE-6 The project owner shall prepare an Operation Waste Management Plan for all wastes generated during operation of the facility and shall submit the plan to ~~BLM's Authorized Officer and~~ the CPM for review and approval. The plan shall contain, at a minimum, the following:

* * *

Verification: The project owner shall submit the Operation Waste Management Plan to ~~BLM's Authorized Officer and~~ the CPM for approval no less than 30 days prior to the start of project operation. The CPM shall approve or identify any material deficiencies in the Operation Waste Management Plan within ~~30~~15 days following receipt of the Plan. The project owner shall submit any required revisions to ~~BLM's Authorized Officer and~~ the CPM within 20 days of notification from ~~BLM's Authorized Officer and~~ the CPM that revisions are necessary.

* * *

WASTE-7

Verification: The project owner shall document all unauthorized releases and spills of hazardous substances, materials, or wastes that occur on the project property or related pipeline and transmission corridors. The documentation shall include, at a minimum, the following information: location of release; date and time of release; reason for release; volume released; amount of contaminated soil/material generated; how release was managed and material cleaned up; if the release was reported; to whom the release was reported; release corrective action and cleanup requirements imposed~~placed~~ by regulating agencies; level of cleanup achieved and actions taken to prevent a similar release or spill; and disposition of any hazardous wastes and/or contaminated soils and materials that may have been generated by the release. Copies of the unauthorized spill documentation shall be provided to BLM's Authorized Officer and the CPM within 30 days of the date the release was discovered.

IV. Correlation to FSA and Hearing Topics

- Waste Management.

Water Resources

I. Introduction

A. Names: Matthew Franck, Kathy Rose, Tim Durbin, Mark Kubik, and Tom Reagan

B. Qualifications: The panel's qualifications are as noted in their resumes contained in Appendix A.

C. Prior Filings: In addition to the statements herein, this testimony includes by reference the following documents submitted in this proceeding:

- Application for Certification, Volume 1 & Volume 2 [Exhibit 1]
- Data Adequacy Supplement A [Exhibit 2]
- Comments to the PSA [Exhibit 57]
- Applicant's Response to CEC Staff Requests, Data Response Set 1A, dated January 14, 2008, Responses to Data Requests 53 through 80 [Exhibit 4]
- Applicant's Response to CEC Staff Requests, Data Response Set 1B, dated February 11, 2008, Responses to Data Requests 53 through 55, 57 through 60, 63 and 68 [Exhibit 5].
- Applicant's Response to CEC Staff Requests, Data Response Set 1D, dated May 9, 2008, Responses to Data Requests 53 through 60, 63, 66 through 68, 75 and 76 [Exhibit 7].
- Applicant's Response to CEC Staff Requests, Data Response Set 1E, dated July 22, 2008, Responses to Data Requests 57 through 58 [Exhibit 8].
- Applicant's Response to CEC Staff Requests, Data Response Set 1N, dated August 5, 2009, Responses to Data Requests 64, 65 [Exhibit 17].
- Applicant's Response to CEC Staff Requests, Data Response Set 1P, dated September 9, 2009, Responses to Data Requests 64 [Exhibit 19].
- Applicant's Response to CEC Staff Requests, Data Response Set 2A, dated June 10, 2008, Responses to Data Requests 133 through 145 [Exhibit 20].
- Applicant's Response to CEC Staff Requests, Data Response Set 2B, dated July 22, 2008, Responses to Data Requests 137, 139, 140 and 145[Exhibit 21].
- Applicant's Response to CEC Staff Requests, Data Response Set 2C, dated August 6, 2008, Responses to Data Requests 140B, 140E and 145 [Exhibit 22].
- Applicant's Response to CEC Staff Requests, Data Response Set 2H, dated May 13, 2009, Responses to Data Requests 140 [Exhibit 27].

- Applicant's Response to CEC Staff Requests, Data Response Set 2I, dated May 18, 2009, Responses to Data Requests 139 [Exhibit 28]
- Applicant's Response to CEC Staff Requests, Data Response Set 2J, dated June 17, 2009, Responses to Data Requests 139 [Exhibit 29]
- Applicant's Response to CEC Staff Requests, Supplemental Data Response Set 1A, dated August 12, 2008, Responses to Data Requests S&W-1 thought S&W-4 [Exhibit 32].
- Applicant's Response to CEC Staff Requests, Supplemental Data Response Set 2B, dated May 13, 2009, Responses to Data Requests Appendix 5.15-A2 [Exhibit 39]
- Applicant's Response to CEC Staff Requests, Supplemental Data Response Set 2F, dated June 5, 2009, Responses to Data Requests S&W-5 [Exhibit 43].

To the best of our knowledge, all of the facts contained in this testimony (including all referenced documents) are true and correct. To the extent this testimony contains opinions, such opinions are our own. We make these statements, and render these opinions freely and under oath for the purpose of constituting sworn testimony in this proceeding.

II. Summary of Testimony

A. Affected Environment

The project site is located within the Ivanpah South portion of the Ivanpah Valley. Ivanpah South includes the 35-square-mile Ivanpah Lake, several ephemeral waterways, and scattered springs along the mountain front. Overall surface drainage in Ivanpah South is towards Ivanpah Lake. The Ivanpah Valley is underlain by the Ivanpah Groundwater Basin. Within the southern portion of the Ivanpah Groundwater Basin, groundwater flow is generally toward Ivanpah Lake and northward toward the Las Vegas Valley. Groundwater altitudes range from about 4,200 feet at the southern end of the Ivanpah Valley to less than 2,400 feet at the northern end of the valley. Local pumping in the valley has produced small but identifiable areas of groundwater decline.

B. Construction Impacts

Project grading during construction would prepare the site for the installation of the heliostats and contour the ground surface to allow for maintenance activities (e.g., washing the heliostats). Existing drainage patterns would be used to the extent possible, but alterations would be necessary in some areas to protect the power blocks and other equipment, the Low Impact Design. Although avoidance would be emphasized, ground disturbance and changes in local drainage patterns would result in an increase in water quality impacts due to erosion, and would require additional mitigation.

C. Operational Impacts

As described above, changes in drainage patterns would be avoided to the extent possible during construction of the solar field area. The project would require impervious surfaces for the power block, power tower, and related facilities, a total of 38.2 acres of impervious surfaces, or 1.14 percent of the project site. These impervious areas would alter natural drainage patterns, increasing the amount of runoff from the site. These changes have been accommodated by the site design, including the installation of diversions ditches and infiltration/evaporation areas.

Groundwater pumping is estimated at less than 100 acre-feet per year. All pumped water would be consumptively used and no groundwater return flows are expected. This is expected to result in minor groundwater level declines over time. Based on the detailed groundwater analyses conducted for the project, groundwater underflow from Ivanpah Valley to Las Vegas Valley is not expected to be measurably impacted by the project and the potential effects are thus less than significant.

Other potential operational impacts (e.g., degradation of water quality from operations or from the proposed septic system) would also be mitigated to a level of less than significant through adherence to standard regulatory processes.

D. Discussion Concerning Storm Water Runoff, Scour and Heliostat Stability Presented in Staff Testimony

Beginning on page 6.9-27 of the Soil and Water Resources section of the FSA/DEIS, a discussion is provided on the potential impacts of stormwater related scour on the project facilities. There are a number of errors in the data presented in that discussion. Table 10 provides a summary of a hydraulic analysis of the 100-year storm event performed by AECOM. The table includes a prediction of the potential number of heliostats that could fail due to storm related water scour. The table indicates that 13,889 heliostats are subject to failure in Ivanpah 1 and 2 and 18,172 heliostats are subject to failure in Ivanpah 3. In fact, each of these values represents AECOM's estimate of potential heliostat failures throughout the entire project site depending on the total number of heliostats that are constructed with the project. The lower number of potential failures (13,889) is based on the current design of a total of 214,000 installed heliostats while the higher number is associated with the maximum allowable total of 280,000 installed heliostats. This error also occurs on Table 9.

The last two rows in Table 10 imply that the Applicant provided estimates of the potential number of pylon failures due to scour. In fact, those estimates were not prepared by the Applicant, but were prepared by AECOM using the Applicant's hydrologic and hydraulic models. In any event, the estimates of heliostat failures appear to be based on a conceptual drawing that showed a preliminary estimate of the heliostat pylon embedment depth prior to any analysis by the Applicant of the potential scour. Since it is not provided in any of the referenced documents, Applicant has no way of knowing the basis for or even what insertion depth was used for the postulated heliostat failures in Table 10.

The discussion on page 6.9-28 indicates that 6 to 9 feet of scour can occur at the project site. This leads to a Staff recommendation in the proposed conditions of certification that all pylons be designed to withstand up to 6.5 feet of scour without failing. The methodology used to determine proposed design scour depth is not presented. Staff’s testimony provides no basis for, or detailed description of how it developed its peak flows and velocities. The referenced AECOM report does not provide data describing assumptions and/or sources of data that defend such a conclusion.

Including proposed insertion depth of the pylons Applicant believes the proposed design values are overly conservative for the following reasons:

a) The Federal Emergency Management Agency has developed equations for estimating the potential channel scour on alluvial fans. The Federal Highway Administration has developed equations for estimating local scour depths at piers. Using those equations produce total scour depths much less than 6.5 feet.

b) There are significant variations in the potential peak flows and velocities across the project site. Also, some portions of the project are located active areas of the alluvial fan where significant channel movement and erosion is expected to occur and some portions of the project are located in inactive areas of the alluvia fan where it is not. As a result, the potential scour depths will vary across the site and the design scour depths should be customized to reflect the unique and varying conditions on the ground.

To support our position the following documents are included in Attachment WR-1.

- Attachment WR-1A, FAN an Alluvial Fan Flooding Computer Program
- Attachment WR-1B, Evaluating Scout at Bridges
- Attachment WR-1C, Wash Data

Proposal for Modification of SOIL&WATER-5

The Applicant intends to embed the pylons to a depth sufficient to prevent any significant pylon failures during a 100-year storm event. That pylon insertion depth will be based on engineering and science processes as proposed in our modification to Condition of Certification **SOIL&WATER-5**. Applicant's proposed process is consistent with direction provided by BLM to utilize the San Bernardino Hydrology Manual for determination of inputs to the HEC-1 and Flo-2D computer based hydrology modeling programs. Applicant also proposes utilization of nationally recognized hydrologic methods for determination of erosion and local scour on alluvial fan formations as described above. Applicant believes that the appropriate process for determination of scour and pylon insertion requirements is through engineers and hydrologists for the Applicant and the CBO, as a delegate for the agencies, working together performing a scientific analysis on a site-specific basis.

As indicated above, the Applicant intends install the heliostat pylons to a depth sufficient to prevent failure of any pylons during a 100-year storm event. The Applicant's proposed process for determining the appropriate pylon embedment depth is presented in our proposed modification to Condition of Certification **SOIL&WATER-5.**

E. Cumulative Impacts

The project is unlikely to have impacts that would combine cumulatively with other closely related past, present, and reasonably foreseeable future projects such as the Las Vegas Valley Water District Pipeline, reoperation of the Molycorp Mine, Southern Nevada Supplemental Airport (Ivanpah Valley Airport), Desert Xpress Rail Line, improvements to Interstate 15, and Table Mountain Wind Generating Facility. Each of those projects would follow applicable regulations to mitigate short-term impacts (e.g., erosion during construction). The groundwater analyses conducted for the project considered potential adverse (and beneficial) effects associated with current and future groundwater pumping.

F. Mitigation

Mitigation for potential water quality impacts due to erosion is described in the testimony for Soils. Additional mitigation measures have not been proposed by the Applicant.

III. Proposed Licensing Conditions

The FSA/DEIS for the project proposes eight Conditions of Certification (Mitigation Measures) for Soil and Water Resources. Two of those conditions (SOIL&WATER-1 and SOIL&WATER-2) pertain to Soils and are discussed in the separate Soils testimony. The remaining six conditions (SOIL& WATER-3 through SOIL&WATER-8) pertain to Water Resources are discussed in this testimony. We agree with the Conditions of Certification set forth in the FSA/DEIS pertaining to Water Resources, except as set forth below.

Proposed Revisions to SOIL&WATER-3

As drafted, the Condition delegates the Commission's one-stop, in-lieu permitting authority to San Bernardino County. The Applicant is pleased to work with San Bernardino County for review and comment. However, for all state law issues, materials should be submitted to the CPM for "review and approval" and to other relevant non-federal governmental entities for "review and comment." SOIL&WATER-3 should be revised to eliminate approval authority granted to San Bernardino County.

SOIL&WATER-3: Pre-Well Installation. The project owner shall construct and operate up to two onsite groundwater wells that produce water from the IVGB. The project owner shall ensure that the wells are completed in accordance with all applicable state and local water well construction permits and requirements. Prior to initiation of well construction activities, the project owner shall submit for review and comment a well construction packet to the County of San Bernardino, in accordance with the County of San Bernardino Code Title 2, Division 3, Chapter 6, Article 5, containing the ~~all~~ documentation~~,~~ and plans,~~, and fees normally required for the county's well permit,~~ with copies to ~~both BLM's Authorized Officer and~~ the CPM. The project shall not construct a well or extract and use groundwater until ~~the County of San Bernardino provides a written concurrence that the proposed well construction and operation activities would comply with all applicable county well requirements, and both BLM's Authorized Officer and~~ the CPM provides approval to construct and operate the well. The County of San Bernardino may provide written comments within 20 days of the submission of the well construction packet. Regardless of whether the County provides written comments, the CPM will provide approval within 30 days of the submission of the well construction packet.

Post-Well Installation. The project owner shall provide documentation to ~~both BLM's Authorized Officer and~~ the CPM that the well has been properly completed. In accordance with California's Water Code section

13754, the driller of the well shall submit to the DWR a Well Completion Report for each well installed~~. The project owner shall ensure the Well Completion reports are submitted. The project owner shall ensure compliance with all county water well standards and requirements for the life of the wells and shall provide BLM's Authorized Officer and the CPM with two (2) copies each of all monitoring or other reports required for compliance with the County of San Bernardino water well standards and operation requirements, as well as any changes made to the operation of the well~~.

Verification: The project owner shall do all of the following:

1. No later than ~~sixty~~ thirty (~~60~~30) days prior to the construction of the onsite groundwater wells, the project owner shall submit to ~~both BLM's Authorized Officer and~~ the CPM a copy of the water well construction packet submitted to the County of San Bernardino for review and comment.

2. No later than ~~thirty~~ ten (~~30~~10) days prior to the construction of the onsite water supply wells, the project owner shall submit a copy of any written ~~concurrence~~ comments received from the County of San Bernardino ~~that the proposed well construction activities comply with all county well requirements and meet the requirements established by the county's water well permit program~~.

3. No later than sixty (60) days after installation of each well at the project site, the project owner shall ~~ensure that the well driller submits a~~ provide copies of the Well Completion Report submitted to the DWR ~~with a copy provided to both BLM's Authorized Officer and the CPM~~. The project owner shall submit to the CPM together with the Well Completion Report a copy of well drilling logs, water quality analyses, and any inspection reports.

4. ~~During well construction and for the operational life of the well, the project owner shall submit two (2) copies each to BLM's Authorized Officer and the CPM of any proposed well construction or operation permit changes within ten (10) days of submittal to or receipt from the County of San Bernardino~~.

5. No later than fifteen (15) days after completion of the onsite water supply wells, the project owner shall submit documentation to ~~BLM's Authorized Officer,~~ the CPM, ~~and the RWQCB~~ confirming that well drilling activities were conducted in compliance with Title 23, California Code of Regulations, Chapter 15, Discharges of Hazardous Wastes to Land, (23 CCR, sections 2510 et seq.) requirements and that any onsite drilling sumps used for project drilling activities were removed in compliance with 23 CCR section 2511(c).

Proposed Revisions to SOIL&WATER-4

There appears to be missing language within the text of SOIL&WATER-4 that prevents us from more fully commenting on this condition. We do not understand the first sentence under "A" regarding allowable groundwater use. Hence, we have added text to help clarify the condition.

SOIL&WATER-4: The project owner ~~proposes to~~ may construct and operate the project in phases, most likely beginning with Ivanpah 1, then Ivanpah 2, and ending with Ivanpah 3. The proposed project's use of groundwater during each year of construction shall not exceed more than the following:

~~A.~~ 200 AFY during the construction of one unit (either Ivanpah 1 or 2); and 250 AFY for all construction (when Ivanpah 3 or multiple units are under construction). ~~and~~ During operations, activities shall not exceed 100 acre-feet per year. Annual average water use shall be calculated using a 5-year rolling average of actual water use starting with the first year of operation. Prior to the use of groundwater for construction, the project owner shall install and maintain metering devices as part of the water supply and distribution system to document project water use and to monitor and record in gallons per day the total volume(s) of water supplied to the project from this water source. The metering devices shall be operational for the life of the project.

Verification: Beginning six (6) months after the start of construction, the project owner shall prepare a semi-annual summary of amount of water used for construction purposes. The summary shall include the monthly range and monthly average of daily water usage in gallons per day.

At least sixty (60) days prior to the start of construction of the proposed project, the project owner shall submit to ~~both BLM's Authorized Officer and~~ the CPM a copy of evidence that metering devices have been installed and are operational.

The project owner shall prepare an annual summary, which will include daily usage, monthly range and monthly average of daily water usage in gallons per day, and total water used on a monthly and annual basis in acre-feet. For years subsequent to the initial year of operation, the annual summary will also include the yearly range and yearly average water use by source. For calculating the total water use, the term "year" will correspond to the date established for the annual compliance report submittal.

Proposed Revisions to SOIL&WATER-5

The Applicant's reasons for the following changes are also presented in Section II.C above. The Applicant intends to embed the pylons to a depth sufficient to prevent any significant pylon failures during a 100-year storm event. That pylon insertion depth will be based on engineering and science processes to determination of scour and pylon insertion requirements. Engineers and hydrologists for the Applicant and the CBO, as a delegate for the agencies, working together performing a scientific analysis on a site-specific basis. Note that most of the changes simply move the specific requirements for the Storm Water Damage Monitoring and Response Plan from Condition language into the Verification portion of the COC. This allows the CPM to make minor implementation modifications and provides additional flexibility for what will be the first of its kind plan.

SOIL&WATER-5: The project owner shall ensure that all heliostats are designed to withstand storm water scour ~~of up to 6.5 feet or greater~~ as estimated by a Pylon Insertion Depth and Heliostat Stability Report to be completed by the applicant. For this report, project owner shall use equations that are federally recognized as the preferred methods for evaluating channel erosion and local scour on alluvial fans.

The project owner shall also develop a Storm Water Damage Monitoring and Response Plan to evaluate potential impacts from storm water, including heliostats that fail due to storm water flow or otherwise break and scatter mirror debris on to the ground surface. ~~The Storm Water Damage Monitoring and Response Plan shall include the following elements:~~

~~Detailed maps showing the installed location of all heliostats within each project phase.~~

~~Each heliostat should be identified by a unique ID number marked to show initial ground surface at its base, and the depth of the pylon below ground.~~

~~Minimum Depth Stability Threshold to be maintained of pylons to meet long-term stability for applicable wind, water and debris loading effects;~~

~~Above and below ground construction details of a typical installed heliostat.~~

~~BMPs to be employed to minimize the potential impact of broken mirrors to soil resources.~~

~~Methods and response time of mirror cleanup and measures that may be used to mitigate further impact to soil resources from broken mirror fragments.~~

~~Monitoring, documenting, and restoring the Ivanpah playa surface when impacted by sedimentation or broken mirror shards.~~

~~Monitor and Inspect Periodically, Before First Seasonal and After Every Storm Event:~~

~~Security and Tortoise Exclusion Fence: Inspect for damage and buildup of sediment or debris~~

~~Heliostats within Drainages or subject to drainage overflow: Inspect for tilting, mirror damage, depth of scour compared to pylon depth below ground and the Minimum Depth Stability Threshold, collapse, and downstream transport.~~

~~Drainage Channels: Inspect for substantial migration or changes in depth, and transport of broken glass.~~

~~Constructed Diversion Channels: Inspect for scour and structural integrity issues caused by erosion, and for sediment and debris buildup.~~

~~Ivanpah Playa Surface: Inspect for changes in the surface texture and quality from sediment buildup, erosion, or broken glass.~~

~~Short-Term Incident-Based Response:~~

~~Security and Tortoise Exclusion Fence: repair damage, and remove built-up of sediment and debris.~~

~~Heliostats: Remove broken glass, damaged structure, and wiring from the ground, and for pylons no longer meeting the Minimum Depth Stability Threshold, either replace/reinforce or remove the mirrors to avoid exposure for broken glass.~~

~~Drainage Channels: no short-term response necessary unless changes indicate risk to facility structures.~~

~~Constructed Diversion Channels: repair damage, maintain erosion control measures and remove built-up sediment and debris.~~

~~Long-Term Design-Based Response:~~

~~Propose operation/BMP modifications to address ongoing issues. Include proposed changes to monitoring and response procedures, frequency, or standards.~~

~~Replace/reinforce pylons no longer meeting the Minimum Depth Stability Threshold or remove the mirrors to avoid exposure for broken glass.~~

~~Propose design modifications to address ongoing issues. This may include construction of active storm water management diversion channels and/or detention ponds.~~

~~Inspection, short-term incident response, and long-term design-based response may include activities both inside and outside of the approved right-of-way. For activities outside of the approved right-of-way, the applicant will notify BLM and acquire environmental review and approval before field activities begin.~~

Verification: The basis for determination of total scour depth will be to employ the step-by-step process identified below with the following criteria:

A. Determination of Peak stormwater flow from a 100-year event:

- Use San Bernardino County (SBC) Hydrology Manual to specify hydrologic parameters to use in calculations
- Hydrologic parameters from SBC will be used to develop HEC-1 and Flo-2D hydrologic models

B. To Determine Potential Channel Erosion and Flow Velocity from peak storm water flow as determined in A above.

- Use Federal Emergency Management Agency (FEMA) equations a FEMA 1990 Report “FAN, An alluvial fan flooding computer program”

C. To Determine Potential Local Scour from peak storm water flow as determined in A above:

- Use Federal Highway Administration equation for local bridge pier scour in Evaluating Scour at Bridges, 2001.

Total scour at a pylon is the total of the results from equations applied in B and C above. To improve local accuracy, the project owner shall apply the engineering process above in Steps A through C in zones on the site to be defined as follows:

Zone 1: Ivanpah 1

Zone 2: Ivanpah 2

Zone 3 Ivanpah 3 South

Zone 4 Ivanpah 3 North

The Storm Water Damage Monitoring and Response Plan shall include the following elements:

- Detailed maps showing the installed location of all heliostats within each project phase.
- Each heliostat should be identified by a unique ID number marked to show initial ground surface at its base, and the depth of the pylon below ground.
- Minimum Depth Stability Threshold to be maintained of pylons to meet long-term stability for applicable wind, water and debris loading effects;
- Above and below ground construction details of a typical installed heliostat.
- Methods and response time of mirror cleanup and measures that may be used to mitigate further impact to soil resources from broken mirror fragments.
- Monitoring, documenting, and restoring the Ivanpah playa surface when impacted by sedimentation or broken mirror shards.

Monitor and Inspect Periodically, Before First Seasonal and After Every Major Storm Event:

- Security and Tortoise Exclusion Fence: Inspect for damage and buildup of sediment or debris
- Heliostats within Drainages or subject to drainage overflow: Inspect for pylon tilting, mirror damage, depth of scour compared to pylon depth below ground and the Minimum Depth Stability Threshold, collapse, and downstream transport.
- Drainage Channels: Inspect for substantial migration or changes in depth, and transport of broken glass.

- Constructed Diversion Channels: Inspect for scour and structural integrity issues caused by erosion, and for sediment and debris buildup.

Short-Term Incident-Based Response:

- Security and Tortoise Exclusion Fence: repair damage, and remove built-up of sediment and debris.
- Heliostats: Remove broken glass, damaged structure, and wiring from the ground, and for pylons no longer meeting the Minimum Depth Stability Threshold, either replace/reinforce or remove the mirrors to avoid exposure for broken glass.
- Drainage Channels: no short-term response necessary unless changes indicate risk to facility structures.
- Constructed Diversion Channels: repair damage, maintain erosion control measures and remove built-up sediment and debris.

Long-Term Design-Based Response:

- Propose operation/BMP modifications to address ongoing issues. Include proposed changes to monitoring and response procedures, frequency, or standards.
- Replace/reinforce pylons no longer meeting the Minimum Depth Stability Threshold or remove the mirrors to avoid exposure for broken glass.
- Propose design modifications to address ongoing issues. This may include construction of active storm water management diversion channels and/or detention ponds.

Inspection, short-term incident response, and long-term design-based response may include activities both inside and outside of the approved right-of-way. For activities outside of the approved right-of-way, the applicant will notify BLM and acquire environmental review and approval before field activities begin.

At least sixty (60) days prior to commercial operation, the project owner shall submit to ~~both BLM's Authorized Officer and~~ the CPM a copy of the Storm Water Damage Monitoring and Response Plan for review and approval prior to commercial operation. The project owner shall retain a copy of this plan onsite at the power plant at all times. The project owner shall prepare an annual summary of the number of heliostats failed, cause of the failure, and cleanup and mitigation performed for each failed heliostat.

Proposed Revisions to SOIL&WATER-6

As drafted, the Condition delegates the Commission's one-stop, in-lieu permitting authority to San Bernardino County. The Applicant is pleased to work with San Bernardino County for review and comment. However, for all state law issues, materials should be submitted to the CPM for "review and approval" and to other relevant non-federal governmental entities for "review and comment."

SOIL&WATER-6: The project owner shall submit a Groundwater Level Monitoring and Reporting Plan to the CPM for review and approval and to San Bernardino County for review and comment ~~both BLM's Authorized Officer and the CPM for review and approval~~ ~~in accordance~~ regarding consistency with the County of San Bernardino Code Title 2, Division 3, Chapter 6, Article 5 (Desert Groundwater Management Ordinance). The Groundwater Level Monitoring and Reporting Plan shall provide ~~detailed~~ a description of the methodology for monitoring background and site groundwater levels. Monitoring shall include pre-construction, construction, and project operation water use. The primary objective for the monitoring is to establish pre-construction and project related groundwater levels ~~trends~~ that can be quantitatively compared against observed and simulated ~~trends~~ levels near the project pumping well and near potentially impacted existing wells.

Prior to project construction, monitoring shall commence to establish pre-construction base-line conditions and shall incorporate the existing monitoring and reporting data collected for the Primm Valley Golf Club. The monitoring network shall be designed to incorporate the ongoing monitoring and reporting program established for the Primm Valley Golf Course. The monitoring plan and network may make use of existing wells in the basin that would satisfy the requirements for the monitoring program.

Verification: The project owner shall complete the following:

1. At least ~~six (6)~~three (3) months prior to construction, a Groundwater Level Monitoring and Reporting Plan shall be submitted to the County of San Bernardino for review and comment, and a copy of the County's comments and the plan shall be submitted to ~~both BLM's Authorized Officer and~~ the CPM for review and approval. The CPM shall provide approval no later than two (2) months after submission of the Plan. The ~~plan~~ Plan shall include a scaled map showing the site and vicinity, existing well locations, and proposed monitoring locations (both existing wells and new monitoring wells proposed for construction). The map shall also include relevant natural and man-made features (existing and proposed as part of this project). The plan also shall provide: (1) well construction information and borehole lithology for each existing well proposed for use as a monitoring well; (2) description of proposed drilling and well installation methods; (3) proposed monitoring well design; and, (4) schedule for completion of the work.

2. At least ~~four~~ two(~~4~~2) months prior to construction, a Well Monitoring Installation and Groundwater Level Network Report shall be submitted to ~~both BLM's Authorized Officer and~~ the CPM. The report shall include a scaled map showing the final monitoring well network. It shall document the drilling methods employed, provide individual well construction as-builds, borehole lithology recorded from the drill cuttings, well development, and well survey results. The well survey shall measure the location and elevation of the top of the well casing

and reference point for all water level measurements, and shall include the coordinate system and datum for the survey measurements. Additionally, the report shall describe the water level monitoring equipment employed in the wells and document their deployment and use.

3. As part of the monitoring well network development, all newly constructed monitoring wells shall be permitted and constructed consistent with San Bernardino County and State specifications.

4. At least ~~three~~ two (~~3~~2) months prior to project construction, all water level monitoring data shall be provided to ~~both BLM's Authorized Officer and~~ the CPM. The data transmittal shall include an assessment of pre-project water level ~~trend~~s, a summary of available climatic information (monthly average temperature and rainfall records from the nearest weather station), and a comparison and assessment of water level data relative to the assumptions and spatial ~~trends~~ levels simulated by the applicant's groundwater model.

5. After project construction and during project operations, the project owner shall submit the monitoring data annually to ~~both BLM's Authorized Office~~ and the CPM. The summary shall document water level monitoring methods, the water level data, water level plots, and a comparison between pre- and post-project start-up water level trends. The report shall also include a summary of actual water use conditions, monthly climatic information (temperature and rainfall), and a comparison and assessment of water level data relative to the assumptions and spatial ~~trends~~levels simulated by the applicant's groundwater model.

Proposed Revisions to SOIL&WATER-7

No changes are proposed for Soil&Water-7.

Proposed Revisions to SOIL&WATER-8

As drafted, the Condition delegates the Commission's one-stop, in-lieu permitting authority to San Bernardino County. The Applicant is pleased to work with San Bernardino County for review and comment. However, for all state law issues, materials should be submitted to the CPM for "review and approval" and to other relevant non-federal governmental entities for "review and comment."

SOIL&WATER-8: Sixty (60) days p~~P~~rior to the start of ~~construction~~commercial operations, the project owner shall ~~comply with the~~ submit to County of San Bernardino for review and comment and to the CPM for review and approval ~~Appendix B, C, and D requirements~~ plans for the construction and operation of the project's ~~proposed~~ sanitary waste septic system and leach field. The CPM will provide a final decision within thirty days of submitting the plans. ~~Project construction shall not proceed until documentation equivalent to the County's required wastewater treatment system permits are issued by the County and approved by both BLM's Authorized Officer and the CPM. The project owner shall remain in compliance with the County requirements for the life of the project.~~

Verification: Sixty (60) days prior to the start of commercial operations, t~~T~~he project owner will submit to the CPM for review and approval plans for the construction and operation of the project's sanitary waste septic system and leach field. The plan shall demonstrate compliance with ~~all necessary information and the appropriate fee to the County of San Bernardino to ensure that the project has complied with~~ the ~~County's and Appendix B, C, and D~~ sanitary waste disposal facilities requirements of San Bernardino County and Appendix B, C, and D~~. A written assessment prepared by the County of San Bernardino of the project's compliance with these requirements must be provided to the CPM sixty (60) days prior to the start of operation.~~. The CPM will consider timely comments from San Bernardino County and provide a final decision within thirty (30) days of submitting the plans.

Proposed Revisions Appendixes B and C

Appendix B and C cite "Attachment 2" and "Attachment 3" on numerous occasions; these appear to referring to Appendix C and Appendix D of the FSE/DEIS, and should be corrected. Other proposed revisions to these appendices are also provided below in order to correct inaccuracies and add clarification. The Applicant suggests the following changes to Appendix B and C.

APPENDIX B, FACTS FOR WASTEWATER DISCHARGE

Proposed Revision to item 9, Mitigation Plan (page 6.9-70)

The Applicant has proposed changes to BIO-20 in the Biological Resources testimony. This change make this item consistent.

> See Condition of Certification **~~Biology~~BIO-20** for a description of the ~~compensation~~ mitigation requirements for impacts to waters of the State.

Proposed Revisions to item 10.a. Storm Water Discharges (page 6.9-70), second paragraph, last sentence

Implementation of the SWPPP and DESCP will not "prevent water quality impacts." The BMPs are designed to avoid and/or minimize impacts to water quality. Hence, the last sentence should be revised to read:

> The Applicant will implement best management practices (BMPs) as described in the SWPPP and DESCP to avoid and/or minimize impacts to water quality ~~prevent water quality impacts~~ during construction.

APPENDIX C, REQUIREMENTS FOR WASTEWATER DISCHARGE

Proposed Revisions to Section II.C.2 (page 6.9-82)

Use of the term "at all times" is vague. Applicant proposes the following clarification.

> The applicant must, at all times during construction and operation of the project, maintain appropriate types and sufficient quantities of material on site to contain any spill or inadvertent release of materials that may cause a condition of pollution or nuisance if the materials reach waters of the State.

Proposed Revisions to Section III.A.3 (page 6.9-83)

This section requires immediate restoration of temporary disturbances and discharges of fill (i.e., soil) to waters of the State (i.e., washes). This language requires that immediately following completion of work in an area that restoration be implemented "to fully restore conditions to support all beneficial uses…" The Applicant is concerned that this requirement does not allow the project owner to wait until the optimum time to perform restoration activities (e.g., planting in the hot summer as opposed to waiting until the cooler fall). Also, it may be more efficient to aggregate impacts within a geographic area and wait until all disturbances to washes are complete before starting restoration activities. The Applicant also has concerns about the provision stating that revegetation with native species will occur in the shortest feasible time, since revegetation obviously takes time to become established before the beneficial use would be fully restored. It would be better if the language stated that revegetation efforts would proceed as described in the approved Revegetation and Rehabilitation Plan. Consequently, the Applicant proposes the following changes:

> Restoration of temporary disturbances and temporary discharges of fill to waters of the State must be ~~achieved immediately following completion of work in an area of the temporary impacts. Restoration must include implementing measures to fully restore conditions to support all beneficial uses for the water body temporarily impacted in the shortest feasible time. Restoration must include, but is not limited to, grading to pre-project contours and revegetation with native species.~~in accordance with the approved Closure, Revegetation and Rehabilitation Plan. The applicant must implement Best Management Practices (BMPs) to control erosion and runoff from areas associated with temporary fills.

Proposed Revisions to Section III.A.4 (page 6.9-84)

This mitigation measure is already addressed in BIO-20. Therefore, the following changes are proposed:

> Mitigation for 29.2 acres of permanent and long-term impacts ~~must be proposed prior to initiation of construction and approved by the BLM's Authorized Officer and CPM.~~shall be in accordance with condition of certification BIO-20, Streambed Impact Minimization and Compensation Measures

Proposed Revisions to Section III.B.14 (page 6.9-86)

The Applicants responsibility to maintain stormwater measures should apply until the site is decommissioned and restored according to the Closure, Revegetation and Rehabilitation Plan.

> The applicant must maintain, ~~in perpetuity,~~ during project operation and decommissioning post-construction control and treatment measures for storm water, or must identify in writing to the BLM's Authorized Officer and CPM, the entity that is legally responsible for maintaining the post-construction controls at the ISEGS project site.

ATTACHMENT A, GENERAL PROVISIONS FOR MONITORING AND REPORTING

Proposed Revisions to Section 1.b (page 6.9-93)

The Applicant suggests the following clarification language to this provision.

> All analyses (with the exception of in-field analyses for pH and turbidity) shall be performed in a laboratory certified to perform such analyses by the California Department of Public Health or a laboratory approved by the ~~BLM's Authorized Officer and~~ CPM. Specific methods of analysis must be identified on each laboratory report.

Proposed Revisions to Section 3.j (page 6.9-94)

This statement is vague as to what "every item" and "requirements" are. The Applicant assumes this section refers to when a stormwater sample exceeds pH or turbidity levels identified in the new Construction General Permit, but we are not certain. In addition, there are different parameters set for Risk Level 2 and Risk Level 3, so it is important to know which Risk Level applies. Please clarify.

IV. Correlation to FSA and Hearing Topics:

- Soil and Water Resources.

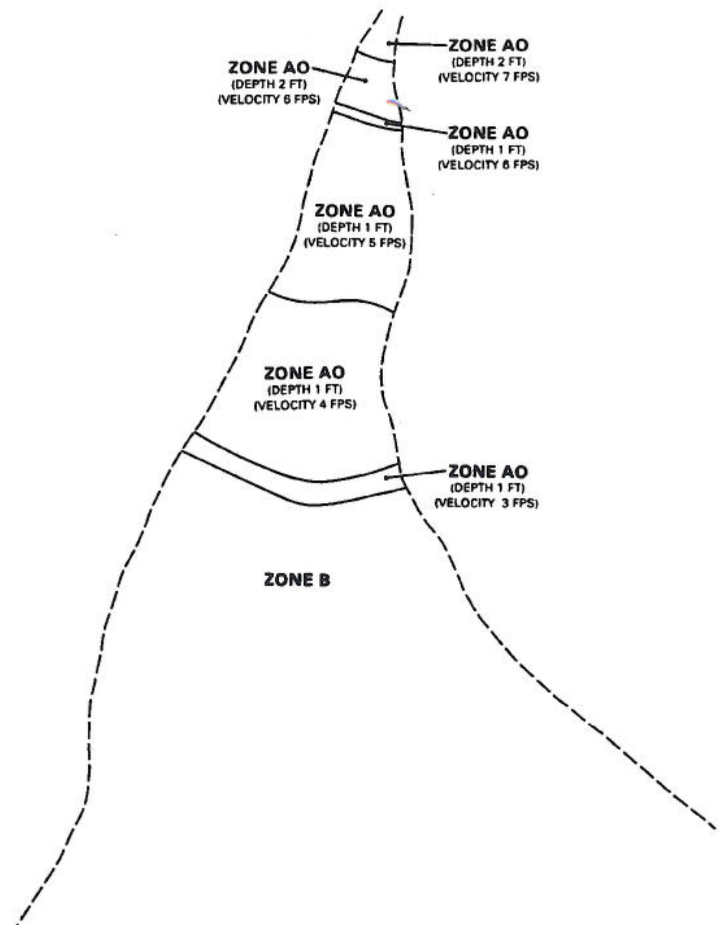
ATTACHMENT WR-1A

FAN an Alluvial Fan Flooding Computer Program

FAN

AN ALLUVIAL FAN FLOODING COMPUTER PROGRAM

USER'S MANUAL AND PROGRAM DISK



September 1990

Federal Emergency Management Agency
Federal Insurance Administration

FAN

An Alluvial Fan Flooding Computer Program

User's Manual and Program Disk

Prepared by

Risk Studies Division
Office of Risk Assessment
Federal Insurance Administration
Federal Emergency Management Agency

September 1990

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SECTION 1 — INTRODUCTION

1.1 BACKGROUND AND PURPOSE

In 1968, the National Flood Insurance Program (NFIP) was created as a multifaceted program for addressing the rising costs of flood damage. Since the passage of the Flood Disaster Protection Act of 1973, a major emphasis of the NFIP, which is administered by the Federal Emergency Management Agency (FEMA), has been identifying and mapping flood hazards in flood-prone communities nationwide. This identification and mapping effort has resulted in the evaluations of flood risks by detailed methods in more than 11,000 communities and evaluations by approximate methods for an additional 7,000 communities.

The flood risk data that were developed from these evaluations have been published on Flood Insurance Rate Maps (FIRMs) and in Flood Insurance Study (FIS) reports. These data provide the basis for flood insurance premium rates as well as for local floodplain management measures required for participation in the NFIP.

By 1979, FEMA recognized that standard procedures for evaluating riverine flood risks could not be used to evaluate flood risks attendant to alluvial fan flooding. Alluvial fan flooding is flooding that occurs on the surface of an alluvial fan or similar landform which originates at the "apex" and is characterized by high-velocity flows; active processes of erosion, sediment transport, and deposition; and unpredictable flow paths. Apex means a point on an alluvial fan or similar landform below which the flow path of the major stream that formed the fan becomes unpredictable and alluvial fan flooding can occur.

Some of the flood hazards associated with alluvial fan flooding are flash flooding, unpredictable flow paths, and a high velocity of flow coupled with the material of the landforms being highly susceptible to erosion. FEMA recognized these significant floods hazards and the high level of interest in development on alluvial fans, and adopted, for flood insurance purposes, a methodology for evaluating and mapping flood hazards on alluvial fans.

The computer program, FAN, discussed in this manual, was developed by FEMA to assist users in applying this methodology.

1.2 MANUAL FORMAT

This manual is comprised of six sections and two appendixes. The topics covered are described below.

- Section 1 Introduction
- Section 2 Derivation of the method for mapping special flood hazards on alluvial fans
- Section 3 Description of the FAN Program
- Section 4 Input requirements and output
- Section 5 FAN Program example runs
- Section 6 References
- Appendix A Supplement to Derivation
- Appendix B Listings

1.3 UNDERSTANDING THE METHODOLOGY

The FEMA methodology for determining flood hazards from alluvial fan flooding is simply the application of the definition for the 100-year flood. The application of the definition that is used in the FAN program is discussed in Section 2. The following analogy is presented to familiarize program users with such an application.

A small structure is to be built at a point on the surface of and 477 feet from, the center of a large cone. However, a man who lives at the peak of the cone makes building this structure a problem. The man has a collection of iron balls, ranging in diameter from 10 to 60 feet. Once a year, this man rolls a die and, depending on the outcome of the roll, chooses a ball from his collection. If he rolls a "1" he chooses a 10-foot-diameter ball; if he rolls a "2" he chooses a 20-foot-diameter ball; and so on. The man takes the chosen ball, places it at the peak of the cone, and lets it go. The ball rolls down the cone, taking an unpredictable path, and flattens anything in its way.
The structure could be built so that it will withstand the impact of an iron ball — but it is not practical to construct it to withstand the impact of a ball with a diameter of more than 20 feet. Therefore, the risk in any given year of being hit by a ball that has a diameter of more than 20 feet must be calculated. Two uncertainties are to be considered in this calculation:

1. The outcome of the roll of the die is unknown.

2. The path that the ball will follow is unknown.

Because the probability of a ball taking any one path is the same as the probability of it taking any other path, the probability of our structure being hit by the ball is equal to the diameter of the ball divided by the circumference of the cone at the point on which the structure is to be built. At the building site, the circumference of the cone is 3,000 feet. Thus, if the man rolled a "2," then the probability of the 20-foot-diameter ball hitting the structure is 20 ÷ 3,000 or 0.0067. The probability of a "2" being rolled is 0.1666.

To account for both uncertainties, the definition of conditional probability is applied. If $P(A)$ is the probability that event A will occur and $P(B)$ is the probability that event B will occur, then the probability that event A will occur, given that event B has occurred, is $P(AB) \div P(B)$ and is written $P(A|B)$. $P(AB)$ is the event of both A and B occurring together.

For the example, AB denotes the event that a certain number was rolled and the structure is hit by the corresponding ball. This is the event whose probability is to be computed. Thus, not knowing whether event B will occur, the probability that events A and B will occur can be calculated as the product $P(A|B)\ P(B) = P(AB)$. The probability of the structure being hit by a 20-foot-diameter ball can be calculated by multiplying the conditional probability given above by the probability that a "2" will be rolled. Therefore, the probability of the structure being hit by a 20-foot-diameter ball in any given year is 0.0067 ÷ 6 = 0.0011.

Before calculating the probability of the structure being demolished, several events that would result in its destruction should be noted. Specifically, the structure will be destroyed if hit by a 30-, 40-, 50-, or 60-foot-diameter ball. Using the nomenclature established above, the probability of event C or event D occurring is written P(CUD) = P(C) + P(D) − P(CD). In the example, C would denote the event that a certain number is rolled and the structure is hit by the ball (denoted AB above); D would denote the event that a different number is rolled and the structure is hit by the ball corresponding to that number. One roll of the die cannot result in two different numbers; therefore, when C includes the event of rolling one number and D includes the event of rolling another, the probability would be calculated as P(CD) = 0.

In that case, P(CUD) = P(C) + P(D). Therefore, the probability that the structure will be destroyed in any given year is the sum of the probabilities of it being hit by a 30-, 40-, 50-, or 60-foot-diameter ball:

$$P(\text{destruction}) = \sum_{k=3}^{6} P_k\,(\text{destruction}\,|D = 10k)\,P\,(D=10k)$$

where P_k is the probability of our structure being hit by a ball of diameter $D = 10k$ feet.

Thus,

$$P(\text{destruction}) = \sum_{k=3}^{6} \left(\frac{10k}{2\pi(477)}\right)\left(\frac{1}{6}\right)$$

$$= \frac{10}{5724\pi} \sum_{k=3}^{6} k$$

$$= \frac{180}{5724\pi}$$

$$= 0.01$$

A 30-foot-diameter ball could be called the 100-year ball. It is the ball with the diameter that is expected to be equaled or exceeded at the building site once in 100 years, on the long-term average. Because the probability of destruction at a given point depends on the circumference of the cone at that point, other locations on the cone will have 100-year balls of different diameters. If regions of the cone defined by their respective 100-year balls are to be mapped, the probability of destruction must be set equal to 0.01 and the equation for the circumference of the cone for each size of ball must be solved.

For example, if the 20-foot-diameter ball region is defined as that region bounded by the circles where the 100-year ball has a 20- or 30-foot-diameter, then it is the surface of the frustum that is bounded by circles of radii, r, and is computed as follows:

$$r = \frac{1}{0.01} \sum_{k=2}^{6} \left(\frac{10k}{2\pi}\right)\left(\frac{1}{6}\right) = 530 \text{ feet}$$

$$r = \frac{1}{0.01} \sum_{k=3}^{6} \left(\frac{10k}{2\pi}\right)\left(\frac{1}{6}\right) = 477 \text{ feet}$$

Thus, the map of the cone will show concentric circles separating the different regions defined by their respective 100-year balls.

If the man is replaced by Mother Nature, the die by a flood-frequency curve, the balls by the maximum peak discharge of the year, and the cone represents an area subject to alluvial fan flooding, a method for mapping special flood hazards on an alluvial fan can be derived. The approach is the same as that just described. However, the derivation is more complex and is presented in Section 2 and Appendix A.

SECTION 2 — DERIVATION

The FEMA methodology for determining flood hazards from alluvial fan flooding is simply the application of the definition of a 100-year flood. The location of the flow path during an alluvial fan flooding event is unpredictable. To determine the probability of a given point on the fan surface being flooded as a result of a storm over the watershed, the probability of the storm occurring and the probability that the flowpath of the floodwaters including that point must be considered. This section presents the derivation that is the basis for how the FAN program computes the magnitude of 100-year flood hazards in areas subject to alluvial fan flooding. If users are to recognize assumptions made in the program that are not consistent with the particular field conditions that are being analyzed and make the appropriate adjustments in their analyses, they must understand the derivation presented in this section and in Appendix A.

In the program, the assumptions made are: (1) The maximum peak discharge at the apex of the alluvial fan in any given year is independent of the peak discharge there in any other year; and (2) Those peak discharges are identically distributed from year to year. In short, the peak flows at the apex are independent and identically distributed. The floodpaths at a given elevation (i.e., on a given contour) are also assumed to be independent and identically distributed. Those assumptions lead to a definition of the 100-year flood discharge at a given point on the alluvial fan as the discharge that is expected to be exceeded at that point with a probability of 0.01 in any given year.

To illustrate the use of that definition, a simple problem that is somewhat analogous to that of defining flood hazards from alluvial fan flooding was presented in Section 1.3. This section presents the derivation of a method for mapping special flood hazards on the alluvial fan. Appendix A presents a supplement to the derivation.

2.1 BASIC APPROACH

Let H be a random variable denoting the occurrence of flooding at a given point subject to alluvial fan flooding. That is,

$$H = \begin{matrix} 1 \text{ if the point is flooded} \\ 0 \text{ if the point is not flooded} \end{matrix} \qquad (2.1)$$

Let Q be a random variable denoting the peak discharge resulting from a storm over the watershed. If f_Q is the probability density function (pdf) of Q, then the probability of the point being inundated by a flood with a peak discharge of at least q_0 cubic feet per second (cfs) is

$$P(H=1) = \int_{q_0}^{\infty} P_{H|Q}(1,q) f_Q(q)\, dq \qquad (2.2)$$

where $P_{H|Q}(1,q)$ is the probability of the point being flooded, given that the peak discharge is q cfs.

The 100-year flood discharge at a given point is defined as that discharge, q_{100}, for which the probability of the point being flooded by at least q_{100} cfs is 0.01 in any given year. That is, for each point subject to alluvial fan flooding, the 100-year flood discharge is that q_{100} which satisfies

$$0.01 = \int_{q_{100}}^{\infty} P_{H|Q}(1,q) f_Q(q)\, dq \tag{2.3}$$

Therefore, if the probability of a given point being flooded by a given discharge varies with its location, then so does the magnitude of the 100-year flood discharge.

If the 100-year flood discharge is to be quantified and the flood insurance zones are to be mapped, the functions in the integrand in Equation (2.2) must be defined. The FAN program defines the conditional probability, $P_{H|Q}(1,q)$, as the ratio of the channel width formed by q cfs, w(q), and the width of the area subject to alluvial fan flooding, W, at the elevation of the point of concern. That is,

$$P_{H|Q}(1, q) = \frac{w(q)}{W} \tag{2.4}$$

The width, W, is called the contour width.

When the contour width is much greater than the channel width [$W >> w(q)$], Equation (2.4) is equivalent to saying that each point on the contour has the same probability of being flooded. The definition of $P_{H|Q}(1,q)$ that is assumed in the FAN program depends on the function w(q) describing a relationship between channel width and peak discharge. This function can be derived from the assumptions outlined by Dawdy (Reference 1).

Consider a constant discharge, q, that creates a rectangular channel and flows at critical depth. Also, assume that this flow erodes the sides of the channel, resulting in an increase in channel width. Because the discharge is constant, an increase in channel width, w, must be accompanied by a decrease in depth, d, if the energy of a unit volume of water is to remain minimum. If this erosion continues until the change in width per change in depth equals -200, then the channel shape satisfies

$$\frac{dw}{dd} = -200 \tag{2.5}$$

where $\frac{d}{dd}$ denotes differentiation with respect to depth.

Using Equation (2.5) to define channel shape, the width, w(q), of a rectangular channel carrying a discharge q at critical velocity can be written

$$w(q) = 9.408\, q^{2/5} \tag{2.6}$$

The derivations of this equation are presented in Appendix A.

Thus, given that a storm will produce a peak discharge of q cfs, the program assumes that the probability of a point being inundated by that flood is

$$P_{H|Q}(1, q) = \frac{w(q)}{W}$$

$$= 9.408\, \frac{q^{2/5}}{W} \tag{2.7}$$

where W is the width of the area subject to alluvial fan flooding at the elevation of the contour on which the point lies — the contour width.

The pdf of Q, f_Q, must also be defined. The log-Pearson Type III distribution is used to define flood frequency in the program.

Let $Y = \log_{10} Q$. Thus, the probability that a storm producing a peak discharge of at least q_0 in any given year is

$$P(Q > q_0) = \int_{y_0}^{\infty} \frac{\lambda^k (y-m)^{k-1}}{\Gamma(k)}\, e^{-\lambda(y-m)}\, dy \tag{2.8}$$

where λ, k, and m are the three parameters of the Pearson Type III distribution, Γ(•) is the gamma function, and y is the base 10 logarithm of the discharge, q. Using y in the depth-discharge relationship assumed above yields

$$w(y) = 9.408(10)^{0.4y}$$

$$= 9.408\, e^{0.4\, y \ln 10}$$

$$= 9.408 e^{0.92y} \tag{2.9}$$

Using Equation (2.2), the pdf defined by the assumptions, the 100-year flood discharge at any point can be defined. It is that discharge, q_{100}, that has a base 10 logarithm such that $\log_{10} q_{100} = y_{100}$ and

$$0.01 = P(H=1) = \int_{y_{100}}^{\infty} 9.408 \frac{e^{0.92y}}{W} \frac{\lambda^k (y-m)^{k-1}}{\Gamma(k)} e^{-\lambda(y-m)} dy \tag{2.10}$$

where W is the contour width at the elevation of the point of interest. Rearranging Equation (2.10) yields

$$0.01 = \frac{9.408C}{W} \int_{y_{100}}^{\infty} \frac{(\lambda')^k (y-m)^{k-1}}{\Gamma(k)} e^{-\lambda'(y-m)} dy \tag{2.11}$$

where

$$C = \frac{e^{0.92m} \lambda^k}{(\lambda - 0.92)^k} \tag{2.12}$$

and

$$\lambda' = \lambda - 0.92 \tag{2.13}$$

A complete version of the derivation from Equation (2.11) to Equation (2.12) is presented in Appendix A.

Note that the integrand above is the Pearson Type III distribution assumed in Equation (2.8) for $\log_{10}Q$ with a change in the scaling parameter from λ to $\lambda' = \lambda - 0.92$. λ is referred to as the scaling parameter because changing it is equivalent to rescaling the random variable. For example, the integral in Equation (2.11) is the probability [defined by Equation (2.8)] that $10^{-0.92m/\lambda'} Q^{\lambda/\lambda'}$ exceeds q_{100} at the apex. Also, note that the C in Equation (2.12) may be undefined for values of λ between and including 0 and 0.92.

When the skew of the flood-frequency curve is 0, f_Q is log-normal. Therefore, instead of Equation (2.10), we have

$$0.01 = \int_{y_{100}}^{\infty} 9.408 \frac{e^{0.92y}}{W} \frac{e^{-\frac{1}{2}\left(\frac{y-\mu}{\sigma}\right)^2}}{\sigma\sqrt{2\pi}} dy \tag{2.14}$$

where μ is the mean of Y and σ is the standard deviation.

Equation (2.14) can be rearranged to

$$0.01 = \frac{9.408\,C}{W} \int_{y_{100}}^{\infty} \frac{e^{-\frac{1}{2}\left(\frac{y-\mu'}{\sigma}\right)^2}}{\sigma\sqrt{2\pi}}\,dy \tag{2.15}$$

where

$$C = e^{0.92\mu + 0.42\sigma^2} \tag{2.16}$$

and

$$\mu' = \mu + 0.92\sigma^2 \tag{2.17}$$

The derivation for Equation (2.15) is provided in Appendix A.

Note that the integrand above is the normal distribution describing the pdf of $\log_{10} Q$ with a change in the mean from μ to $\mu' = \mu + 0.92\sigma^2$. Again, the change is equivalent to rescaling the random variable. That is, the integral in Equation (2.15) is the probability that $10^{0.92\sigma^2} Q$ exceeds q_{100} at the apex.

Thus, by defining the flood-frequency distribution and the boundaries within which all possible flood paths lie, a 100-year flood can be defined for any point subject to alluvial fan flooding. That flood is defined by FEMA in terms of velocity, v, and energy depth, D. Energy depth is the specific energy above the bottom of the rectangular channel. The discharge associated with an energy depth of D feet is

$$q = 274.4\,D^{2.5} \tag{2.18}$$

Similarly, the discharge associated with a velocity, v, is

$$q = 0.1289v^5 \tag{2.19}$$

Thus, to find the contour on which, in any given year, each point has a 0.01 probability of being inundated by a flood whose energy depth exceeds 0.5 foot, Equation (2.11) [or Equation (2.15) if the skew is zero] should be solved for the contour width, W, with

$$y_{100} = \log_{10}[(274.4)(0.5)^{2.5}]$$

$$= \log_{10}(48.5) \tag{2.20}$$

That is,

$$W_{0.5} = \frac{9.408C}{0.01} P(Z > \log_{10} 48.5) \tag{2.21}$$

where C is given by Equation (2.12) [or (2.16) if the skew is zero] and $P(Z > \log_{10} q)$ represents the integral in Equation (2.11) [or (2.15) if the skew is zero]. The contour width corresponding to a 100-year energy depth of 1.5 feet is

$$W_{1.5} = 940.8\,C\,P(Z > \log_{10} 756) \tag{2.22}$$

The flood insurance zone for the area bounded by lines at the elevations where the contour widths are $W_{0.5}$ and $W_{1.5}$ is labeled as "ZONE AO, DEPTH 1". All contour widths corresponding to energy depths of the form $D = n + 0.5$, where n is an integer, and satisfying the condition

$$274.4\,D^{2.5} < Q_{100} \tag{2.23}$$

are computed, and the flood insurance zones are labeled accordingly. The upper limit defined by Condition (2.23) is simply a reiteration of the fact that, in any given year, the probability of being hit by a flood with a depth greater than that created by the 100-year flood discharge at the apex, Q_{100}, is less than or equal to 0.01. Condition (2.23) is discussed further in Subsection 2.3.

The flood insurance zones in areas subject to alluvial fan flooding are also labeled with the 100-year flood velocities. The velocity zone boundary widths are computed using a method similar to the one used to compute the depth zone boundary widths. For example, the width corresponding to a 100-year velocity of 3.5 fps is computed by first determining the discharge associated with that velocity

$$q=(0.1289)(3.5)^5=67.7 \qquad (2.24)$$

and then computing the width

$$W=940.8\,C\;P(Z>\log_{10}67.7) \qquad (2.25)$$

All contour widths corresponding to velocities of the form $v = n + 0.5$, where n is an integer, and satisfying the condition

$$48.5 < 0.1289v^5 < Q_{100} \qquad (2.26)$$

are computed, and the flood insurance zones are labeled accordingly. Setting the lower limit at 48.5 cfs is equivalent to saying that floods with energy depths of less than 0.5 foot do not create the special flood hazards associated with A zones on FIRMs.

2.2 AVULSIONS

During a flood, the flow may abandon one path and follow a new one. That occurrence, termed an avulsion, can result from floodwater overtopping a channel bank and creating a new channel. The overtopping may be caused by the sudden deposition of sediment and/or debris or by the undercutting and subsequent failure of a channel bank. Because points below the avulsion may be in the path taken by the floodflow either before or after the avulsion occurs, their probability of being hit by the flood is greater than if the avulsion had not occurred.

That increase in probability is accounted for by multiplying 1 plus the probability of an avulsion by the probability of being hit. Thus, if, during any flood, the probability of an avulsion is 0.5, the probability $P(H=1)$, would be multiplied by the factor 1.5, the avulsion factor. Including the notion of an avulsion factor in our previous discussion yields a probability that a point will be hit by a flood of a magnitude greater than q cfs in any given year of

$$P(H=1) = \frac{9.408AC}{W}\,P(Z>\log_{10}q) \qquad (2.27)$$

where A is the avulsion factor.

Accounting for the uncertainty of an avulsion by using a constant factor implies that the avulsion occurs during the peak of the flood and upfan of the point in question. Because we use the program to model the entire area subject to alluvial fan flooding, the latter implication is equivalent to saying that the avulsion occurs at the apex.

2.3 CORRECTION FOR HIGH FLOW VALUES

Without Condition (2.23),

$$274.4\, D^{2.5} < Q_{100} \tag{2.23}$$

contour widths that correspond to discharges greater than the 100-year flood discharge at the apex could be calculated. This is seen by considering the general form of Equation (2.21), including an avulsion factor

$$W = 940.8\, ACP(Z > \log_{10} q) \tag{2.28}$$

Note that when $P(Z > \log_{10} q)$ is greater than 0, W is greater than 0. Also note that $P(Z > \log_{10} q)$ is greater than 0 for all q when the skew of the flood-frequency curve is zero; for all $q > 10^m$ when the skew is greater than zero; and for all $q < 10^m$ when the skew is less than zero, where m is the Pearson Type III parameter in Equation (2.8).

Thus, for discharge values greater than the 100-year flood discharge at the apex, there are values for W that satisfy Equation (2.28). This implies that the probability of a point being hit by a flood of a magnitude greater than the 500-year flood discharge at the apex is 0.01 in any given year. The root of this contradiction is seen by reviewing the definition of the conditional probability given by Equation (2.4):

$$P_{H|Q}(1,q) = \frac{w(q)}{W} \tag{2.4}$$

For very large values of q and relatively small values of W, w (q) may be greater than W. In that case, the probability given by Equation (2.4) of a given point being hit by q cfs, given that that discharge is realized at the apex, is

$$P_{H|Q}(1,q) = \frac{w(q)}{W} > 1 \tag{2.29}$$

which is absurd.

To avoid small errors introduced by the contradiction in Equation (2.29), the calculation of the contour width must be adjusted. If q_W denotes the discharge that creates a channel as wide as the contour width, the probability given by Equation (2.11) can be corrected by replacing the upper limit of the integral with $\log_{10}q_W$ and adding the probability that q_W is exceeded at the apex. That is, find the contour width, W, such that

$$0.01 = \frac{9.408\,C}{W}\,P(\log_{10}q_W > Z > \log_{10}q_i) + P(Q > q_W)$$

$$= \frac{9.408\,C}{W}\left[P(Z > \log_{10}q_i) - P(Z > \log_{10}q_W)\right] + P(Q > q_W) \qquad (2.30)$$

where q_i is the discharge associated with the depth or velocity being investigated, $P(Z > \log_{10}q_i)$ is the integral in Equation (2.11), and $P(Q > q_W)$ is given by Equation (2.8).

Including an avulsion factor, A, gives the final expression of the problem to be solved. That is, for each q_i associated with the depths and velocities described above, the contour width, W, that satisfies the following equation is found:

$$0.01 = \frac{9.408\,AC}{W}\left[P(Z > \log_{10}q_i) - P(Z > \log_{10}q_W)\right] + P(Q > q_W) \qquad (2.31)$$

The W given by Equation (2.31) and the W given by Equation (2.28) differ by a small amount — negligible in most cases. For avulsion factors greater than 1.0, a solution of Equation (2.31) is not the exact solution to the problem. However, the error introduced by using avulsion factors greater than 1.0 is much smaller than the difference between widths given by Equations (2.28) and (2.31). It is mentioned here only as a matter of detail.

If the path taken by the floodflow is as wide as the alluvial fan at some elevation, then avulsions above that elevation are impossible. Thus, for widths between 9.408 $q_W^{2/5}$ and 18.816 $q_W^{2/5}$, the avulsion factor accounts for more risk than is present. Again, the more risk is negligible.

2.4 MULTIPLE CHANNELS

On many alluvial fans, floods do not remain within a single channel from the apex down to the toe. Instead, a flood may be carried by a single channel to some point down the fan and then by several channels below that point. The point at which the single channel becomes multiple channels is referred to as the bifurcation point. Analyses of several well-documented alluvial fan flooding events indicate that the cumulative width of the multiple channels is 3.8 times the width of the single channel above the bifurcation point (Reference 2). Therefore, in the multiple-channel region, we redefine the width-discharge relationship given by Equation (2.7) as

$$w(q) = (3.8)(9.408)q^{2/5} = 35.7504\,q^{2/5} \qquad (2.32)$$

Note that the multiple channels may be regarded as equivalent to a single channel with a width 3.8 times greater than the width of a single channel above the bifurcation point.

In addition, it is assumed that the depth and velocity of floodflows in the multiple-channel region can be estimated using Manning's equation with the friction slope set equal to the slope of the alluvial fan (i.e., a normal depth approximation). Thus, the relationship between depth of water, d, and discharge, q, is defined as

$$d = 0.0922\,n^{0.6}s^{-0.3}q^{0.36} \qquad (2.33)$$

where n is the roughness coefficient (Manning's "n") and s is the slope of the alluvial fan.

The velocity-discharge relationship can be written

$$v = 0.3033\,n^{-0.6}s^{0.3}q^{0.24} \qquad (2.34)$$

The velocity head can be computed from Equation (2.34):

$$\frac{v^2}{2g} = \frac{1}{2g}\left(0.3033\,n^{-0.6}s^{0.3}q^{0.24}\right)^2$$

$$= 0.00143\,n^{-1.2}s^{0.6}q^{0.48} \qquad (2.35)$$

Therefore, the energy depth, D, in the multiple-channel region is

$$D = d + \frac{v^2}{2g}$$

$$= 0.0922\,n^{0.6}s^{-0.3}q^{0.36} + 0.00143\,n^{-1.2}s^{0.6}q^{0.48} \qquad (2.36)$$

To find the discharges associated with the energy depths used to map the depth zone boundaries in the multiple-channel region, Equation (2.36) must be solved for $D = n + 0.5$, where n is an integer. Those discharges are bounded above by Q_{100}. Similarly, to determine the discharges associated with the velocities used to map the velocity zone boundaries in the multiple-channel region, Equation (2.34) must be solved for $v = n + 0.5$, where n is an integer. The discharge values given by Equation (2.34) are restricted to values less than Q_{100} and greater than the discharge that satisfies Equation (2.36) with $D = 0.5$. The restrictions on the discharges are analogous to Conditions (2.23) and (2.26) for the single-channel region.

The problem to be solved for the multiple-channel region is the same as that expressed by Equation (2.31) with the expression in brackets multiplied by 3.8. That is, for each q_i associated with the depths and velocities just described, the contour width, W, that satisfies

$$0.01 = \frac{35.7504\,AC}{W}\left[P(Z > \log_{10} q_i) - P(Z > \log_{10} q_W)\right] + P(Q > q_W) \qquad (2.37)$$

must be found. The FAN program computes the contour widths, corresponding to the depths and velocities of interest, that satisfy Equation (2.31) for the single-channel region [and Equation (2.37) for the multiple-channel region]. The values of cumulative distribution functions (CDFs), denoted P(•) in those equations, are determined by linear interpolation using the Pearson Type III tables published in Guidelines for Determining Flood Flow Frequency, Bulletin No. 17B (Reference 3). The tables in the program contain the number of standard deviations (k-values) that a given value of the random variable is from the mean. The tables in the program contain k-values corresponding to various values of the CDF for skew values varying by 0.1 and ranging from -4.1 to 4.1.

SECTION 3 — FAN PROGRAM DESCRIPTION

3.1 PROGRAM PURPOSE

The primary purpose of the FAN program is to compute the contour widths corresponding to the flood insurance zone boundaries (for example, the width of the area subject to alluvial fan flooding where the 100-year flood depth is 1.5 feet — the boundary between the 1- and 2-foot-depth zones). That purpose is accomplished by (1) using Equations (2.18) and (2.36) to determine the discharge (q) value that corresponds to each energy depth (D) value for the single- and multiple-channel regions, respectively; (2) using Equations (2.19) and (2.34) to determine the q value that corresponds to each velocity (v) value for the single-and multiple-channel regions, respectively; and (3) using Equations (2.31) and (2.37) to determine the contour width (W) value for the single- and multiple-channel regions, respectively.

Probabilities, denoted $P(\cdot)$ in Equations (2.31) and (2.37), are computed using the Pearson Type III tables published in Guidelines for Determining Flood Flow Frequency (Reference 2). To use those tables, the program must define the flood-frequency curve in terms of its mean (μ), standard deviation (σ), and skew coefficient (G). The tables consist of pairs of probabilities and numbers (k-values) of standard deviations between the corresponding value and the mean of the random variable for each skew value.

Given the statistics of the Pearson Type III distribution, the discharge, q_p, that has a probability of being exceeded in any given year is given by

$$\log_{10} q_p = \sigma\, k + \mu \tag{3.1}$$

Similarly, given a discharge, q, and the Pearson Type III statistics, the probability is determined by computing k from Equation (3.1) and finding the corresponding probability through linear interpolation.

Because the contour width depends on the probability that the rescaled (transformed) random variable exceeds the discharge in question [see Equation (2.11)], the mean, standard deviation, and skew coefficient of the rescaled random variable must be calculated to use the tables. For random variables that are Pearson Type III distributed, those statistics are related to the parameters of the distribution by

$$\mu = m + \frac{k}{\lambda} \tag{3.2}$$

$$\sigma = \frac{\sqrt{k}}{\lambda} \tag{3.3}$$

$$G = \frac{2}{\sqrt{k}} \tag{3.4}$$

Note that the skew coefficient, G, is independent of the scaling parameter, λ, and, therefore, is the same for both distributions. The mean and standard deviation of the rescaled random variable are found by replacing λ with $\lambda' = \lambda - 0.92$ in Equations (3.2) and (3.3), respectively.

3.2 PROGRAM STRUCTURE

Program flow is controlled by a DOS batch file, named FAN.BAT, that calls four programs written in BASIC (see Appendix B for listings). Compiled versions of those programs, PEARSN.EXE, FANINP.EXE, FANRUN.EXE, and AGAIN.EXE, reside in a directory named AL-FAN on the diskette that accompanies this manual. The order in which FAN calls those programs is shown in Figure 3-1.

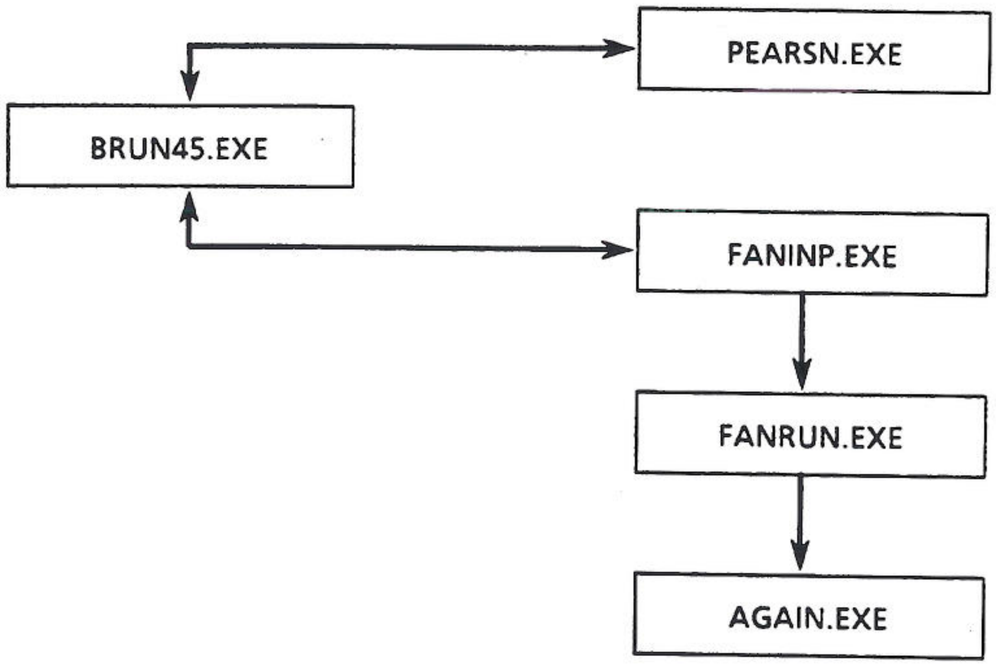


Figure 3-1. Flow of FAN.BAT

PEARSN.EXE assigns the k-values of the Pearson Type III tables. FANINP.EXE accepts the input data and defines the flood-frequency curve. These programs interact through BRUN45.EXE, a copyrighted program of Microsoft Corporation, 1982-88. After the flood-frequency curve has been defined, FANRUN.EXE computes the flood risk data. AGAIN.EXE, a short program, gives users the option to make another run.

3.3 PROGRAM FLOW

Figure 3-2 shows the flow of the FANRUN.BAS program. The flow of the contour width computations corresponding to the selected energy depth and selected velocity is shown in Figure 3-3 and 3-4, respectively. The numbers shown in each box are the line numbers of FANRUN.BAS that perform the given task. A listing of the program is given in Appendix B.

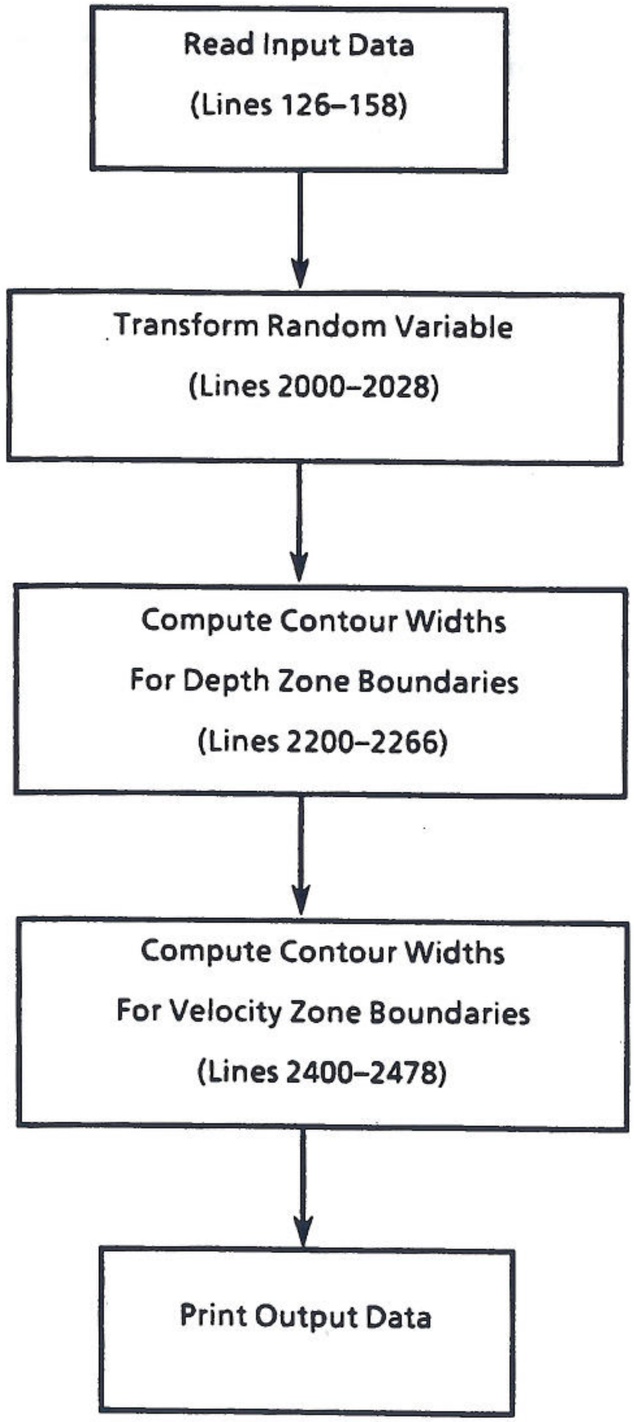


Figure 3—2. Flow of FANRUN.BAS Program

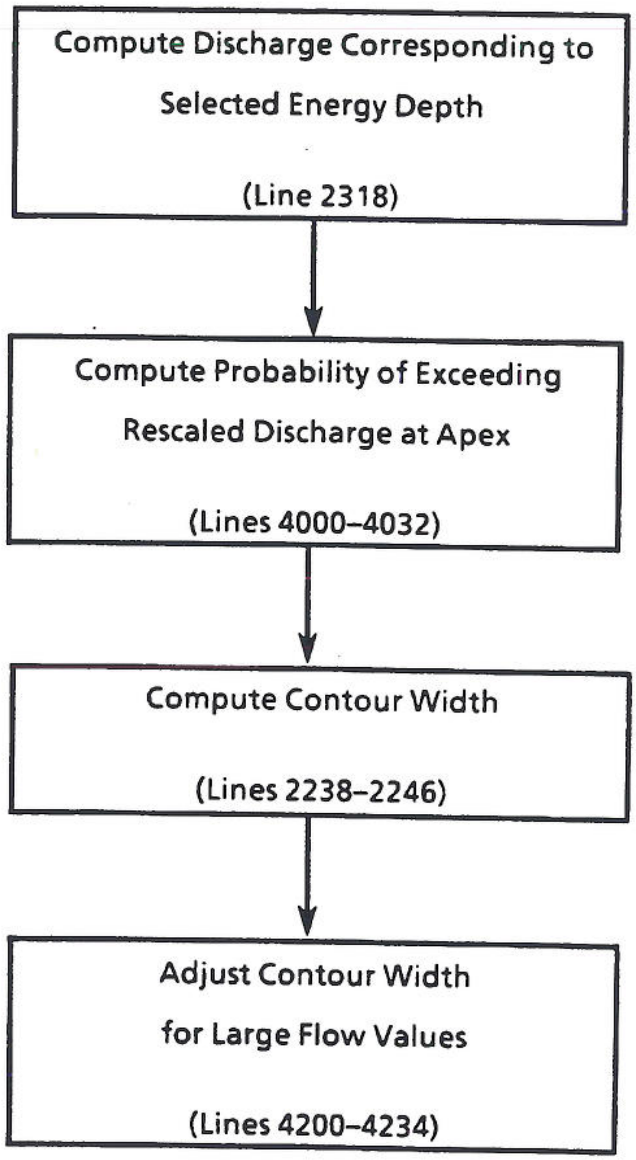


Figure 3—3. Flow of Contour Width Computations Corresponding to Selected Energy Depth

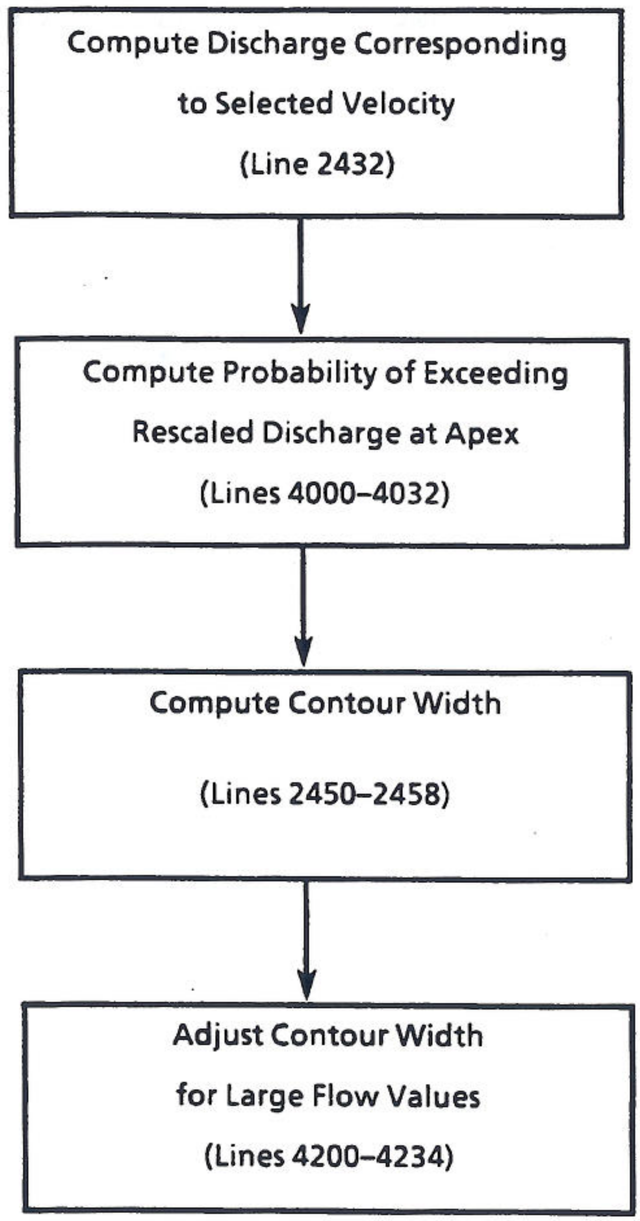


Figure 3—4. Flow of Contour Width Computations Corresponding to Selected Velocity

3.4 HARDWARE AND SOFTWARE REQUIREMENTS

The FAN program can be run on any IBM personal computer (PC) or any IBM-compatible PC that uses DOS release 2.0 or higher. The program, which is supplied on the 5-1/4-inch diskette that accompanies this manual, is compatible with fixed- and floppy-disk systems. This version of the program was written for single-user systems.

3.4.1 INSTALLATION

The program can be installed on another disk (hard or floppy) by inserting the enclosed diskette in Drive A and typing

A:INSTALL <*drive*>:

and pressing ENTER . The notation <*drive*> represents the letter of the drive. If Drive C is the hard disk in the system, the program can be installed on the hard disk by typing

A:INSTALL C:

If a properly formatted floppy diskette is inserted in Drive B, the program can be installed by typing

A:INSTALL B: ENTER

which copies the program onto that diskette.

The installation process copies the batch file FAN.BAT to the root directory of the disk in the specified drive and creates a directory, AL-FAN. The executable files associated with the program are copied into that new directory.

3.4.2 RUNNING THE PROGRAM

The program is run by typing **FAN** in response to the prompt for the root directory of the correct drive. If the prompt is for the correct drive but not the root directory, the user can access the root directory by entering **CD**.

For example, if the user inserts the diskette that accompanies this manual in Drive A and enters **A**, one of the following will appear on the screen:

A:\> or A:\AL-FAN>

The first prompt, A:\>, is for the root directory. If A:\> appears, entering **FAN** in response to that prompt will allow the user run the program. If A:\AL-FAN> appears, entering **CD** accesses the root directory.

The user is guided by prompts on the screen. The examples provided in Section 5 illustrate the interaction between the user and the program.

SECTION 4 — FAN PROGRAM INPUT AND OUTPUT

4.1 INPUT REQUIREMENTS

The input data required to run the FAN program are entered as responses to prompts. Data should be entered one value at a time. Values are entered by pressing **ENTER**. Responses to yes or no questions may be entered by pressing **Y** or **N** (capital or lower case), and then pressing **ENTER**.

The minimum input data required are flood-frequency data and the avulsion factor. If the user chooses to compute contour widths for the multiple-channel region, the alluvial fan slope and the roughness coefficient (Manning's "n") are also required.

4.1.1 Options for Entering Flood-Frequency Data

The program offers two options for entering flood-frequency data. Once the flood-frequency curve is defined, the program computes the 100-year flood discharge. If the 100-year flood discharge value is less than 50 (cfs) or greater than 500,000 cfs, the program will not run. If the 100-year flood discharge value is within that range, the program computes the product of the standard deviation and skew coefficient to see if the transformation constant, C, can be defined. [See Equation (2.12)] If that product is greater than 2.1, the program will not run.

4.1.1.1 Option 1 — Entering Statistics of the Distribution

Option 1 for defining the flood-frequency curve is to enter the mean, standard deviation, and skew coefficient. Standard deviations cannot be less than 0.1 and skew coefficients must be within the range of -4.1 to 4.1.

4.1.1.2 Option 2 — Entering Pairs of Recurrence Intervals and Discharge Values

Option 2 for defining the flood-frequency curve is to enter a minimum of three pairs of recurrence intervals and discharge values. The program finds the "best fit" of those data to a log-Pearson Type III distribution. The best fit is the log-Pearson Type III distribution that results in the maximum correlation coefficient of a least-squares fit of the data. Restrictions on the input data are as follows:

- Recurrence intervals must be between 1.001 and 1,000 years.
- Each recurrence interval may be entered only once (e.g., two 10-year flood discharge values cannot be entered).
- Discharge values must be greater than zero.
- Discharge values cannot decrease as the recurrence interval increases (e.g., if the 100-year flood discharge value is 1,000 cfs, the 90-year flood discharge value cannot be 2,000 cfs).

The least-squares fit is the straight line through a set of data pairs that minimizes the sum of the squares of the differences between the given values in the range of the data and the corresponding values predicted by the straight line. For each skew value tested, the program computes the k-values that correspond to the entered recurrence intervals and the base 10 logarithms of the entered discharges. The resulting pairs of data are then fit to the line defined by Equation (3.1), and the correlation coefficient is computed. The skew value that results in the greatest correlation coefficient is the skew of the flood-frequency curve. That skew value is found by treating the correlation coefficient as a function of skew with, at most, one critical point—its maximum. The mean and standard deviation of the flood-frequency curve are the slope and intercept, respectively, of the least-squares fit line for the chosen skew value.

4.1.2 Avulsion Factor

The avulsion factor may take any value. However, because the contour widths are proportional to the avulsion factor, using an avulsion factor of 0 will result in all contour widths being 0. Therefore, if the user enters a value of 0, the program will change it to 1.0.

4.1.3 Data Requirements for the Multiple-Channel Option

If the user chooses to compute contour widths for the multiple-channel region, additional data are required. Specifically, the user must supply the alluvial fan slope value and the roughness coefficients to be used in Manning's equation. The alluvial fan slope values (dimensionless) are restricted to a range of 0.000001 to 1.0; roughness coefficients are restricted to a range of 0.001 to 1.0. If the user enters a value outside those ranges, a message on the screen will advise the user that the input is too small or too large and instruct the user to re-enter the values.

4.2 OUTPUT

Output from a run consists of two or three pages, depending on the options chosen by the user. The output is written to a file named FAN.OUT. When a run is completed, the user is given three options; (1) to view the output on the screen, (2) to print the output, and (3) to make another run. When a new run is executed the file FAN.OUT is erased to make room for the new output. Thus, if the user wishes to save the output from a previous run, the option to print the output must be selected.

4.2.1 Flood-Frequency Data and Avulsion Factor

The first page of output data lists the avulsion factor and information pertaining to the flood-frequency curve. The flood-frequency data consists of the following:

- The option chosen to define the flood-frequency curve (If the option to enter pairs of flood-frequency data was selected, those data and the discharges corresponding to the entered recurrence intervals and defined by the least-squares fit of the data are listed.)

- The mean, standard deviation, and skew of the flood-frequency curve

- The 10-, 50-, 100-, and 500-year flood discharges (for use in the "Summary of Discharges" table in the FIS report)

- The scale change in (i.e., transformation of) the random variable denoting $\log_{10} Q$, the statistics of the distribution of the changed random variable, and the transformation constant (C)

4.2.2 Single-Channel Region Mapping Parameters

The second page of the output data lists the special flood hazard information for the flood insurance zone boundaries in the single-channel region. The information for the depth zones is given first; the information for the velocity zones is given second. That information is in tabular form and consists of the following:

- 100-year energy depths and velocities
- Depth of water associated with each energy depth or velocity
- Discharge associated with each energy depth or velocity
- Probability that that discharge is exceeded at the apex
- Probability that the "rescaled discharge" is exceeded at the apex
- Contour width associated with each energy depth or velocity

4.2.3 Multiple-Channel Region Mapping Parameters

The third page of the output data lists the special flood hazard information for the multiple-channel region, and consists of the following:

- Slope of the alluvial fan
- Roughness coefficient used in the energy depth and velocity computations
- 100-year energy depths and velocities
- Depth of water associated with each energy depth or velocity
- Discharge associated with each energy depth or velocity
- Probability that that discharge is exceeded at the apex
- Probability that the "rescaled discharge" is exceeded at the apex
- Contour width associated with each energy depth or velocity

SECTION 5 — FAN PROGRAM EXAMPLE RUNS

5.1 INTRODUCTION

The example runs in this section illustrate the interaction between the user and the FAN program. The alluvial fan used in the examples is shown in Figure 5–1. The flood-frequency curve for these examples is log-normal with a standard deviation of 1.0 and a 2-year flood discharge of 10 cfs. Therefore, the distribution has a mean of 1.0 and a skew of 0. The flood-frequency curve is shown in Figure 5–2. The slope of the alluvial fan is 0.085 and the roughness coefficient is 0.05. The avulsion factor is 1.0.

Two examples have been selected to demonstrate the various options. In Example Number 1 (Subsection 5.2), the mean, standard deviation, and skew coefficient are used to define the flood-frequency curve; the option of computing contour widths for the multiple-channel region is not chosen. In Example Number 2 (Subsection 5.3), flood-frequency data are entered in the form of pairs of recurrence interval and discharge values. The option of computing contour widths for the multiple-channel region is chosen. In the examples, user-supplied information is denoted by bold print.

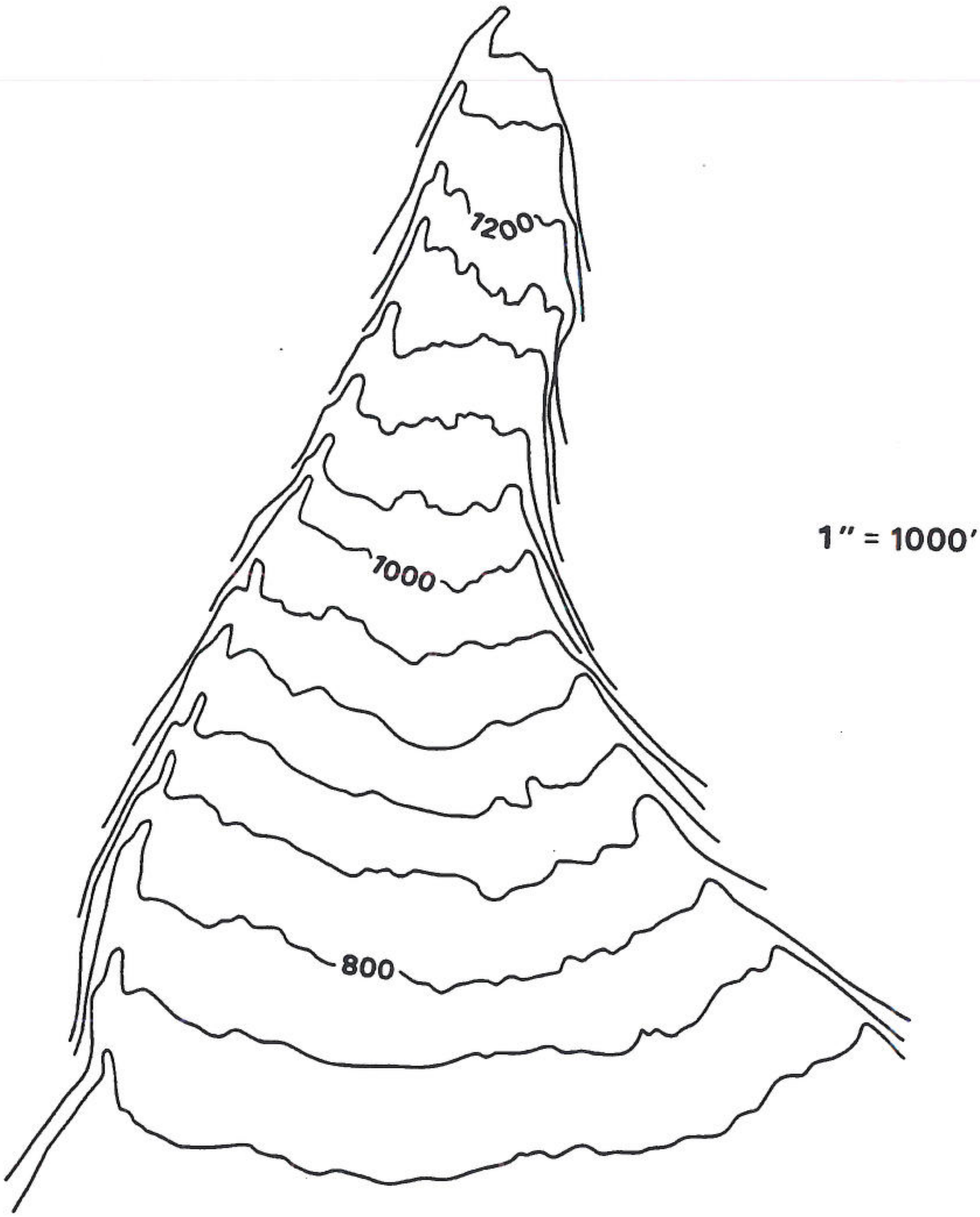


Figure 5—1. Example Alluvial Fan

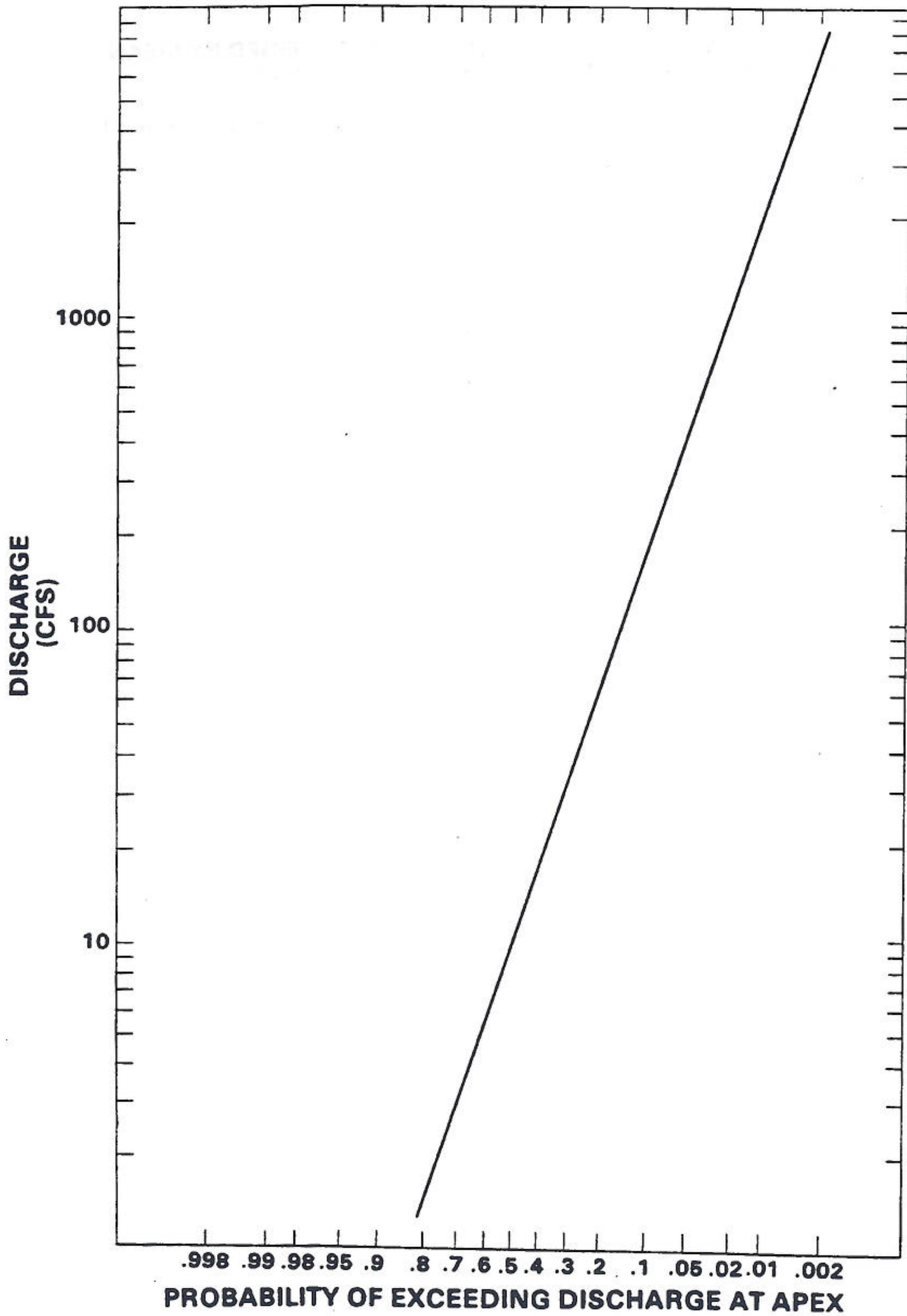


Figure 5—2. Example Flood-Frequency Curve

5.2 EXAMPLE 1 — FLOOD-FREQUENCY CURVE DEFINED BY MEAN, STANDARD DEVIATION, AND SKEW COEFFICIENT

When the user types **FAN**, the batch file is called, the program begins to run, and the following message appears on the screen:

> ALLUVIAL FAN FLOODING COMPUTER PROGRAM
>
> In response to prompts, enter data one value at a time, then press enter.
>
> Answer yes-or-no questions with the corresponding letters (i.e., Y or N).
>
> PEARSON TYPE-III TABLES BEING LOADED.......

While that message is on the screen, the program assigns the k-values of the Pearson Type III distributions. That process takes a few seconds to complete. When the k-values have been assigned, the message changes.

> ALLUVIAL FAN FLOODING COMPUTER PROGRAM
>
> In response to prompts, enter data one value at a time, then press enter.
>
> Answer yes-or-no questions with the corresponding letters (i.e., Y or N).
>
> PRESS ENTER TO PROCEED.................. ENTER

If the user presses ENTER , the screen is cleared and the first prompt appears. The user is asked to enter the name of the alluvial fan. Therefore, **Example Number 1** is entered.

```
Press F1 and then press ENTER to exit

            ENTER THE NAME OF THE ALLUVIAL FAN

            EXAMPLE NUMBER 1        ENTER
```

The next prompt concerns the multiple-channel option. In the first example, contour widths will be computed for the single-channel region only. Therefore, the answer to the prompt is "N."

```
Press F1 and then press ENTER to exit

DO YOU WISH TO COMPUTE ZONE BOUNDARIES

FOR MULTIPLE CHANNELS (Y/N)? N        ENTER
```

Next, the user is asked to enter the avulsion factor. An avulsion factor of 1.0 is entered.

```
Press F1 and then press ENTER to exit

ENTER AVULSION FACTOR 1        [ENTER]
```

The next prompt concerns the option for defining the flood-frequency curve. The flood-frequency curve is to be defined by statistics; therefore, Option 1 is selected.

```
Press F1 and then press ENTER to exit

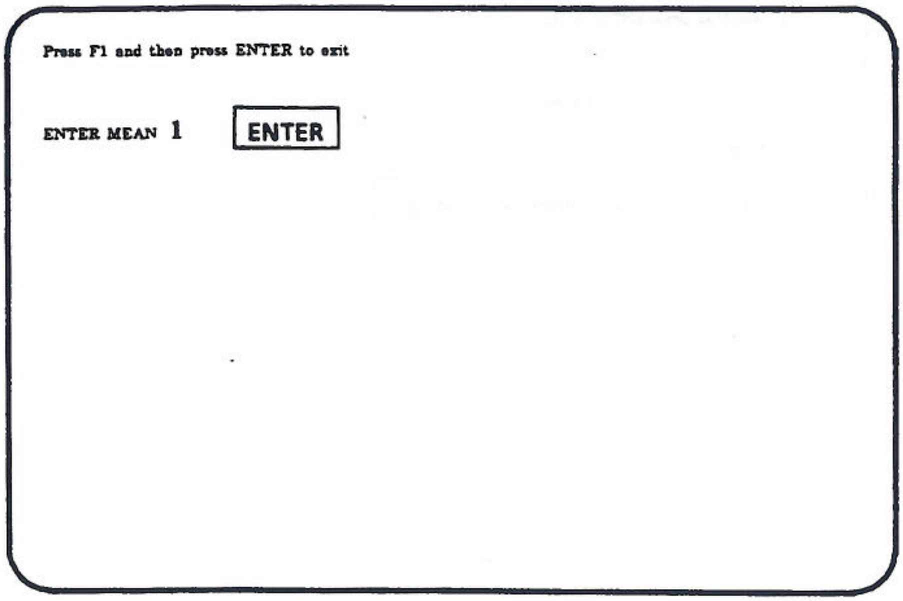
YOU MAY DEFINE THE FLOOD FREQUENCY CURVE BY:

(1)....ENTERING THE MEAN, STANDARD DEVIATION, AND SKEW COEFFICIENT
       OF THE PEARSON TYPE-III DISTRIBUTION

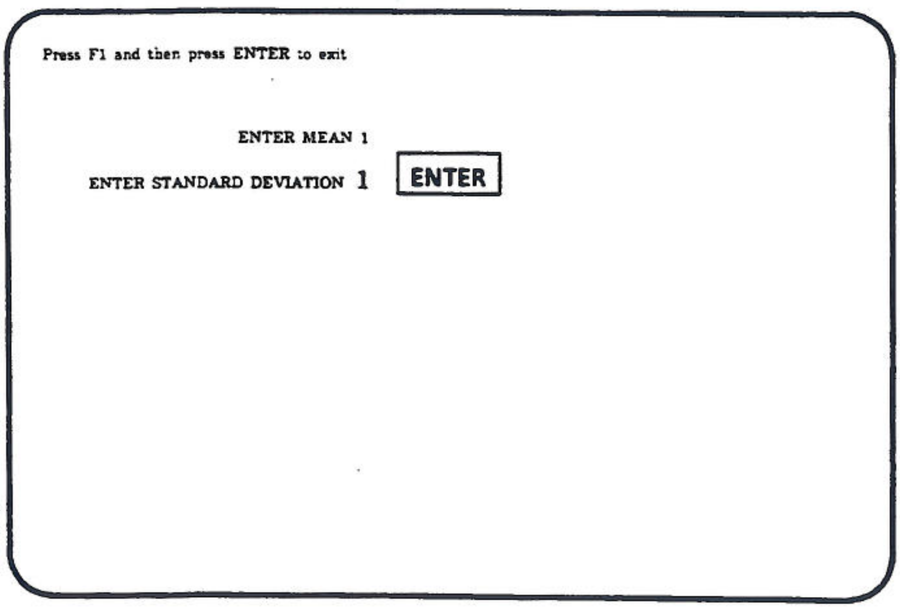
(2)....ENTERING (AT LEAST THREE) PAIRS OF RETURN INTERVALS AND DISCHARGES

PLEASE ENTER OPTION NUMBER (1 OR 2) 1        [ENTER]
```

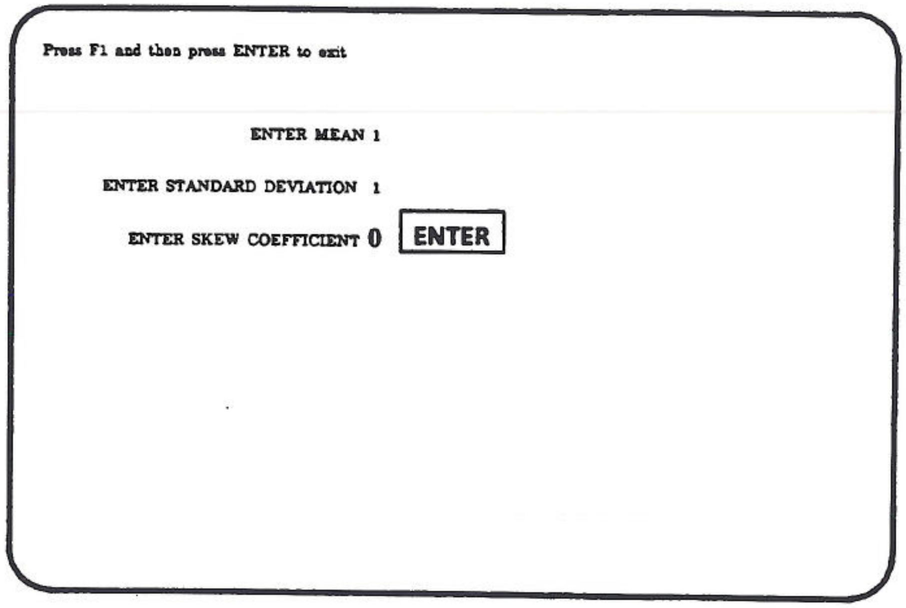
After choosing Option 1, three statistics are to be entered, one value at a time, beginning with the mean.



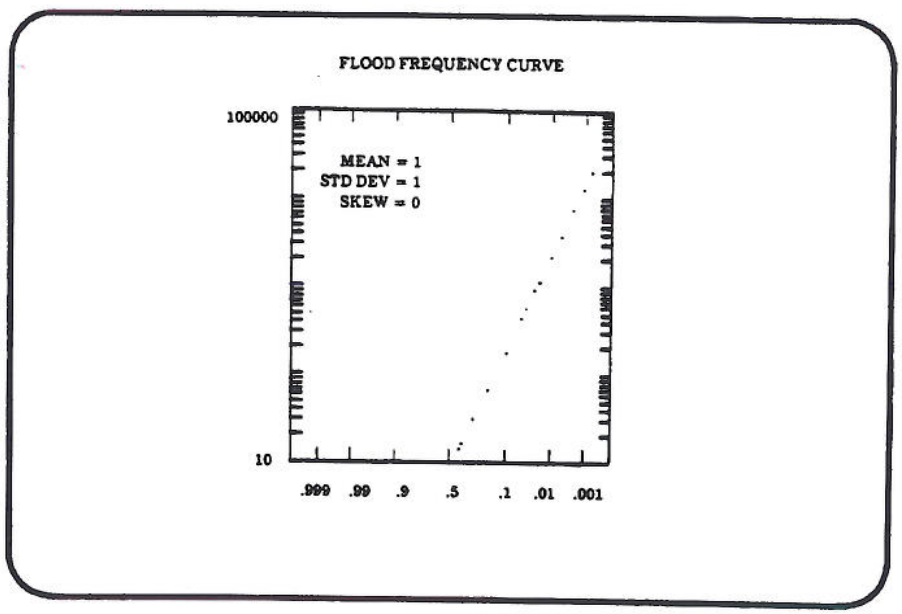
The mean remains on the screen, and the user is prompted to enter the standard deviation.



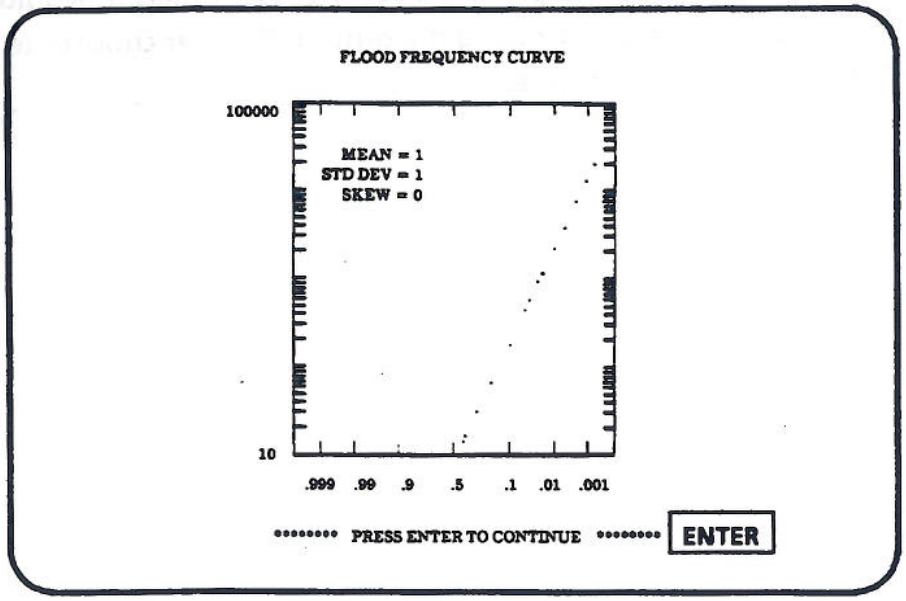
The mean and standard deviation remain on the screen, and the user is prompted to enter the skew coefficient.



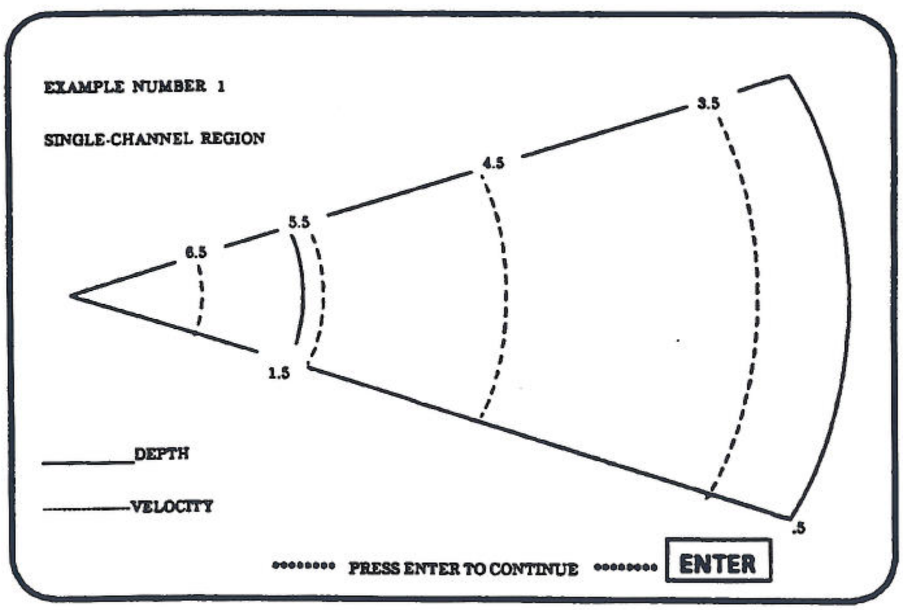
Entering the skew coefficient completes the data entry requirements. The screen is cleared and the flood-frequency curve is drawn.



When the computations of the contour widths are complete, the message "PRESS ENTER TO CONTINUE" appears at the bottom of the screen.



Pressing ENTER , clears the screen, and a picture of an alluvial fan (with simple boundaries) is drawn. The depth (solid lines) and velocity (dashed lines) zone boundaries with the computed contour widths for the single-channel region are drawn on the alluvial fan. When the picture is complete, the "PRESS ENTER TO CONTINUE" message appears again.



When ENTER is pressed, a prompt asks the user if he/she would like to view the output. Instead of viewing the output, the user chooses to print it out. Therefore, an "N" is entered.

```
DO YOU WISH TO VIEW SOME OUTPUT (Y/N)? N    ENTER
```

In response to the next prompt, "Y" is entered to activate the printing option.

```
DO YOU WISH TO PRINT THE OUTPUT (Y/N)? Y    ENTER
```

As a result of this response, the output file is sent to the printer. A final prompt gives the user the option to make another run.

```
DO YOU WISH TO MAKE ANOTHER RUN (Y/N)? Y    [ENTER]
```

By responding "Yes" to the prompt, the program returns the user to the beginning.

5.3 EXAMPLE 2 — FLOOD-FREQUENCY CURVE DEFINED BY PAIRS OF RECURRENCE INTERVALS AND DISCHARGES

ALLUVIAL FAN FLOODING COMPUTER PROGRAM

In response to prompts, enter data one value at a time, then press enter.

Answer yes-or-no questions with the corresponding letters (i.e., Y or N).

PEARSON TYPE-III TABLES BEING LOADED.......

Again, the program assigns the k-values of the Pearson Type III distributions, and the message changes.

ALLUVIAL FAN FLOODING COMPUTER PROGRAM

In response to prompts, enter data one value at a time, then press enter.

Answer yes-or-no questions with the corresponding letters (i.e., Y or N).

PRESS ENTER TO PROCEED.................. ENTER

The same procedure is followed as for the first example run. Therefore, the first entry is the alluvial fan name.

```
Press F1 and then press ENTER to exit

                ENTER THE NAME OF THE ALLUVIAL FAN

                EXAMPLE NUMBER 2      [ENTER]
```

For this example, the multiple-channel option is to be used. Therefore, the answer to the next prompt is "Yes."

```
Press F1 and then press ENTER to exit

DO YOU WISH TO COMPUTE ZONE BOUNDARIES

FOR MULTIPLE CHANNELS (Y/N)? Y      [ENTER]
```

Having selected the multiple-channel option, the user is first asked to enter the slope of the alluvial fan.

```
Press F1 and then press ENTER to exit

ENTER SLOPE OF ALLUVIAL FAN  .085        [ENTER]
```

The slope is displayed, and the user is prompted to enter the n-value.

```
Press F1 and then press ENTER to exit

SLOPE = .085

ENTER ROUGHNESS COEFFICIENT (N-VALUE) .05        [ENTER]
```

The slope and n-value are displayed, and the user is asked to enter the avulsion factor.

```
Press F1 and then press ENTER to exit

MULTIPLE CHANNEL PARAMETERS:
                              SLOPE = .085
                            N-VALUE = .05

ENTER AVULSION FACTOR 1       [ENTER]
```

The user now chooses the option for determining the flood-frequency curve. For this example, Option 2, least-squares fit of data, is selected.

```
Press F1 and then press ENTER to exit

YOU MAY DEFINE THE FLOOD FREQUENCY CURVE BY:

(1).... ENTERING THE MEAN, STANDARD DEVIATION, AND SKEW COEFFICIENT
        OF THE PEARSON TYPE-III DISTRIBUTION

(2).... ENTERING (AT LEAST THREE) PAIRS OF RECURRENCE INTERVALS AND DISCHARGES

PLEASE ENTER OPTION NUMBER (1 OR 2) 2       [ENTER]
```

The first prompt in Option 2 asks for the number of pairs of data to be entered.

```
Press F1 and then press ENTER to exit

HOW MANY PAIRS OF DISCHARGES AND
RECURRENCE INTERVALS DO YOU WISH TO ENTER? 6        ENTER
```

Next, the user is asked to enter the first recurrence interval.

```
Press F1 and then press ENTER to exit

HOW MANY PAIRS OF DISCHARGES AND
RECURRENCE INTERVALS DO YOU WISH TO ENTER? 6

ENTER RECURRENCE INTERVAL NUMBER 1  2        ENTER
```

The user is then asked to enter the corresponding discharge.

```
Press F1 and then press ENTER to exit

HOW MANY PAIRS OF DISCHARGES AND
RECURRENCE INTERVALS DO YOU WISH TO ENTER? 6

ENTER RECURRENCE INTERVAL NUMBER 1      2

     ENTER 2 - YEAR DISCHARGE                              10 [ENTER]
```

After the discharge is entered, the screen is cleared and the first pair of data is displayed; the user is prompted to enter the next recurrence interval.

```
Press F1 and then press ENTER to exit

        DATA PAIR                RECURRENCE INTERVAL     DISCHARGE
            1                             2                  10


ENTER RECURRENCE INTERVAL NUMBER 2        5 [ENTER]
```

Again, the user is asked to enter the corresponding discharge.

Press F1 and then press ENTER to exit

DATA PAIR	RECURRENCE INTERVAL	DISCHARGE
1	2	10

ENTER RECURRENCE INTERVAL NUMBER 2 5

ENTER 5 - YEAR DISCHARGE 69 ENTER

The same procedure continues until all of the data have been entered (six pairs in this example).

Press F1 and then press ENTER to exit

DATA PAIR	RECURRENCE INTERVAL	DISCHARGE
1	2	10
2	5	69

ENTER RECURRENCE INTERVAL NUMBER 3 10 ENTER

Press F1 and then press ENTER to exit

DATA PAIR	RECURRENCE INTERVAL	DISCHARGE
1	2	10
2	5	69

ENTER RECURRENCE INTERVAL NUMBER 3 10

ENTER 10 - YEAR DISCHARGE 191 ENTER

Press F1 and then press ENTER to exit

DATA PAIR	RECURRENCE INTERVAL	DISCHARGE
1	2	10
2	5	69
3	10	191

ENTER RECURRENCE INTERVAL NUMBER 4 20 ENTER

Press F1 and then press ENTER to exit

DATA PAIR	RECURRENCE INTERVAL	DISCHARGE
1	2	10
2	5	69
3	10	191

ENTER RECURRENCE INTERVAL NUMBER 4 20

ENTER 20 - YEAR DISCHARGE 441 ENTER

Press F1 and then press ENTER to exit

DATA PAIR	RECURRENCE INTERVAL	DISCHARGE
1	2	10
2	5	69
3	10	191
4	20	441

ENTER RECURRENCE INTERVAL NUMBER 5 50 ENTER

Press F1 and then press ENTER to exit

DATA PAIR	RECURRENCE INTERVAL	DISCHARGE
1	2	10
2	5	69
3	10	191
4	20	441

ENTER RECURRENCE INTERVAL NUMBER 5 50

ENTER 50 - YEAR DISCHARGE 1132 ENTER

Press F1 and then press ENTER to exit

DATA PAIR	RECURRENCE INTERVAL	DISCHARGE
1	2	10
2	5	69
3	10	191
4	20	441
5	50	1132

ENTER RECURRENCE INTERVAL NUMBER 6 100 ENTER

Press F1 and then press ENTER to exit

DATA PAIR	RECURRENCE INTERVAL	DISCHARGE
1	2	10
2	5	69
3	10	191
4	20	441
5	50	1132

ENTER RECURRENCE INTERVAL NUMBER 6 100

ENTER 100 - YEAR DISCHARGE 212 ENTER

Note an error has been made in entering the 100-year flood discharge. The program notifies us of the error with the following screen.

Press F1 and then press ENTER to exit

SORRY, DISCHARGE VALUES CANNOT DECREASE WITH INCREASING RECURRENCE INTERVALS

THE 100 - YEAR DISCHARGE (212 CFS) IS LESS THAN
THE 50 - YEAR DISCHARGE (1132 CFS)

DATA PAIR	RECURRENCE INTERVAL	DISCHARGE
5	50	1132
6	100	212

WHICH PAIR OF DATA DO YOU WISH TO CHANGE -
DATA PAIR NUMBER 5 OR DATA PAIR NUMBER 6 ? 6 ENTER

The user is then prompted to re-enter the recurrence interval first.

```
Press F1 and then press ENTER to exit

SORRY DISCHARGE VALUES CANNOT DECREASE WITH INCREASING RECURRENCE INTERVALS

THE 100 - YEAR DISCHARGE ( 212 CFS ) IS LESS THAN
THE  50 - YEAR DISCHARGE ( 1132 CFS )

        DATA PAIR            RECURRENCE INTERVAL     DISCHARGE
            5                                 50          1132
            6                                100           212

WHICH PAIR OF DATA DO YOU WISH TO CHANGE --
DATA PAIR NUMBER 5 OR DATA PAIR NUMBER  6 ?  6

RE--ENTER RECURRENCE INTERVAL NUMBER  6     100  [ENTER]
```

Now, the discharge value that was entered incorrectly is corrected.

```
Press F1 and then press ENTER to exit

SORRY, DISCHARGE VALUES CANNOT DECREASE WITH INCREASING RECURRENCE INTERVALS

THE 100 - YEAR DISCHARGE ( 212 CFS ) IS LESS THAN
THE  50 - YEAR DISCHARGE ( 1132 CFS )

        DATA PAIR            RECURRENCE INTERVAL     DISCHARGE
            5                                 50          1132
            6                                100           212

WHICH PAIR OF DATA DO YOU WISH TO CHANGE -
DATA PAIR NUMBER 5 OR DATA PAIR NUMBER  6 ?  6

RE--ENTER RECURRENCE INTERVAL NUMBER  6     100
ENTER 100 - YEAR DISCHARGE                            2120  [ENTER]
```

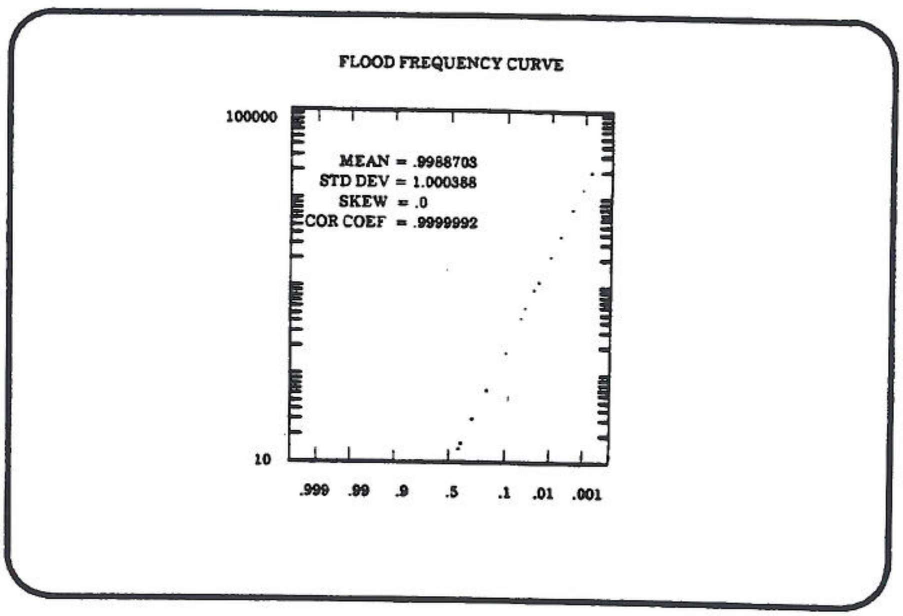
The user may review the final set of data pairs. If no revisions are necessary, an "N" is entered.

Press F1 and then press ENTER to exit

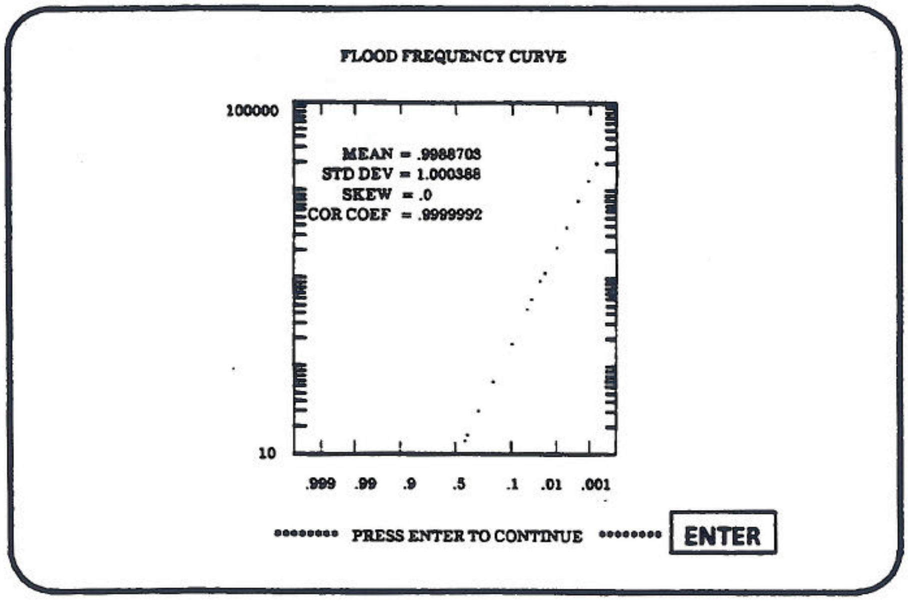
DATA PAIR	RECURRENCE INTERVAL	DISCHARGE
1	2	10
2	5	69
3	10	191
4	20	441
5	50	1132
6	100	2120

DO YOU WISH TO CHANGE ANY DATA (Y/N)? N ENTER

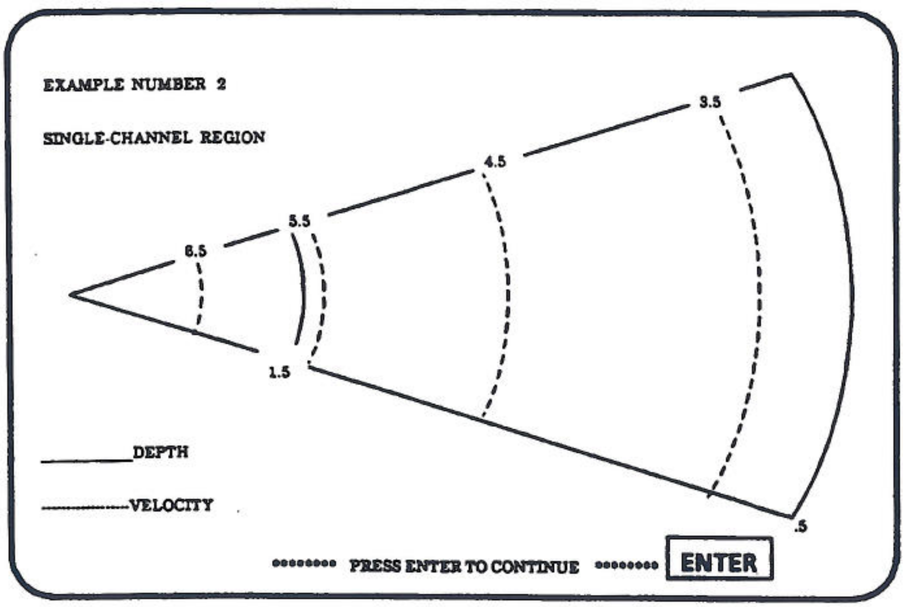
Entering "N" completes the data entry requirements. When the computation of the flood-frequency curve is complete, the statistics are displayed and the curve is drawn.



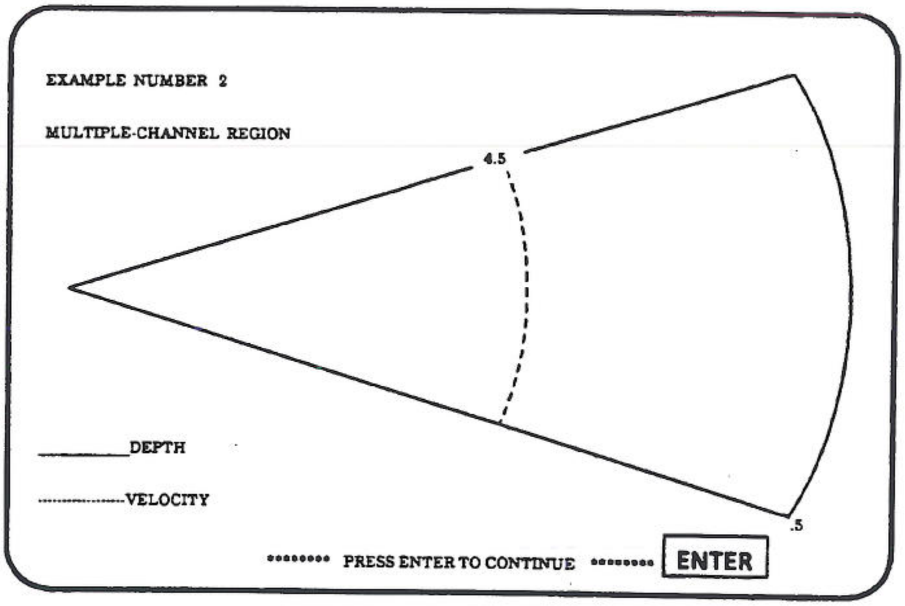
As in the first example, when the computations of the contour widths are completed, a message appears at the bottom of the screen.



By pressing ENTER, the user clears the screen and the picture of the the alluvial fan is drawn.



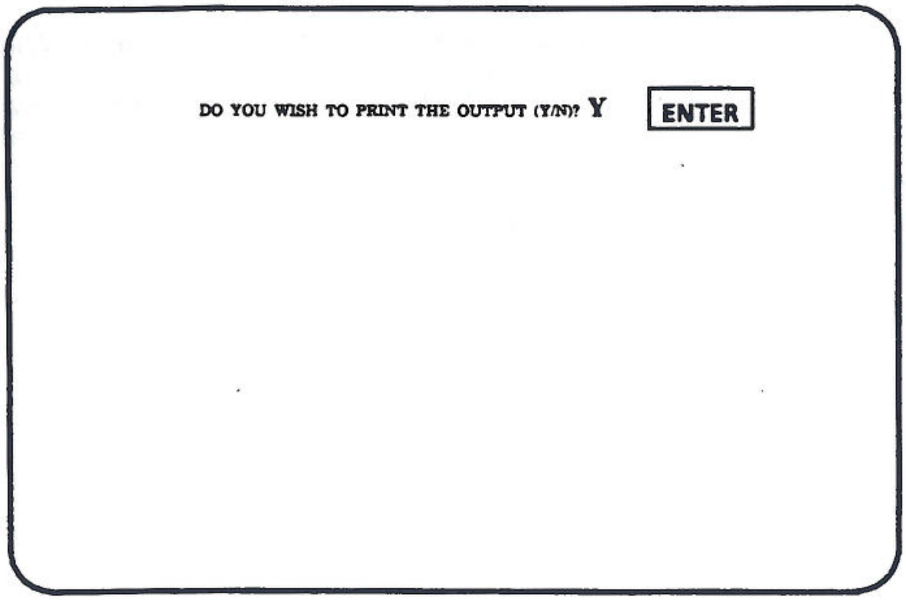
By pressing ENTER, the user clears the screen and the picture is drawn again—this time with the boundaries for the zones for the multiple-channel region shown.



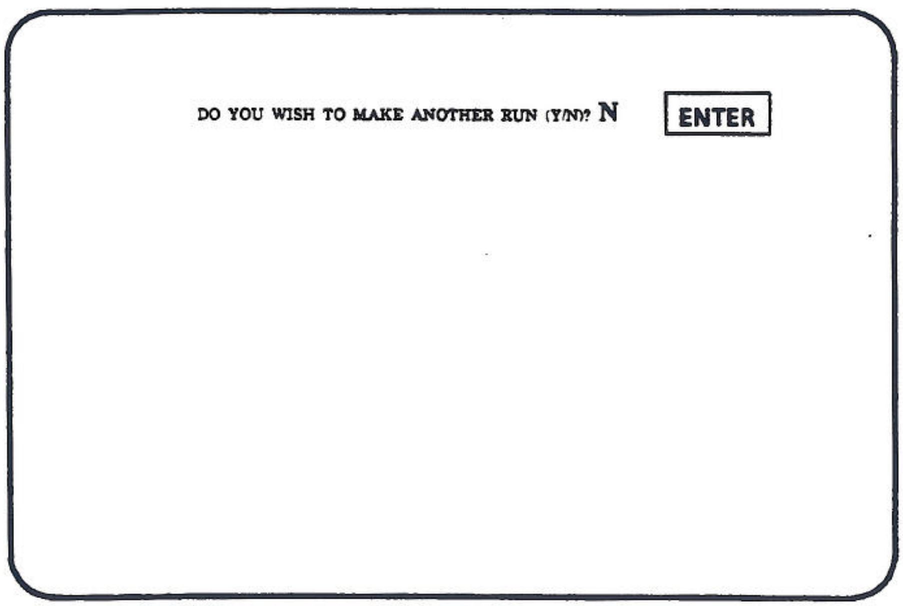
The run is complete. The user is asked again if he/she wants to view the output. The answer is "No," so an "N" is entered.

DO YOU WISH TO VIEW SOME OUTPUT DATA (Y/N)? N ENTER

The next prompt asks if the user wishes to print the output.



By entering "Y", the user sends the output file to the printer.
Before exiting, the program will ask the user if another run is to be made.



By entering "N", the user returns control to DOS.

5.4 OUTPUT FROM EXAMPLES

The output data for the two examples are presented in Figures 5-3 to 5-7.

To demonstrate that the contour widths given in the last column (Width) of the second and third pages of the output do indeed satisfy Equation (2.31) (or Equation (2.37) for the multiple-channel region), the calculation for the contour width of 623 feet corresponding to an energy of 1.5 feet will be reproduced. Note that at critical flow the depth of water in a wide rectangular channel is two-thirds of the energy. Thus, the depth in the second column of the output is 1.0 foot, as shown in Figure 5-4. The discharge associated with alluvial fan flooding with an energy of 1.5 feet in the single-channel region is determined below (see Equation 2.18).

$$
\begin{aligned}
q &= 274.4D^{2.5} \\
&= 274.4(1.5)^{2.5} \\
&= 756 \text{ cfs}
\end{aligned}
\qquad (5.1)
$$

This discharge is given in the third column of the output (Figure 5-4.).

The fourth and fifth columns of the output are the probabilities that 756 cfs is exceeded at the apex by the discharge, Q, and by 8.3176 times the first power of the discharge, 8.3176 $Q^{1.0000}$, respectively. Table 5-1 shows the information used by the program to calculate those probabilities.

EXAMPLE NUMBER 1

AVULSION FACTOR = 1.0000

FLOOD FREQUENCY CURVE DEFINED BY MEAN, STANDARD DEVIATION, AND SKEW

MEAN =	1.000000
STANDARD DEVIATION =	1.000000
SKEW =	0.0

SUMMARY OF DISCHARGES:

10-YEAR DISCHARGE =	191
50-YEAR DISCHARGE =	1132
100-YEAR DISCHARGE =	2120
500-YEAR DISCHARGE =	7554

STATISTICS AFTER TRANSFORMATION OF Y=LOG(Q) TO Z=0.9200+LOG(Q)

MEAN OF Z =	1.920000
STANDARD DEVIATION =	1.000000
SKEW =	0.000000
TRANSFORMATION CONSTANT =	3.819044

Figure 5—3. Page 1 of Output for Example Number 1

SINGLE-CHANNEL REGION

ENERGY (FT)	DEPTH (FT)	DISCHARGE (CFS)	PROBABILITY OF DISCHARGE BEING EXCEEDED AT THE APEX BY: Q	1.0000 8.3176 Q	WIDTH (FT)
0.5	0.3	49	0.24912	0.59255	2129
1.5	1.0	756	0.03083	0.17341	623

VELOCITY (FT/SEC)	DEPTH (FT)	DISCHARGE (CFS)	PROBABILITY OF DISCHARGE BEING EXCEEDED AT THE APEX BY: Q	1.0000 8.3176 Q	WIDTH (FT)
3.5	0.4	68	0.20348	0.53549	1924
4.5	0.6	238	0.08696	0.32512	1166
5.5	0.9	649	0.03560	0.18853	676
6.5	1.3	1496	0.01556	0.10608	366

Figure 5—4. Page 2 of Output for Example Number 1

EXAMPLE NUMBER 2

AVULSION FACTOR = 1.0000

FLOOD FREQUENCY CURVE DEFINED BY LEAST-SQUARES FIT OF DATA

RETURN INTERVAL (YEARS)	INPUT DISCHARGE (CFS)	BEST FIT DISCHARGE (CFS)
2	10	10
5	69	69
10	191	191
20	441	441
50	1132	1131
100	2120	2119

MEAN = 0.998870
STANDARD DEVIATION = 1.000388
SKEW = 0.0

SUMMARY OF DISCHARGES:

10-YEAR DISCHARGE = 191
50-YEAR DISCHARGE = 1131
100-YEAR DISCHARGE = 2119
500-YEAR DISCHARGE = 7553

STATISTICS AFTER TRANSFORMATION OF Y=LOG(Q) TO Z=0.9207+LOG(Q)

MEAN OF Z = 1.919584
STANDARD DEVIATION = 1.000388
SKEW = 0.000000
TRANSFORMATION CONSTANT = 3.816320

Figure 5—5. Page 1 of Output for Example Number 2

SINGLE-CHANNEL REGION

ENERGY (FT)	DEPTH (FT)	DISCHARGE (CFS)	PROBABILITY OF DISCHARGE BEING EXCEEDED AT THE APEX BY: Q	1.0000 8.3313 Q	WIDTH (FT)
0.5	0.3	49	0.24885	0.59235	2127
1.5	1.0	756	0.03080	0.17340	623

VELOCITY (FT/SEC)	DEPTH (FT)	DISCHARGE (CFS)	PROBABILITY OF DISCHARGE BEING EXCEEDED AT THE APEX BY: Q	1.0000 8.3313 Q	WIDTH (FT)
3.5	0.4	68	0.20322	0.53532	1922
4.5	0.6	238	0.08688	0.32503	1165
5.5	0.9	649	0.03557	0.18852	676
6.5	1.3	1496	0.01555	0.10609	366

Figure 5—6. Page 2 of Output for Example Number 2

MULTIPLE-CHANNEL REGION

SLOPE = 0.0850000
N-VALUE = 0.0500000

ENERGY (FT)	DEPTH (FT)	DISCHARGE (CFS)	PROBABILITY OF DISCHARGE BEING EXCEEDED AT THE APEX BY: Q	1.0000 8.3313 Q	WIDTH (FT)
0.5	0.3	426	0.05206	0.24163	3277

VELOCITY (FT/SEC)	DEPTH (FT)	DISCHARGE (CFS)	PROBABILITY OF DISCHARGE BEING EXCEEDED AT THE APEX BY: Q	1.0000 8.3313 Q	WIDTH (FT)
4.5	0.4	925	0.02465	0.15351	2085

Figure 5—7. Page 3 of Output for Example Number 2

Table 5–1. Probabilities and Corresponding Values of k (Skew = 0)

Probability	k
0.9999	-3.71902
0.9995	-3.29053
0.9990	-3.09023
0.9980	-2.87816
0.9950	-2.57583
0.9900	-2.32635
0.9800	-2.05375
0.9750	-1.95996
0.9600	-1.75069
0.9500	-1.64485
0.9000	-1.28155
0.8000	-0.84162
0.7000	-0.52440
0.6000	-0.25335
0.5704	-0.17733
0.5000	0.00000
0.4296	0.17733
0.4000	0.25335
0.3000	0.52440
0.2000	0.84162
0.1000	1.28155
0.0500	1.64485
0.0400	1.75069
0.0250	1.95996
0.0200	2.05375
0.0100	2.32635
0.0050	2.57583
0.0020	2.87816
0.0010	3.09023
0.0005	3.29053
0.0001	3.71902

Note that $\log_{10}$ (756) is 2.87852 and is, therefore, 1.87852 standard deviations (of 1.0) from the mean 1.0. Thus, the probability of 756 cfs being exceeded at the apex by Q is 0.03083, and is computed as follows:

$$P(Q>756) = \left(\frac{1.95996-1.87852}{1.95996-1.75069}\right)(0.040-0.025) + 0.025$$

$$= 0.03083 \tag{5.2}$$

This is given in the fourth column of the output, shown in Figure 5-4.

Also note that $\log_{10}$ (756) is 0.95852 standard deviations (of 1.0) from the mean 1.92 (See Equation 2.17). Thus, the probability of 8.3176 $Q^{1.0000}$ exceeding 756 cfs at the apex is 0.17342, and is computed as follows:

$$P(8.3176\, Q^{1.0000} > 756) = \left(\frac{1.28155 - 0.95852}{1.28155 - 0.84162}\right)(0.2000 - 0.1000) + 0.1000$$

$$= 0.17342 \tag{5.3}$$

This is given in the fifth column of the output. (The 0.00001 difference between the value above and that shown in Figure 5-4 arises from rounding the discharge value and its base 10 logarithm.) The latter probability could also be calculated using the change in scale of Q. That is,

$$P(8.3176\, Q^{1.0000} > 756) = P(Q > 756/8.3176 = 90.89) \tag{5.4}$$

Because $\log_{10}$ (90.89) is 1.95852 and is, therefore, 0.95852 standard deviations (of 1.0) from the mean 1.0, Equation (5.3) holds for the rescaled discharge and the flood-frequency curve defined at the apex for Q.

Equation (5.3) defines the probability denoted $P(Z > \log_{10} q_i)$ in Equation (2.31). The transformation constant C in Equation (2.31) is defined by Equation (2.16)

$$C = e^{0.92\mu + 0.42\sigma^2}$$

$$= e^{0.92 + 0.42}$$

$$= 3.819044 \tag{5.5}$$

This is given on the first page of the output (see Figure 5-3). The avulsion factor in the examples is 1.0.

The correction for high flow values is accomplished by calculating the value of the discharge (denoted q_w in equation (2.31)) that would create a channel as wide as the contour width, 623 feet. By using Equation (2.6), q_w is determined by

$$q_w = \left(\frac{W}{9.408}\right)^{2.5}$$

$$= \left(\frac{623}{9.408}\right)^{2.5}$$

$$= 35684 \text{ cfs} \tag{5.6}$$

The k-values that correspond to q_w, and the rescaled q_w are

$$\log_{10}(35684) - 1.0 = 3.55247 \tag{5.7}$$

and

$$\log_{10}(35684) - 1.92 = \log_{10}(35684/8.3176) - 1.0$$

$$= 2.63247 \tag{5.8}$$

Therefore, the two probabilities, $P(Q>q_w)$ and $P(Z> \log_{10} q_w)$ in Equation (2.31) are

$$P(Q>q_w) = \left(\frac{3.71902-3.55247}{3.71902-3.29053}\right)(0.0005-0.0001)+0.0001$$

$$= 0.00026 \tag{5.9}$$

and

$$P(Z>\log_{10}q_w) = P\left(8.3176Q^{1.0000}>35684\right)$$

$$= \left(\frac{2.87816-2.63247}{2.87816-2.57583}\right)(0.0050-0.0020)+0.0020$$

$$= 0.00444 \tag{5.10}$$

Thus, the probability that a point on the contour at which the area subject to flooding is 623 feet wide will be inundated by a flood discharge of 756 cfs or more is from Equation (2.31)

$$P(H=1) = \frac{9.408\ AC}{W}\left[P(Z > \log_{10} q_i) - P(Z> \log_{10}q_w)\right] + P(Q >q_w)$$

$$= \frac{(9.408)(1.0)(3.819044)}{623}\left[0.17342 - 0.00444\right] + 0.00026$$

$$= 0.01001 \qquad (5.11)$$

Carrying out the calculations to a precision of eight significant figures yields a value of 0.01000021 for the probability. A calculation without the large flow correction, but with the same precision, yields a probability of 0.01000068.

5.5 DELINEATING FLOOD INSURANCE ZONE BOUNDARIES

FEMA designates areas subject to 100-year alluvial fan flooding as Zone AO, with 100-year flood depths and velocities shown. The depths shown are summations of the pressure heads and velocity heads. Depths are rounded to the nearest whole foot and velocities are rounded to the nearest foot per second (fps). Thus, the area subject to alluvial fan flooding with 100-year flood depths between 1.5 and 2.5 feet and 100-year flood velocities between 5.5 and 6.5 fps is labeled:

ZONE AO
(DEPTH 2)
(VELOCITY 6 FPS)

The net output data of the FAN program are the energies (labeled DEPTH on the FIRM), velocities, and the corresponding widths. The latter are the widths of the area subject to alluvial fan flooding at which the corresponding energy or velocity has a 1-percent chance of being exceeded in any given year. To delineate the flood insurance zone boundaries, the elevation at which the area subject to alluvial fan flooding has a width equal to the width in the output must be determined. To shorten the terminology, that width is referred to as the contour width (because it is measured along a contour).

The contour width is not the length of the contour, including every bend and wind, between the boundaries of the area subject to flooding. Instead, it is the length of a "smoothed" contour that has the same general alignment as the "true" contour. Figure 5-8 shows the alluvial fan used in the previous examples with the boundaries of the area subject to flooding. Figure 5-9 shows the same fan with "smoothed" contours for width measurements.

The output data from Example Number 1 will be used to demonstrate. To delineate the boundary between the zones where the 100-year flood depths are 1 foot and less than 1 foot, the elevation at which the contour width is 2,129 feet is located. To delineate the boundary between the zones where the 100-year flood depths are 1 foot and 2 feet, the elevation at which the contour width is 623 feet is located. The zone boundaries are delineated at those elevations using the alignments suggested by the smooth contours on Figure 5-9. The area between those boundaries will be labeled "DEPTH 1." The area above the 623-foot-long boundary will be labeled "DEPTH 2." Note that, in the example, the 100-year flood discharge at the apex is 2,120 cfs. Therefore, the maximum 100-year flood depth is 2.26 feet and, so, the maximum depth labeled is 2 feet. (See Condition 2.23.)

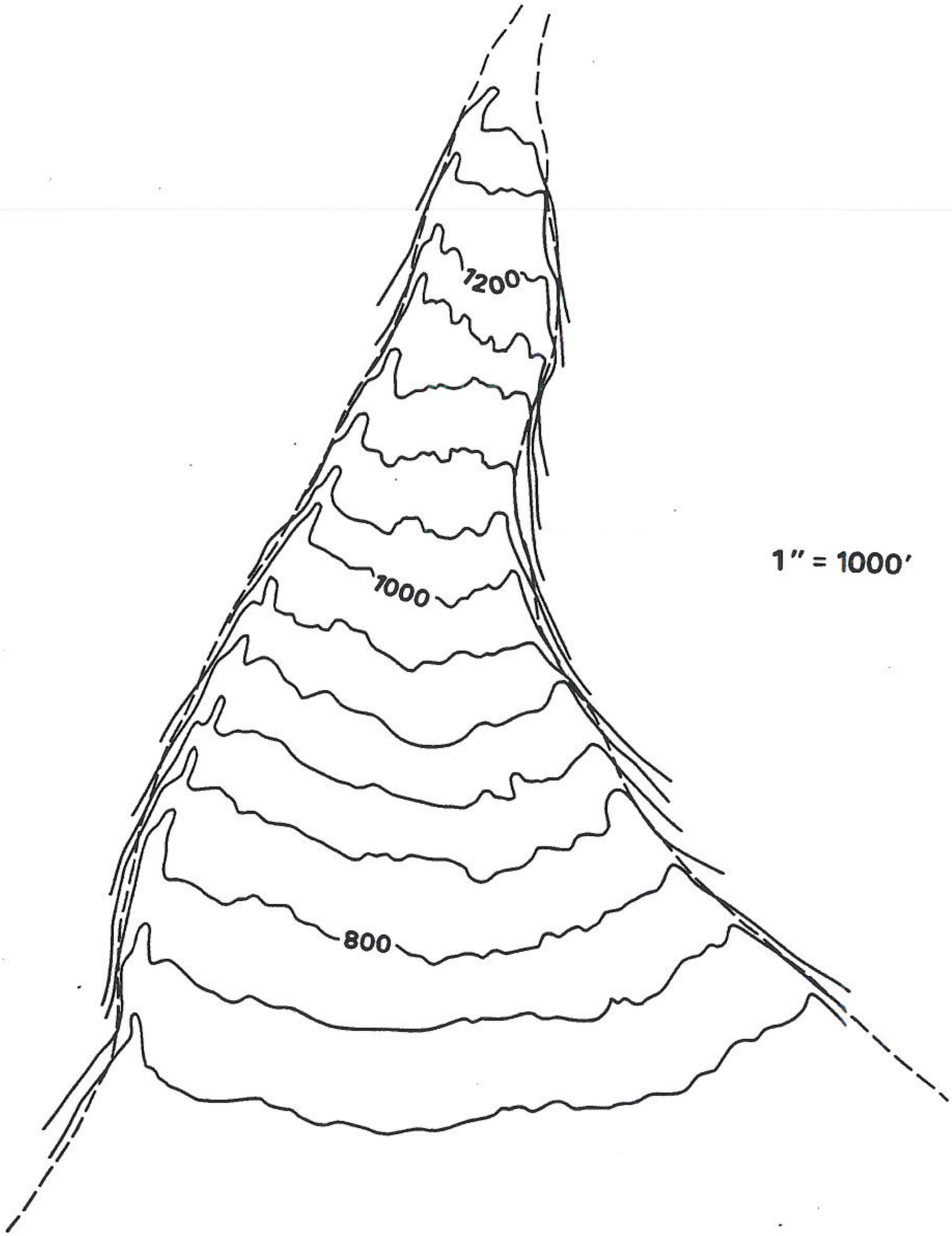


Figure 5—8. Alluvial Fan Boundaries

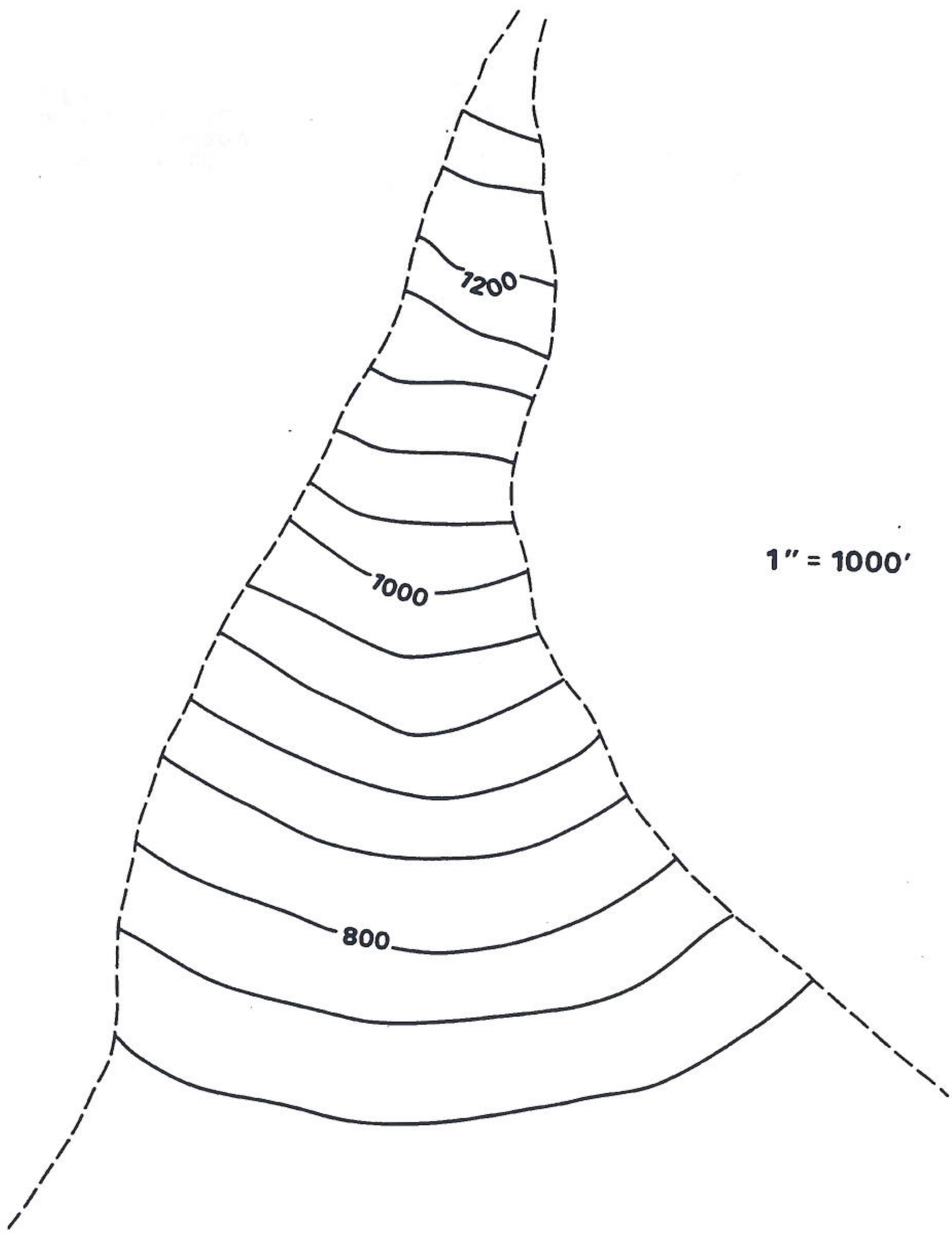


Figure 5—9. "Smooth" Contours for Width Measurements

The delineations of the depth zone boundaries for this single-channel region are shown on Figure 5-10. The same procedure is followed to delineate the velocity zone boundaries. The velocity zone boundaries subdivide the depth zones. The end result is a map depicting a series of regions defined by pairs of 100-year flood depths and velocities. Figure 5-11 shows that result with the appropriate labels in the flood insurance zones.

In multiple-channel regions, the widths resulting from the multiple-channel analysis are used. In Example Number 2, the multiple-channel region is the area with elevations below 1,180 feet. Because the area subject to flooding is 1,050 feet wide at that elevation, all widths greater than 1,050 feet are taken from the multiple-channel output data. For example, the boundary between the 4-fps and 5-fps zones is where the area subject to flooding is 2,085 feet wide--not 1,165 feet wide (single-channel region); the boundary between the 1-foot depth zone and areas where 100-year flood depths are less than 0.5 foot is where the area subject to flooding is 3,277 feet wide--not 2,127 feet wide (single-channel region). Figure 5-12 shows the results corresponding to Example Number 2 with bifurcation points at elevation 1,180 feet.

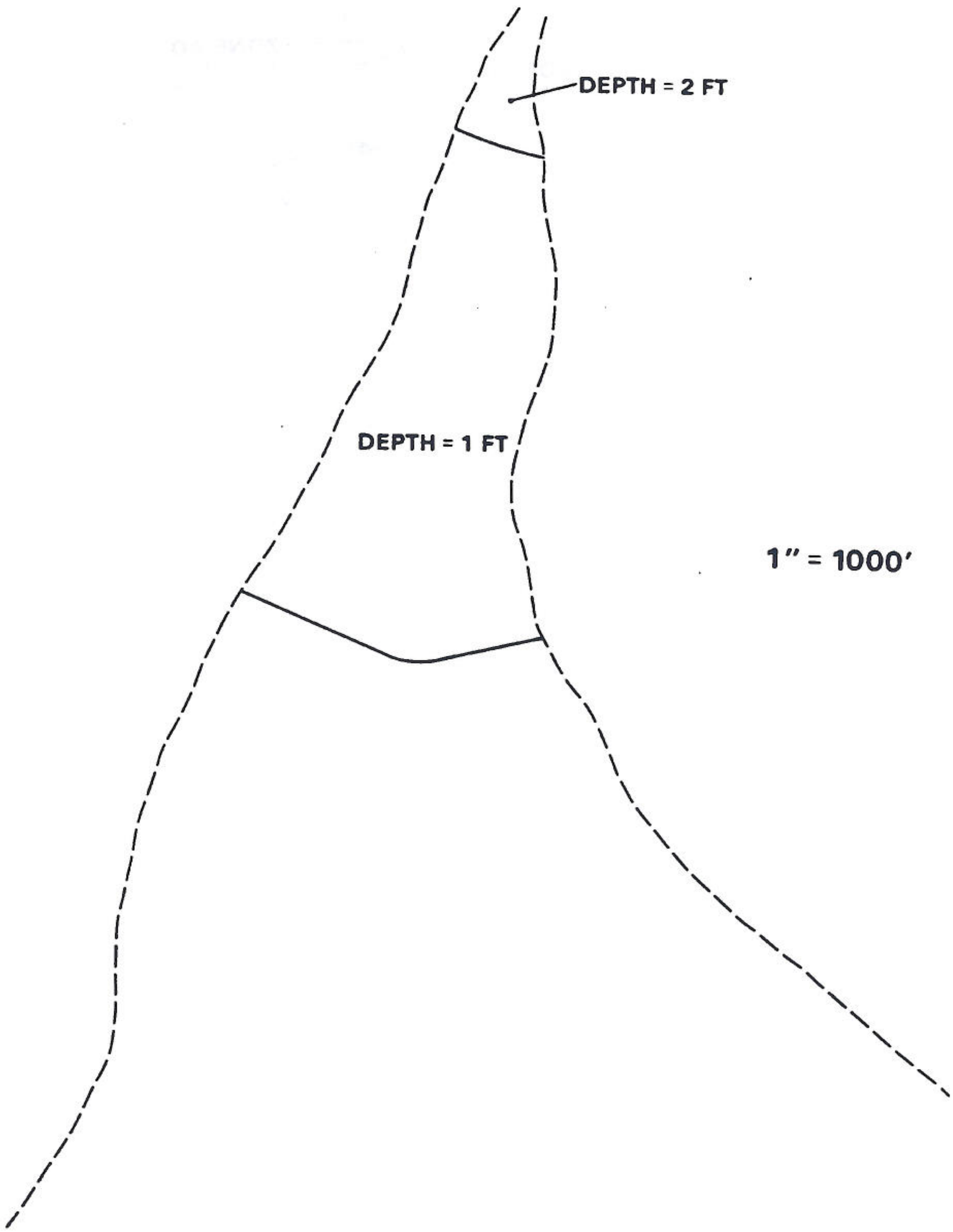


Figure 5—10. Depth Zone Boundaries (Single-Channel Region)

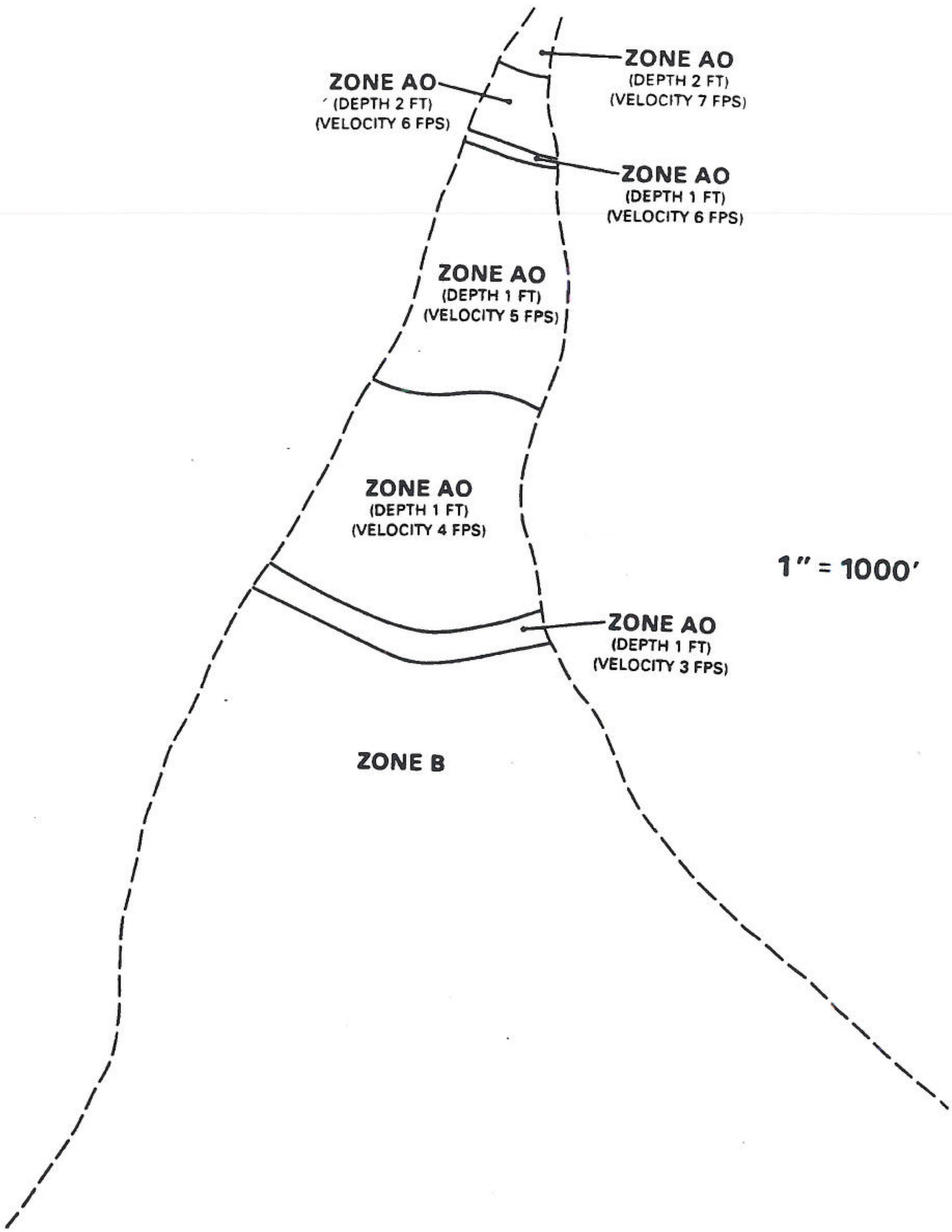


Figure 5—11. Flood Insurance Zones (Single-Channel Region)

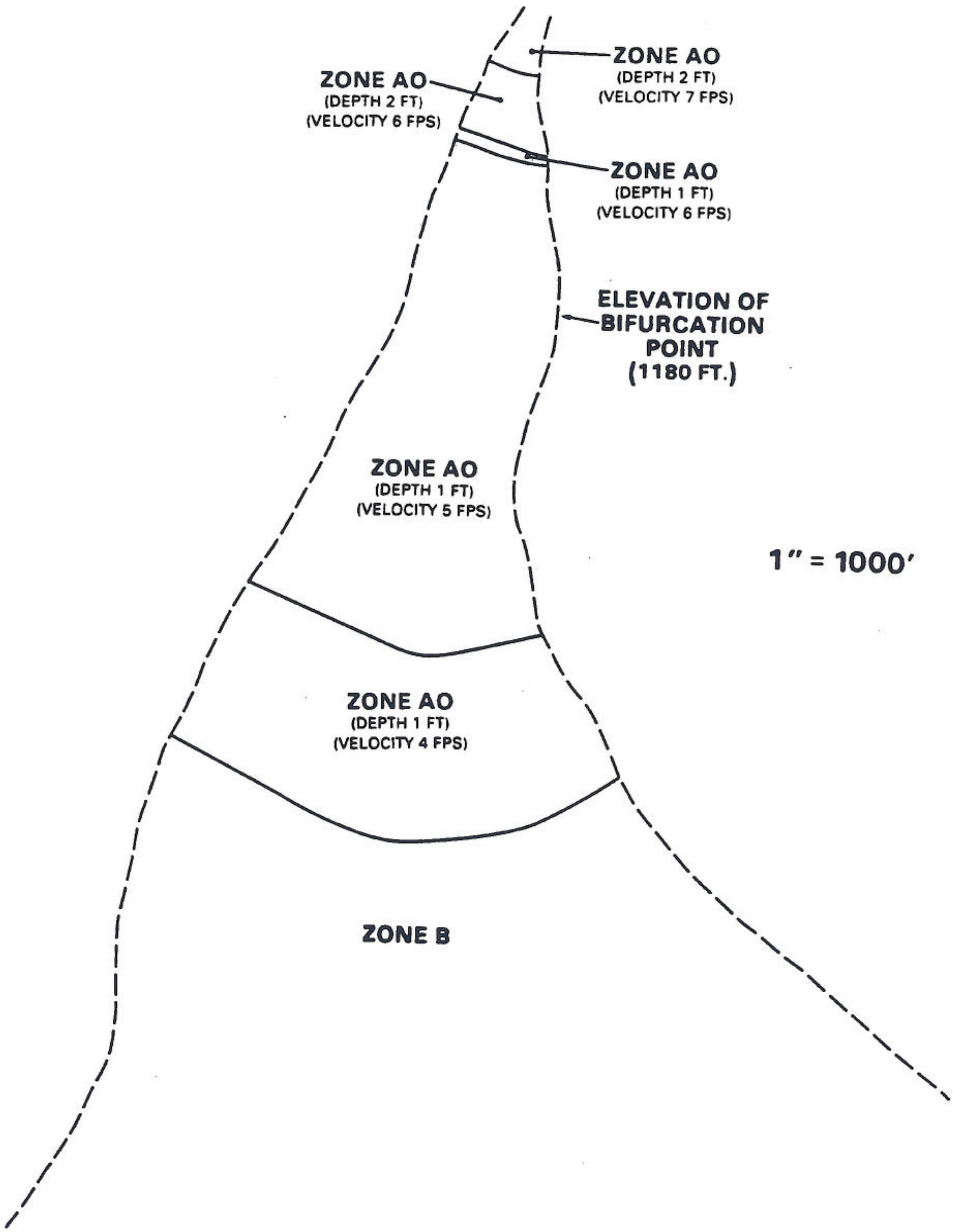


Figure 5—12. Flood Insurance Zones (Multiple-Channels Region)

SECTION 6 — REFERENCES

1. David R. Dawdy, "Flood Frequency Estimates on Alluvial Fans," in Journal of Hydraulics Division, ASCE, Proceedings, Vol. 105, No. HYII, 1979, pp 1407-1413.

2. DMA Consulting Engineers, Alluvial Fan Flooding Methodology, An Analysis, prepared for Federal Emergency Management Agency, 1985.

3. Interagency Advisory Committee on Water Data, Hydrology Subcommittee, Bulletin No. 17B, Guidelines for Determining Flood Flow Frequency, 1982.

APPENDIX A — SUPPLEMENT TO DERIVATION

A.1 HYDRAULIC CHARACTERISTICS: SINGLE-CHANNEL REGION

Recall that, at critical depth, the water depth, d, in a rectangular channel is twice the velocity head, $v^2/2g$. Thus, for a constant discharge, q, flowing at critical velocity, v, in a rectangular channel of cross-sectional area, A, and width, w,

$$d = 2\left(\frac{v^2}{2g}\right) = \frac{1}{g} v^2$$

$$= \frac{1}{g}\left(\frac{q}{A}\right)^2$$

$$= \frac{1}{g} \frac{q^2}{d^2 w^2} \qquad \text{(A.1)}$$

where g is the acceleration of gravity.

Therefore,

$$w^2 = \frac{q^2}{g} \frac{1}{d^3} \qquad \text{(A.2)}$$

or, writing the width as a function of depth,

$$w(d) = \frac{q}{\sqrt{g}} d^{-3/2} \qquad \text{(A.3)}$$

The condition

$$\frac{dw}{dd} = -200 \qquad \text{(A.4)}$$

where $\frac{d}{dd}$ denotes differentiation with respect to depth, yields

$$\frac{dw}{dd} = -\frac{q}{\sqrt{g}}\left(\frac{3}{2}\right)d^{-5/2} \tag{A.5}$$

That is, Condition (A.4) is satisfied when

$$d = \left[\left(\frac{q}{\sqrt{g}}\right)\left(\frac{3}{2}\right)\left(\frac{1}{200}\right)\right]^{2/5}$$

$$= 0.07056\, q^{2/5} \tag{A.6}$$

Writing the width as a function of discharge [Equation (A.3)] yields

$$w(q) = \frac{q}{\sqrt{g}}\, d^{-3/2}$$

$$= \frac{q}{\sqrt{g}}\left[\frac{3q}{400\sqrt{g}}\right]^{-3/5}$$

$$= 9.408\, q^{2/5} \tag{A.7}$$

The discharge associated with an energy depth of D feet is the discharge corresponding to a water depth of $d = 2D/3$ feet in Equation (A.6):

$$\frac{2}{3}D = \left[\left(\frac{q}{\sqrt{g}}\right)\left(\frac{3}{2}\right)\frac{1}{200}\right]^{2/5} \tag{A.8}$$

That is,

$$q = 200\sqrt{g}\left(\frac{3}{2}\right)^{-3.5} D^{2.5}$$

$$= 274.4\, D^{2.5} \tag{A.9}$$

Similarly, the discharge associated with a velocity, v, is the discharge corresponding to a water depth of $d = 2(v^2/2g)$ feet in Equation (A.6):

$$\frac{v^2}{2g} = \frac{1}{2}\left[\left(\frac{q}{\sqrt{g}}\right)\left(\frac{3}{2}\right)\left(\frac{1}{200}\right)\right]^{2/5} \tag{A.10}$$

That is,

$$q = \left(\frac{400}{3g^2}\right) v^5$$

$$= 0.1289v^5 \tag{A.11}$$

A.2 HYDRAULIC CHARACTERISTCS: MULTIPLE-CHANNEL REGION

Assume that the depth and velocity of floodflows can be estimated with Manning's equation with the friction slope set equal to the slope of the alluvial fan (i.e., a normal depth approximation). Thus, we define the relationship between depth and discharge as

$$q = \frac{1.486}{n} AR^{2/3} s^{1/2} \tag{A.12}$$

where s is the slope of the alluvial fan, R is the hydraulic radius of the channel, and A is the cross-sectional area of that part of the channel under water. Assuming a wide rectangular channel, we can approximate the hydraulic radius, R, by the depth of water, d. Writing the width as

$$w(q) = (3.8)(9.408)q^{2/5} = 35.7504\, q^{2/5} \tag{A.13}$$

and the area as width times depth yields

$$q = \frac{1.486}{n} w(q)\, d^{5/3} s^{1/2}$$

$$= \frac{1.486}{n} 35.7504\, q^{2/5} d^{5/3} s^{1/2}$$

$$= \frac{53.1251}{n} q^{2/5} d^{5/3} s^{1/2} \tag{A.14}$$

Solving for d yields

$$d = \left[\frac{n}{53.1251} q^{3/5} s^{-1/2} \right]^{3/5}$$

$$= 0.0922\, n^{0.6} s^{-0.3} q^{0.36} \quad \text{(A.15)}$$

Manning's equation for velocity, v, is

$$v = \frac{1.486}{n} R^{2/3} s^{1/2} \quad \text{(A.16)}$$

Making the same substitution for R as before yields the depth-velocity relationship

$$v = \frac{1.486}{n} d^{2/3} s^{1/2} \quad \text{(A.17)}$$

Using Equation (A.15) for d yields

$$v = \frac{1.486}{n} \left(0.0922\, n^{0.6} s^{-0.3} q^{0.36} \right)^{2/3} s^{1/2}$$

$$= .3033\, n^{-.6} s^{.3} q^{.24} \quad \text{(A.18)}$$

The velocity head can be computed from Equation (A.18):

$$\frac{v^2}{2g} = \frac{1}{2g} \left(0.3033\, n^{-0.6} s^{0.3} q^{0.24} \right)^2$$

$$= 0.00143\, n^{-1.2} s^{0.6} q^{0.48} \quad \text{(A.19)}$$

Therefore, the energy depth, D, in the multiple-channel region is

$$D = d + \frac{v^2}{2g}$$

$$= 0.0922\, n^{0.6} s^{-0.3} q^{0.36} + 0.00143\, n^{-1.2} s^{0.6} q^{0.48} \qquad \text{(A.20)}$$

A.3 DERIVATION OF EQUATION (2.11)

Rearranging

$$0.01 = P(H=1) = \int_{y_{100}}^{\infty} 9.408 \frac{e^{0.92y}}{W} \frac{\lambda^k (y-m)^{k-1}}{\Gamma(k)} e^{-\lambda(y-m)}\, dy \qquad \text{(A.21)}$$

to

$$0.01 = \frac{9.408}{W} \int_{y_{100}}^{\infty} \frac{(y-m)^{k-1}}{\Gamma(k)} \left\{ \lambda^k e^{-\lambda(y-m)} e^{0.92y} \right\} dy \qquad \text{(A.22)}$$

suggests writing

$$\left\{ \lambda^k e^{-\lambda(y-m)} e^{0.92y} \right\}$$

as

$$\frac{(\lambda-0.92)^k}{(\lambda-0.92)^k} \lambda^k e^{-(\lambda-0.92)(y-m)} e^{0.92m}$$

which is the same as

$$\frac{e^{0.92m} \lambda^k}{(\lambda-0.92)^k} (\lambda-0.92)^k e^{-(\lambda-0.92)(y-m)}$$

Defining the constant

$$C = \frac{e^{0.92m} \lambda^k}{(\lambda-0.92)^k} \qquad \text{(A.23)}$$

and the new parameter

$$\lambda' = \lambda - 0.92 \tag{A.24}$$

yields

$$0.01 = \frac{9.408C}{W} \int_{y_{100}}^{\infty} \frac{(\lambda')^k (y-m)^{k-1}}{\Gamma(k)} e^{-\lambda'(y-m)} dy \tag{A.25}$$

A.4 DERIVATION OF EQUATION (2.15)

Start with

$$0.01 = \int_{y_{100}}^{\infty} 9.408 \frac{e^{0.92y}}{W} \frac{e^{-\frac{1}{2}\left(\frac{y-\mu}{\sigma}\right)^2}}{\sigma\sqrt{2\pi}} dy \tag{A.26}$$

Expand the square, rearrange, and add and subtract

$$\left[0.92\mu + \frac{(0.92\sigma)^2}{2}\right]$$

to the exponent.

That is, the exponent can be written

$$0.92\,y - \tfrac{1}{2}\left(\frac{y-\mu}{\sigma}\right)^2 = \frac{1}{2\sigma^2}\left\{[0.92\,y\,2\sigma^2 - y^2 + 2y\mu - \mu^2] + [2\sigma^2 0.92\mu + (0.92\sigma^2)^2] - [2\sigma^2 0.92\mu + (0.92\sigma^2)^2]\right\}$$

$$= -\frac{1}{2\sigma^2}[y - (\mu + 0.92\sigma^2)]^2 + 0.92\mu + 0.42\sigma^2 \tag{A.27}$$

Defining the constant

$$C = e^{0.92\mu + 0.42\sigma^2} \tag{A.28}$$

and the new mean

$$\mu' = \mu + 0.92\sigma^2 \tag{A.29}$$

yields

$$0.01 = \frac{9.408\,C}{W} \int_{y_{100}}^{\infty} \frac{e^{-\frac{1}{2}\left(\frac{y-\mu'}{\sigma}\right)^2}}{\sigma\sqrt{2\pi}}\,dy \tag{A.30}$$

APPENDIX B — LISTING

Table of Contents for Program FAN

Table of Contents for Program FAN (Cont'd)

```
1 REM
2 REM    PPPPPPPP  EEEEEEE       AAA       RRRRRRRR   SSSSSSSS  NNN      NNN
3 REM    PPP   PPP EEE          AAAAA      RRR   RRR SSS        NNNN     NNN
4 REM    PPP   PPP EEE         AAA AAA     RRR   RRR SSS        NNNNNN   NNN
5 REM    PPPPPPPP  EEEEEE     AAA   AAA    RRRRRRRR   SSSSSSS   NNN NNN  NNN
6 REM    PPP       EEE       AAAAAAAAAAA   RRR   RRR        SSS NNN   NNNNNN
7 REM    PPP       EEE      AAA       AAA  RRR    RRR       SSS NNN     NNNN
8 REM    PPP       EEEEEEE  AAA       AAA  RRR    RRR SSSSSSSS  NNN      NNN
9 REM
10  REM Portions (c) Copyright
11  REM Microsoft Corporation
12  REM 1982-1988
13  REM All Rights Reserved
14  REM
15  REM
16  REM ALLUVIAL FAN / Michael Baker Jr., Inc. - JULY 1990
17  REM
18  REM
19 COMMON SHARED P(), K(), KG(), KO()
20 DIM P(30), K(83, 30), KG(30), KO(30)
21 REM
22 ON KEY(1) GOSUB 10000
23 KEY(1) ON
24 CLS : KEY OFF: SCREEN 0: COLOR 10
25 LOCATE 6, 20
26 PRINT "ALLUVIAL FAN FLOODING COMPUTER PROGRAM"
28 LOCATE 9, 20
30 PRINT "In response to prompts, enter data one "
31 LOCATE 10, 20
32 PRINT "value at a time, then press enter."
34 LOCATE 14, 20
35 PRINT "Answer yes-or-no questions with the"
36 LOCATE 15, 20
37 PRINT "corresponding letters (i.e., Y or N)."
38 LOCATE 20, 20


40 REM
52 REM
54 REM       ****************************************************************
56 REM                              OPEN OUTPUT FILE
58 REM       ****************************************************************
60 REM
62 OPEN "FAN.IN" FOR OUTPUT AS #1
63 LOCATE 20, 20
64 COLOR 18
66 PRINT "PEARSON TYPE-III TABLES BEING LOADED"
```

```
9000 REM
9002 REM      ***************************************************************
9004 REM                       ASSIGN K VALUES FOR PEARSON TYPE 3
9006 REM      ***************************************************************
9008 REM
9012  P(0) = .9999: P(1) = .9995: P(2) = .999: P(3) = .998: P(4) = .995:
P(5) = .99: P(6) = .98
9014  P(7) = .975: P(8) = .96: P(9) = .95: P(10) = .9: P(11) = .8: P(12)
= .7: P(13) = .6
9016  P(14) = .5704: P(15) = .5: P(16) = .4296: P(17) = .4: P(18) = .3: P
(19) = .2: P(20) = .1
9018  P(21) = .05: P(22) = .04: P(23) = .025: P(24) = .02: P(25) = .01: P
(26) = .005: P(27) = .002
9020  P(28) = .001: P(29) = .0005: P(30) = .0001
9022  K(0, 0) = -3.71902: K(1, 0) = -3.50703: K(2, 0) = -3.29921: K(3, 0)
 = -3.09631: K(4, 0) = -2.89907: K(5, 0) = -2.70836: K(6, 0) = -2.52507
9024  K(0, 1) = -3.29053: K(1, 1) = -3.12767: K(2, 1) = -2.96698: K(3, 1)
 = -2.80889: K(4, 1) = -2.6539: K(5, 1) = -2.50257: K(6, 1) = -2.35549
9026  K(0, 2) = -3.09023: K(1, 2) = -2.94834: K(2, 2) = -2.80786: K(3, 2)
 = -2.66915: K(4, 2) = -2.53261: K(5, 2) = -2.39867: K(6, 2) = -2.2678
9028  K(0, 3) = -2.87816: K(1, 3) = -2.75706: K(2, 3) = -2.63672: K(3, 3)
 = -2.51741: K(4, 3) = -2.39942: K(5, 3) = -2.28311: K(6, 3) = -2.16884
9030  K(0, 4) = -2.57583: K(1, 4) = -2.48187: K(2, 4) = -2.38795: K(3, 4)
 = -2.29423: K(4, 4) = -2.20092: K(5, 4) = -2.10825: K(6, 4) = -2.01644
9032  K(0, 5) = -2.32635: K(1, 5) = -2.25258: K(2, 5) = -2.1784: K(3, 5)
= -2.10394: K(4, 5) = -2.02933: K(5, 5) = -1.95472: K(6, 5) = -1.88029
9034  K(0, 6) = -2.05375: K(1, 6) = -1.99973: K(2, 6) = -1.94499: K(3, 6)
 = -1.88959: K(4, 6) = -1.83361: K(5, 6) = -1.77716: K(6, 6) = -1.7203
9036  K(0, 7) = -1.95996: K(1, 7) = -1.91219: K(2, 7) = -1.8636: K(3, 7)
= -1.81427: K(4, 7) = -1.76427: K(5, 7) = -1.71366: K(6, 7) = -1.66253
9038  K(0, 8) = -1.75069: K(1, 8) = -1.7158: K(2, 8) = -1.67999: K(3, 8)
= -1.64329: K(4, 8) = -1.60574: K(5, 8) = -1.5674: K(6, 8) = -1.5283
9040  K(0, 9) = -1.64485: K(1, 9) = -1.61594: K(2, 9) = -1.58607: K(3, 9)
 = -1.55527: K(4, 9) = -1.52357: K(5, 9) = -1.49101: K(6, 9) = -1.45762
9042  K(0, 10) = -1.28155: K(1, 10) = -1.27037: K(2, 10) = -1.25824: K(3,
 10) = -1.24516: K(4, 10) = -1.23114: K(5, 10) = -1.21618: K(6, 10) = -1.
20028
9044  K(0, 11) = -.84162: K(1, 11) = -.84611: K(2, 11) = -.84986: K(3, 11
) = -.85285: K(4, 11) = -.85508: K(5, 11) = -.85653: K(6, 11) = -.85718
9046  K(0, 12) = -.5244: K(1, 12) = -.53624: K(2, 12) = -.54757: K(3, 12)
 = -.55839: K(4, 12) = -.56867: K(5, 12) = -.5784: K(6, 12) = -.58757
9048  K(0, 13) = -.25335: K(1, 13) = -.26882: K(2, 13) = -.28403: K(3, 13
) = -.29897: K(4, 13) = -.31362: K(5, 13) = -.32796: K(6, 13) = -.34198
9050  K(0, 14) = -.17733: K(1, 14) = -.19339: K(2, 14) = -.20925: K(3, 14
) = -.22492: K(4, 14) = -.24037: K(5, 14) = -.25558: K(6, 14) = -.27047
9052  K(0, 15) = 0!: K(1, 15) = -.01662: K(2, 15) = -.03325: K(3, 15) = -
.04993: K(4, 15) = -.06651: K(5, 15) = -.08302: K(6, 15) = -.09945
9054  K(0, 16) = .17733: K(1, 16) = .16111: K(2, 16) = .14472: K(3, 16) =
 .1282: K(4, 16) = .11154: K(5, 16) = .09478: K(6, 16) = .07791
9056  K(0, 17) = .25335: K(1, 17) = .23763: K(2, 17) = .22168: K(3, 17) =
 .20552: K(4, 17) = .18916: K(5, 17) = .17261: K(6, 17) = .15589
9058  K(0, 18) = .5244: K(1, 18) = .51207: K(2, 18) = .49927: K(3, 18) =
.486: K(4, 18) = .47228: K(5, 18) = .45812: K(6, 18) = .44352
9060  K(0, 19) = .84162: K(1, 19) = .83639: K(2, 19) = .83044: K(3, 19) =
 .82377: K(4, 19) = .81638: K(5, 19) = .80829: K(6, 19) = .7995
9062  K(0, 20) = 1.28155: K(1, 20) = 1.29178: K(2, 20) = 1.30105: K(3, 20
) = 1.30936: K(4, 20) = 1.31671: K(5, 20) = 1.32309: K(6, 20) = 1.3285
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9064 K(0, 21) = 1.64485: K(1, 21) = 1.67279: K(2, 21) = 1.69971: K(3, 21
) = 1.72562: K(4, 21) = 1.75048: K(5, 21) = 1.77428: K(6, 21) = 1.79701
9066 K(0, 22) = 1.75069: K(1, 22) = 1.78462: K(2, 22) = 1.81756: K(3, 22
) = 1.84949: K(4, 22) = 1.88039: K(5, 22) = 1.91022: K(6, 22) = 1.93896
9068 K(0, 23) = 1.95996: K(1, 23) = 2.00688: K(2, 23) = 2.0529: K(3, 23)
= 2.09795: K(4, 23) = 2.14202: K(5, 23) = 2.18505: K(6, 23) = 2.22702
9070 K(0, 24) = 2.05375: K(1, 24) = 2.10697: K(2, 24) = 2.15935: K(3, 24
) = 2.21081: K(4, 24) = 2.26133: K(5, 24) = 2.31084: K(6, 24) = 2.35931
9072 K(0, 25) = 2.32635: K(1, 25) = 2.39961: K(2, 25) = 2.47226: K(3, 25
) = 2.54421: K(4, 25) = 2.61539: K(5, 25) = 2.68572: K(6, 25) = 2.75514
9074 K(0, 26) = 2.57583: K(1, 26) = 2.66965: K(2, 26) = 2.76321: K(3, 26
) = 2.85636: K(4, 26) = 2.949: K(5, 26) = 3.04102: K(6, 26) = 3.13232
9076 K(0, 27) = 2.87816: K(1, 27) = 2.99978: K(2, 27) = 3.12169: K(3, 27
) = 3.24371: K(4, 27) = 3.36566: K(5, 27) = 3.48737: K(6, 27) = 3.60872
9078 K(0, 28) = 3.09023: K(1, 28) = 3.23322: K(2, 28) = 3.37703: K(3, 28
) = 3.52139: K(4, 28) = 3.66608: K(5, 28) = 3.8109: K(6, 28) = 3.95567
9080 K(0, 29) = 3.29053: K(1, 29) = 3.45513: K(2, 29) = 3.62113: K(3, 29
) = 3.7882: K(4, 29) = 3.95605: K(5, 29) = 4.12443: K(6, 29) = 4.29311
9082 K(0, 30) = 3.71902: K(1, 30) = 3.93453: K(2, 30) = 4.15301: K(3, 30
) = 4.37394: K(4, 30) = 4.59687: K(5, 30) = 4.82141: K(6, 30) = 5.04718
9084 K(7, 0) = -2.35015: K(8, 0) = -2.18448: K(9, 0) = -2.02891: K(10, 0
) = -1.8841: K(11, 0) = -1.75053: K(12, 0) = -1.62838: K(13, 0) = -1.5175
2
9086 K(7, 1) = -2.21328: K(8, 1) = -2.07661: K(9, 1) = -1.94611: K(10, 1
) = -1.82241: K(11, 1) = -1.70603: K(12, 1) = -1.59738: K(13, 1) = -1.496
73
9088 K(7, 2) = -2.14053: K(8, 2) = -2.01739: K(9, 2) = -1.89894: K(10, 2
) = -1.78572: K(11, 2) = -1.67825: K(12, 2) = -1.57695: K(13, 2) = -1.482
16
9090 K(7, 3) = -2.05701: K(8, 3) = -1.94806: K(9, 3) = -1.84244: K(10, 3
) = -1.74062: K(11, 3) = -1.64305: K(12, 3) = -1.55016: K(13, 3) = -1.462
32
9092 K(7, 4) = -1.9258: K(8, 4) = -1.8366: K(9, 4) = -1.74919: K(10, 4)
= -1.6639: K(11, 4) = -1.5811: K(12, 4) = -1.50114: K(13, 4) = -1.42439
9094 K(7, 5) = -1.80621: K(8, 5) = -1.73271: K(9, 5) = -1.66001: K(10, 5
) = -1.58838: K(11, 5) = -1.51808: K(12, 5) = -1.44942: K(13, 5) = -1.382
67
9096 K(7, 6) = -1.66325: K(8, 6) = -1.60604: K(9, 6) = -1.54886: K(10, 6
) = -1.49188: K(11, 6) = -1.43529: K(12, 6) = -1.37929: K(13, 6) = -1.324
12
9098 K(7, 7) = -1.61099: K(8, 7) = -1.55914: K(9, 7) = -1.50712: K(10, 7
) = -1.45507: K(11, 7) = -1.40314: K(12, 7) = -1.35153: K(13, 7) = -1.300
42
9100 K(7, 8) = -1.48852: K(8, 8) = -1.44813: K(9, 8) = -1.4072: K(10, 8)
 = -1.36584: K(11, 8) = -1.32414: K(12, 8) = -1.28225: K(13, 8) = -1.2402
8
9102 K(7, 9) = -1.42345: K(8, 9) = -1.38855: K(9, 9) = -1.35299: K(10, 9
) = -1.31684: K(11, 9) = -1.28019: K(12, 9) = -1.24313: K(13, 9) = -1.205
78
9104 K(7, 10) = -1.18347: K(8, 10) = -1.16574: K(9, 10) = -1.14712: K(10
, 10) = -1.12762: K(11, 10) = -1.10726: K(12, 10) = -1.08608: K(13, 10) =
 -1.06413
9106 K(7, 11) = -.85703: K(8, 11) = -.85607: K(9, 11) = -.85426: K(10, 1
1) = -.85161: K(11, 11) = -.84809: K(12, 11) = -.84369: K(13, 11) = -.838
41
9108 K(7, 12) = -.59615: K(8, 12) = -.60412: K(9, 12) = -.61146: K(10, 1
2) = -.61815: K(11, 12) = -.62415: K(12, 12) = -.62944: K(13, 12) = -.634
9110 K(7, 13) = -.35565: K(8, 13) = -.36889: K(9, 13) = -.38186: K(10, 1
3) = -.39434: K(11, 13) = -.40638: K(12, 13) = -.41794: K(13, 13) = -.428
99
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9112  K(7, 14) = -.28516: K(8, 14) = -.29961: K(9, 14) = -.31368: K(10, 1
4) = -.3274: K(11, 14) = -.34075: K(12, 14) = -.3537: K(13, 14) = -.3662
9114  K(7, 15) = -.11578: K(8, 15) = -.13199: K(9, 15) = -.14807: K(10, 1
5) = -.16397: K(11, 15) = -.17968: K(12, 15) = -.19517: K(13, 15) = -.210
4
9116  K(7, 16) = .06097: K(8, 16) = .04397: K(9, 16) = .02693: K(10, 16)
= .00987: K(11, 16) = -.00719: K(12, 16) = -.02421: K(13, 16) = -.04116
9118  K(7, 17) = .13901: K(8, 17) = .12199: K(9, 17) = .10486: K(10, 17)
= 8.763001E-02: K(11, 17) = .07032: K(12, 17) = .05297: K(13, 17) = .0356
9120  K(7, 18) = .42851: K(8, 18) = .41309: K(9, 18) = .39729: K(10, 18)
= .38111: K(11, 18) = .36458: K(12, 18) = .34772: K(13, 18) = .33054
9122  K(7, 19) = .79002: K(8, 19) = .77986: K(9, 19) = .76902: K(10, 19)
= .75752: K(11, 19) = .74537: K(12, 19) = .73257: K(13, 19) = .71915
9124  K(7, 20) = 1.33294: K(8, 20) = 1.3364: K(9, 20) = 1.33889: K(10, 20
) = 1.34039: K(11, 20) = 1.34092: K(12, 20) = 1.34047: K(13, 20) = 1.3390
4
9126  K(7, 21) = 1.81864: K(8, 21) = 1.83916: K(9, 21) = 1.85856: K(10, 2
1) = 1.87683: K(11, 21) = 1.89395: K(12, 21) = 1.90992: K(13, 21) = 1.924
72
9128  K(7, 22) = 1.9666: K(8, 22) = 1.99311: K(9, 22) = 2.01848: K(10, 22
) = 2.04269: K(11, 22) = 2.06573: K(12, 22) = 2.08758: K(13, 22) = 2.1082
3
9130  K(7, 23) = 2.2679: K(8, 23) = 2.30764: K(9, 23) = 2.34623: K(10, 23
) = 2.38364: K(11, 23) = 2.41984: K(12, 23) = 2.45482: K(13, 23) = 2.4885
5
9132  K(7, 24) = 2.4067: K(8, 24) = 2.45298: K(9, 24) = 2.49811: K(10, 24
) = 2.54206: K(11, 24) = 2.5848: K(12, 24) = 2.62631: K(13, 24) = 2.66657
9134  K(7, 25) = 2.82359: K(8, 25) = 2.89101: K(9, 25) = 2.95735: K(10, 2
5) = 3.02256: K(11, 25) = 3.0866: K(12, 25) = 3.14944: K(13, 25) = 3.2110
3
9136  K(7, 26) = 3.22281: K(8, 26) = 3.31243: K(9, 26) = 3.40109: K(10, 2
6) = 3.48874: K(11, 26) = 3.5753: K(12, 26) = 3.66073: K(13, 26) = 3.7449
7
9138  K(7, 27) = 3.72957: K(8, 27) = 3.84981: K(9, 27) = 3.96932: K(10, 2
7) = 4.08802: K(11, 27) = 4.20582: K(12, 27) = 4.32263: K(13, 27) = 4.438
39
9140  K(7, 28) = 4.10022: K(8, 28) = 4.24439: K(9, 28) = 4.38807: K(10, 2
8) = 4.53112: K(11, 28) = 4.67344: K(12, 28) = 4.81492: K(13, 28) = 4.955
49
9142  K(7, 29) = 4.46189: K(8, 29) = 4.63057: K(9, 29) = 4.79899: K(10, 2
9) = 4.96701: K(11, 29) = 5.13449: K(12, 29) = 5.3013: K(13, 29) = 5.4673
5
9144  K(7, 30) = 5.27389: K(8, 30) = 5.50124: K(9, 30) = 5.72899: K(10, 3
0) = 5.95691: K(11, 30) = 6.1848: K(12, 30) = 6.41249: K(13, 30) = 6.6398
9146  K(14, 0) = -1.41753: K(15, 0) = -1.32774: K(16, 0) = -1.24728: K(17
, 0) = -1.1752: K(18, 0) = -1.11054: K(19, 0) = -1.05239: K(20, 0) = -.99
99
9148  K(14, 1) = -1.40413: K(15, 1) = -1.31944: K(16, 1) = -1.24235: K(17
, 1) = -1.1724: K(18, 1) = -1.10901: K(19, 1) = -1.05159: K(20, 1) = -.99
95
9150  K(14, 2) = -1.39408: K(15, 2) = -1.31275: K(16, 2) = -1.23805: K(17
, 2) = -1.16974: K(18, 2) = -1.10743: K(19, 2) = -1.05068: K(20, 2) = -.9
99
9152  K(14, 3) = -1.37981: K(15, 3) = -1.30279: K(16, 3) = -1.23132: K(17
, 3) = -1.16534: K(18, 3) = -1.10465: K(19, 3) = -1.04898: K(20, 3) = -.9
98
9154  K(14, 4) = -1.35114: K(15, 4) = -1.28167: K(16, 4) = -1.21618: K(17
, 4) = -1.15477: K(18, 4) = -1.09749: K(19, 4) = -1.04427: K(20, 4) = -.9
9499
9156  K(14, 5) = -1.31815: K(15, 5) = -1.25611: K(16, 5) = -1.1968: K(17,
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5) = -1.14042: K(18, 5) = -1.08711: K(19, 5) = -1.03695: K(20, 5) = -.98
995
9158  K(14, 6) = -1.26999: K(15, 6) = -1.21716: K(16, 6) = -1.16584: K(17
, 6) = -1.11628: K(18, 6) = -1.06864: K(19, 6) = -1.02311: K(20, 6) = -.9
798
9160  K(14, 7) = -1.25004: K(15, 7) = -1.20059: K(16, 7) = -1.15229: K(17
, 7) = -1.10537: K(18, 7) = -1.06001: K(19, 7) = -1.0164: K(20, 7) = -.97
468
9162  K(14, 8) = -1.19842: K(15, 8) = -1.15682: K(16, 8) = -1.11566: K(17
, 8) = -1.07513: K(18, 8) = -1.03543: K(19, 8) = -.99672: K(20, 8) = -.95
918
9164  K(14, 9) = -1.16827: K(15, 9) = -1.13075: K(16, 9) = -1.09338: K(17
, 9) = -1.05631: K(18, 9) = -1.01973: K(19, 9) = -.98381: K(20, 9) = -.94
871
9166  K(14, 10) = -1.04144: K(15, 10) = -1.0181: K(16, 10) = -.99418: K(1
7, 10) = -.96977: K(18, 10) = -.94496: K(19, 10) = -.91988: K(20, 10) = -
.89464
9168  K(14, 11) = -.83223: K(15, 11) = -.82516: K(16, 11) = -.8172: K(17,
 11) = -.80837: K(18, 11) = -.79868: K(19, 11) = -.78816: K(20, 11) = -.7
7686
9170  K(14, 12) = -.63779: K(15, 12) = -.6408: K(16, 12) = -.643: K(17, 1
2) = -.64436: K(18, 12) = -.64488: K(19, 12) = -.64453: K(20, 12) = -.643
33
9172  K(14, 13) = -.43949: K(15, 13) = -.44942: K(16, 13) = -.45873: K(17
, 13) = -.46739: K(18, 13) = -.47538: K(19, 13) = -.48265: K(20, 13) = -.
48917
9174  K(14, 14) = -.37824: K(15, 14) = -.38977: K(16, 14) = -.40075: K(17
, 14) = -.41116: K(18, 14) = -.42095: K(19, 14) = -.43008: K(20, 14) = -.
43854
9176  K(14, 15) = -.22535: K(15, 15) = -.23996: K(16, 15) = -.25422: K(17
, 15) = -.26808: K(18, 15) = -.2815: K(19, 15) = -.29443: K(20, 15) = -.3
0685
9178  K(14, 16) = -.05803: K(15, 16) = -.07476: K(16, 16) = -.09132: K(17
, 16) = -.10769: K(18, 16) = -.12381: K(19, 16) = -.13964: K(20, 16) = -.
15516
9180  K(14, 17) = .01824: K(15, 17) = .00092: K(16, 17) = -.01631: K(17,
17) = -.03344: K(18, 17) = -.0504: K(19, 17) = -.06718: K(20, 17) = -.083
71
9182  K(14, 18) = .31307: K(15, 18) = .29535: K(16, 18) = .2774: K(17, 18
) = .25925: K(18, 18) = .24094: K(19, 18) = .2225: K(20, 18) = .20397
9184  K(14, 19) = .70512: K(15, 19) = .6905: K(16, 19) = .67532: K(17, 19
) = .65959: K(18, 19) = .64335: K(19, 19) = .62662: K(20, 19) = .60944
9186  K(14, 20) = 1.33665: K(15, 20) = 1.3333: K(16, 20) = 1.329: K(17, 2
0) = 1.32376: K(18, 20) = 1.3176: K(19, 20) = 1.31054: K(20, 20) = 1.3025
9
9188  K(14, 21) = 1.93836: K(15, 21) = 1.95083: K(16, 21) = 1.96213: K(17
, 21) = 1.97227: K(18, 21) = 1.98124: K(19, 21) = 1.98906: K(20, 21) = 1.
99573
9190  K(14, 22) = 2.12768: K(15, 22) = 2.14591: K(16, 22) = 2.16293: K(17
, 22) = 2.17873: K(18, 22) = 2.19332: K(19, 22) = 2.2067: K(20, 22) = 2.2
1888
9192  K(14, 23) = 2.52102: K(15, 23) = 2.55222: K(16, 23) = 2.58214: K(17
, 23) = 2.61076: K(18, 23) = 2.6381: K(19, 23) = 2.66413: K(20, 23) = 2.6
8888
9194  K(14, 24) = 2.70556: K(15, 24) = 2.74325: K(16, 24) = 2.77964: K(17
, 24) = 2.81472: K(18, 24) = 2.84848: K(19, 24) = 2.88091: K(20, 24) = 2.
91202
9196  K(14, 25) = 3.27134: K(15, 25) = 3.33035: K(16, 25) = 3.38804: K(17
, 25) = 3.44438: K(18, 25) = 3.49935: K(19, 25) = 3.55295: K(20, 25) = 3.
60517
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9198  K(14, 26) = 3.82798: K(15, 26) = 3.90973: K(16, 26) = 3.99016: K(17
, 26) = 4.06926: K(18, 26) = 4.147: K(19, 26) = 4.22336: K(20, 26) = 4.29
832
9200  K(14, 27) = 4.55304: K(15, 27) = 4.66651: K(16, 27) = 4.77875: K(17
, 27) = 4.88971: K(18, 27) = 4.99937: K(19, 27) = 5.10768: K(20, 27) =
21461
9202  K(14, 28) = 5.09505: K(15, 28) = 5.23353: K(16, 28) = 5.37087: K(17
, 28) = 5.50701: K(18, 28) = 5.6419: K(19, 28) = 5.77549: K(20, 28) = 5.9
0776
9204  K(14, 29) = 5.63252: K(15, 29) = 5.79673: K(16, 29) = 5.9599: K(17,
 29) = 6.12196: K(18, 29) = 6.28285: K(19, 29) = 6.44251: K(20, 29) = 6.6
009
9206  K(14, 30) = 6.86661: K(15, 30) = 7.09277: K(16, 30) = 7.31818: K(17
, 30) = 7.54272: K(18, 30) = 7.76632: K(19, 30) = 7.98888: K(20, 30) = 8.
21034
9208  K(21, 0) = -.95234: K(22, 0) = -.90908: K(23, 0) = -.86956: K(24, 0
) = -.83333: K(25, 0) = -.8: K(26, 0) = -.76923: K(27, 0) = -.74074
9210  K(21, 1) = -.95215: K(22, 1) = -.90899: K(23, 1) = -.86952: K(24, 1
) = -.83331: K(25, 1) = -.79999: K(26, 1) = -.76923: K(27, 1) = -.74074
9212  K(21, 2) = -.95188: K(22, 2) = -.90885: K(23, 2) = -.86945: K(24, 2
) = -.83328: K(25, 2) = -.79998: K(26, 2) = -.76922: K(27, 2) = -.74074
9214  K(21, 3) = -.95131: K(22, 3) = -.90854: K(23, 3) = -.86929: K(24, 3
) = -.8332: K(25, 3) = -.79994: K(26, 3) = -.7692: K(27, 3) = -.74073
9216  K(21, 4) = -.94945: K(22, 4) = -.90742: K(23, 4) = -.86863: K(24, 4
) = -.83283: K(25, 4) = -.79973: K(26, 4) = -.76909: K(27, 4) = -.74067
9218  K(21, 5) = -.94607: K(22, 5) = -.90521: K(23, 5) = -.86723: K(24, 5
) = -.83196: K(25, 5) = -.79921: K(26, 5) = -.76878: K(27, 5) = -.74049
9220  K(21, 6) = -.93878: K(22, 6) = -.90009: K(23, 6) = -.86371: K(24, 6
) = -.82959: K(25, 6) = -.79765: K(26, 6) = -.76779: K(27, 6) = -.73987
9222  K(21, 7) = -.93495: K(22, 7) = -.89728: K(23, 7) = -.86169: K(24  7
) = -.82817: K(25, 7) = -.79667: K(26, 7) = -.76712: K(27, 7) = -.7394
9224  K(21, 8) = -.92295: K(22, 8) = -.88814: K(23, 8) = -.85486: K(24, 8
) = -.82315: K(25, 8) = -.79306: K(26, 8) = -.76456: K(27, 8) = -.73765
9226  K(21, 9) = -.91458: K(22, 9) = -.88156: K(23, 9) = -.84976: K(24, 9
) = -.81927: K(25, 9) = -.79015: K(26, 9) = -.76242: K(27, 9) = -.7361
9228  K(21, 10) = -.86938: K(22, 10) = -.84422: K(23, 10) = -.81929: K(24
, 10) = -.79472: K(25, 10) = -.77062: K(26, 10) = -.74709: K(27, 10) = -.
72422
9230  K(21, 11) = -.76482: K(22, 11) = -.75211: K(23, 11) = -.7388: K(24,
 11) = -.72495: K(25, 11) = -.71067: K(26, 11) = -.69602: K(27, 11) = -.6
8111
9232  K(21, 12) = -.64125: K(22, 12) = -.63833: K(23, 12) = -.63456: K(24
, 12) = -.62999: K(25, 12) = -.62463: K(26, 12) = -.61854: K(27, 12) = -.
61176
9234  K(21, 13) = -.49494: K(22, 13) = -.49991: K(23, 13) = -.50409: K(24
, 13) = -.50744: K(25, 13) = -.50999: K(26, 13) = -.51171: K(27, 13) = -.
51263
9236  K(21, 14) = -.44628: K(22, 14) = -.45329: K(23, 14) = -.45953: K(24
, 14) = -.46499: K(25, 14) = -.46966: K(26, 14) = -.47353: K(27, 14) = -.
4766
9238  K(21, 15) = -.31872: K(22, 15) = -.32999: K(23, 15) = -.34063: K(24
, 15) = -.35062: K(25, 15) = -.35992: K(26, 15) = -.36852: K(27, 15) = -.
3764
9240  K(21, 16) = -.1703: K(22, 16) = -.18504: K(23, 16) = -.19933: K(24,
 16) = -.21313: K(25, 16) = -.22642: K(26, 16) = -.23915: K(27, 16) = -.2
5129
9242  K(21, 17) = -.09997: K(22, 17) = -.1159: K(23, 17) = -.13148: K(24,
 17) = -.14665: K(25, 17) = -.16138: K(26, 17) = -.17564: K(27, 17) = -.1
8939
9244  K(21, 18) = .1854: K(22, 18) = .16682: K(23, 18) = .14827: K(24, 18
```

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) = .12979: K(25, 18) = .11143: K(26, 18) = .09323: K(27, 18) = .07523
9246  K(21, 19) = .59183: K(22, 19) = .57383: K(23, 19) = .55549: K(24, 1
9) = .53683: K(25, 19) = .51789: K(26, 19) = .49872: K(27, 19) = .47934
9248  K(21, 20) = 1.29377: K(22, 20) = 1.28412: K(23, 20) = 1.27365: K(24
, 20) = 1.2624: K(25, 20) = 1.25039: K(26, 20) = 1.23766: K(27, 20) = 1.2
2422
9250  K(21, 21) = 2.00128: K(22, 21) = 2.0057: K(23, 21) = 2.00903: K(24,
 21) = 2.01128: K(25, 21) = 2.01247: K(26, 21) = 2.01263: K(27, 21) = 2.0
1177
9252  K(21, 22) = 2.22986: K(22, 22) = 2.23967: K(23, 22) = 2.24831: K(24
, 22) = 2.25581: K(25, 22) = 2.26217: K(26, 22) = 2.26743: K(27, 22) = 2.
2716
9254  K(21, 23) = 2.71234: K(22, 23) = 2.73451: K(23, 23) = 2.75541: K(24
, 23) = 2.77506: K(25, 23) = 2.79345: K(26, 23) = 2.81062: K(27, 23) = 2.
82658
9256  K(21, 24) = 2.94181: K(22, 24) = 2.97028: K(23, 24) = 2.99744: K(24
, 24) = 3.0233: K(25, 24) = 3.04787: K(26, 24) = 3.07116: K(27, 24) = 3.0
932
9258  K(21, 25) = 3.656: K(22, 25) = 3.70543: K(23, 25) = 3.75347: K(24,
25) = 3.80013: K(25, 25) = 3.8454: K(26, 25) = 3.8893: K(27, 25) = 3.9318
3
9260  K(21, 26) = 4.37186: K(22, 26) = 4.44398: K(23, 26) = 4.51467: K(24
, 26) = 4.58393: K(25, 26) = 4.65176: K(26, 26) = 4.71815: K(27, 26) = 4.
78313
9262  K(21, 27) = 5.32014: K(22, 27) = 5.42426: K(23, 27) = 5.52694: K(24
, 27) = 5.62818: K(25, 27) = 5.72796: K(26, 27) = 5.82629: K(27, 27) = 5.
92316
9264  K(21, 28) = 6.03865: K(22, 28) = 6.16816: K(23, 28) = 6.29626: K(24
, 28) = 6.42292: K(25, 28) = 6.54814: K(26, 28) = 6.67191: K(27, 28) = 6.
79421
9266  K(21, 29) = 6.75798: K(22, 29) = 6.9137: K(23, 29) = 7.06804: K(24,
 29) = 7.22098: K(25, 29) = 7.3725: K(26, 29) = 7.52258: K(27, 29) = 7.67
121
9268  K(21, 30) = 8.43064: K(22, 30) = 8.64971: K(23, 30) = 8.86753: K(24
, 30) = 9.08403: K(25, 30) = 9.2992: K(26, 30) = 9.51301: K(27, 30) = 9.7
25429
9270  K(28, 0) = -.71429: K(29, 0) = -.68966: K(30, 0) = -.66667: K(31, 0
) = -.64516: K(32, 0) = -.625: K(33, 0) = -.60606: K(34, 0) = -.58824
9272  K(28, 1) = -.71429: K(29, 1) = -.68966: K(30, 1) = -.66667: K(31, 1
) = -.64516: K(32, 1) = -.625: K(33, 1) = -.60606: K(34, 1) = -.58824
9274  K(28, 2) = -.71428: K(29, 2) = -.68965: K(30, 2) = -.66667: K(31, 2
) = -.64516: K(32, 2) = -.625: K(33, 2) = -.60606: K(34, 2) = -.58824
9276  K(28, 3) = -.71428: K(29, 3) = -.68965: K(30, 3) = -.66667: K(31, 3
) = -.64516: K(32, 3) = -.625: K(33, 3) = -.60606: K(34, 3) = -.58824
9278  K(28, 4) = -.71425: K(29, 4) = -.68964: K(30, 4) = -.66666: K(31, 4
) = -.64516: K(32, 4) = -.625: K(33, 4) = -.60606: K(34, 4) = -.58824
9280  K(28, 5) = -.71415: K(29, 5) = -.68959: K(30, 5) = -.66663: K(31, 5
) = -.64514: K(32, 5) = -.62499: K(33, 5) = -.60606: K(34, 5) = -.58823
9282  K(28, 6) = -.71377: K(29, 6) = -.68935: K(30, 6) = -.66649: K(31, 6
) = -.64507: K(32, 6) = -.62495: K(33, 6) = -.60603: K(34, 6) = -.58822
9284  K(28, 7) = -.71348: K(29, 7) = -.68917: K(30, 7) = -.66638: K(31, 7
) = -.645: K(32, 7) = -.62491: K(33, 7) = -.60601: K(34, 7) = -.58821
9286  K(28, 8) = -.71227: K(29, 8) = -.68836: K(30, 8) = -.66585: K(31, 8
) = -.64465: K(32, 8) = -.62469: K(33, 8) = -.60587: K(34, 8) = -.58812
9288  K(28, 9) = -.71116: K(29, 9) = -.68759: K(30, 9) = -.66532: K(31, 9
) = -.64429: K(32, 9) = -.62445: K(33, 9) = -.60572: K(34, 9) = -.58802
9290  K(28, 10) = -.70209: K(29, 10) = -.68075: K(30, 10) = -.66023: K(31
, 10) = -.64056: K(32, 10) = -.62175: K(33, 10) = -.60379: K(34, 10) = -.
58666
9292  K(28, 11) = -.66603: K(29, 11) = -.65086: K(30, 11) = -.63569: K(31
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, 11) = -.6206: K(32, 11) = -.60567: K(33, 11) = -.59096: K(34, 11) = -.5
7652
9294  K(28, 12) = -.60434: K(29, 12) = -.59634: K(30, 12) = -.58783: K(31
, 12) = -.57887: K(32, 12) = -.56953: K(33, 12) = -.55989: K(34, 12) = -.
55
9296  K(28, 13) = -.51276: K(29, 13) = -.51212: K(30, 13) = -.5107301: K(
31, 13) = -.50863: K(32, 13) = -.5058501: K(33, 13) = -.50244: K(34, 13)
= -.49844
9298  K(28, 14) = -.47888: K(29, 14) = -.48037: K(30, 14) = -.48109: K(31
, 14) = -.48107: K(32, 14) = -.48033: K(33, 14) = -.4789: K(34, 14) = -.4
7682
9300  K(28, 15) = -.38353: K(29, 15) = -.38991: K(30, 15) = -.39554: K(31
, 15) = -.40041: K(32, 15) = -.40454: K(33, 15) = -.40792: K(34, 15) = -.
41058
9302  K(28, 16) = -.26282: K(29, 16) = -.27372: K(30, 16) = -.28395: K(31
, 16) = -.29351: K(32, 16) = -.30238: K(33, 16) = -.31055: K(34, 16) = -.
31802
9304  K(28, 17) = -.20259: K(29, 17) = -.21523: K(30, 17) = -.22726: K(31
, 17) = -.23868: K(32, 17) = -.24946: K(33, 17) = -.25958: K(34, 17) = -.
26904
9306  K(28, 18) = .05746: K(29, 18) = .03997: K(30, 18) = .02279: K(31, 1
8) = .00596: K(32, 18) = -.0105: K(33, 18) = -.02654: K(34, 18) = -.04215
9308  K(28, 19) = .4598: K(29, 19) = .44015: K(30, 19) = .4204: K(31, 19)
 = .40061: K(32, 19) = .38081: K(33, 19) = .36104: K(34, 19) = .34133
9310  K(28, 20) = 1.21013: K(29, 20) = 1.19539: K(30, 20) = 1.18006: K(31
, 20) = 1.16416: K(32, 20) = 1.14772: K(33, 20) = 1.13078: K(34, 20) = 1.
11337
9312  K(28, 21) = 2.00992: K(29, 21) = 2.0071: K(30, 21) = 2.00335: K(31,
 21) = 1.99869: K(32, 21) = 1.99314: K(33, 21) = 1.98674: K(34, 21) = 1.9
7951
9314  K(28, 22) = 2.2747: K(29, 22) = 2.27676: K(30, 22) = 2.2778: K(
22) = 2.27785: K(32, 22) = 2.27693: K(33, 22) = 2.27506: K(34, 22) = 2.27
229
9316  K(28, 23) = 2.84134: K(29, 23) = 2.85492: K(30, 23) = 2.86735: K(31
, 23) = 2.87865: K(32, 23) = 2.88884: K(33, 23) = 2.89795: K(34, 23) = 2.
90599
9318  K(28, 24) = 3.11399: K(29, 24) = 3.13356: K(30, 24) = 3.15193: K(31
, 24) = 3.16911: K(32, 24) = 3.18512: K(33, 24) = 3.2: K(34, 24) = 3.2137
5
9320  K(28, 25) = 3.97301: K(29, 25) = 4.01286: K(30, 25) = 4.05138: K(31
, 25) = 4.08859: K(32, 25) = 4.12452: K(33, 25) = 4.15917: K(34, 25) = 4.
19257
9322  K(28, 26) = 4.84669: K(29, 26) = 4.90884: K(30, 26) = 4.96959: K(31
, 26) = 5.02897: K(32, 26) = 5.08697: K(33, 26) = 5.14362: K(34, 26) = 5.
19892
9324  K(28, 27) = 6.01858: K(29, 27) = 6.11254: K(30, 27) = 6.20506: K(31
, 27) = 6.29613: K(32, 27) = 6.38578: K(33, 27) = 6.47401: K(34, 27) = 6.
56084
9326  K(28, 28) = 6.91505: K(29, 28) = 7.03443: K(30, 28) = 7.15235: K(31
, 28) = 7.26881: K(32, 28) = 7.38382: K(33, 28) = 7.49739: K(34, 28) = 7.
60953
9328  K(28, 29) = 7.81839: K(29, 29) = 7.96411: K(30, 29) = 8.10836: K(31
, 29) = 8.25115: K(32, 29) = 8.39248: K(33, 29) = 8.53236: K(34, 29) = 8.
67079
9330  K(28, 30) = 9.93643: K(29, 30) = 10.14602: K(30, 30) = 10.35418: K(
31, 30) = 10.5609: K(32, 30) = 10.76618: K(33, 30) = 10.97001: K(34,
= 11.17239
9332  K(35, 0) = -.57143: K(36, 0) = -.55556: K(37, 0) = -.54054: K(38, 0
) = -.52632: K(39, 0) = -.51282: K(40, 0) = -.5: K(41, 0) = -.4878
9334  K(35, 1) = -.57143: K(36, 1) = -.55556: K(37, 1) = -.54054: K(38, 1
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```
) = -.52632: K(39, 1) = -.51282: K(40, 1) = -.5: K(41, 1) = -.4878
9336  K(35, 2) = -.57143: K(36, 2) = -.55556: K(37, 2) = -.54054: K(38, 2
) = -.52632: K(39, 2) = -.51282: K(40, 2) = -.5: K(41, 2) = -.4878
9338  K(35, 3) = -.57143: K(36, 3) = -.55556: K(37, 3) = -.54054: K(38, 3
) = -.52632: K(39, 3) = -.51282: K(40, 3) = -.5: K(41, 3) = -.4878
9340  K(35, 4) = -.57143: K(36, 4) = -.55556: K(37, 4) = -.54054: K(38, 4
) = -.52632: K(39, 4) = -.51282: K(40, 4) = -.5: K(41, 4) = -.4878
9342  K(35, 5) = -.57143: K(36, 5) = -.55556: K(37, 5) = -.54054: K(38, 5
) = -.52632: K(39, 5) = -.51282: K(40, 5) = -.5: K(41, 5) = -.4878
9344  K(35, 6) = -.57142: K(36, 6) = -.55555: K(37, 6) = -.54054: K(38, 6
) = -.52631: K(39, 6) = -.51282: K(40, 6) = -.5: K(41, 6) = -.4878
9346  K(35, 7) = -.57141: K(36, 7) = -.55555: K(37, 7) = -.54054: K(38, 7
) = -.52631: K(39, 7) = -.51282: K(40, 7) = -.5: K(41, 7) = -.4878
9348  K(35, 8) = -.57136: K(36, 8) = -.55552: K(37, 8) = -.54052: K(38, 8
) = -.5263: K(39, 8) = -.51281: K(40, 8) = -.5: K(41, 8) = -.4878
9350  K(35, 9) = -.5713: K(36, 9) = -.55548: K(37, 9) = -.5405: K(38, 9)
= -.52629: K(39, 9) = -.51281: K(40, 9) = -.49999: K(41, 9) = -.4878
9352  K(35, 10) = -.57035: K(36, 10) = -.55483: K(37, 10) = -.54006: K(38
, 10) = -.526: K(39, 10) = -.5126101: K(40, 10) = -.49986: K(41, 10) = -.
48772
9354  K(35, 11) = -.56242: K(36, 11) = -.54867: K(37, 11) = -.53533: K(38
, 11) = -.5224001: K(39, 11) = -.5099: K(40, 11) = -.49784: K(41, 11) = -
.48622
9356  K(35, 12) = -.53993: K(36, 12) = -.52975: K(37, 12) = -.51952: K(38
, 12) = -.50929: K(39, 12) = -.49911: K(40, 12) = -.48902: K(41, 12) = -.
47906
9358  K(35, 13) = -.49391: K(36, 13) = -.48888: K(37, 13) = -.48342: K(38
, 13) = -.47758: K(39, 13) = -.47141: K(40, 13) = -.46496: K(41, 13) = -.
45828
9360  K(35, 14) = -.47413: K(36, 14) = -.47088: K(37, 14) = -.46711: K(38
, 14) = -.46286: K(39, 14) = -.45819: K(40, 14) = -.45314: K(41, 14) = -.
44777
9362  K(35, 15) = -.41253: K(36, 15) = -.41381: K(37, 15) = -.41442: K(38
, 15) = -.41441: K(39, 15) = -.41381: K(40, 15) = -.41265: K(41, 15) = -.
41097
9364  K(35, 16) = -.32479: K(36, 16) = -.33085: K(37, 16) = -.33623: K(38
, 16) = -.34092: K(39, 16) = -.34494: K(40, 16) = -.34831: K(41, 16) = -.
35105
9366  K(35, 17) = -.27782: K(36, 17) = -.28592: K(37, 17) = -.29335: K(38
, 17) = -.3001: K(39, 17) = -.30617: K(40, 17) = -.31159: K(41, 17) = -.3
1635
9368  K(35, 18) = -.0573: K(36, 18) = -7.195001E-02: K(37, 18) = -.0861:
K(38, 18) = -.09972: K(39, 18) = -.11279: K(40, 18) = -.1253: K(41, 18) =
 -.13725
9370  K(35, 19) = .32171: K(36, 19) = .30223: K(37, 19) = .2829: K(38, 19
) = .26376: K(39, 19) = .24484: K(40, 19) = .22617: K(41, 19) = .20777
9372  K(35, 20) = 1.09552: K(36, 20) = 1.07726: K(37, 20) = 1.05863: K(38
, 20) = 1.03965: K(39, 20) = 1.02036: K(40, 20) = 1.00079: K(41, 20) = .9
8096
9374  K(35, 21) = 1.97147: K(36, 21) = 1.96266: K(37, 21) = 1.95311: K(38
, 21) = 1.94283: K(39, 21) = 1.93186: K(40, 21) = 1.92023: K(41, 21) = 1.
90796
9376  K(35, 22) = 2.26862: K(36, 22) = 2.26409: K(37, 22) = 2.25872: K(38
, 22) = 2.25254: K(39, 22) = 2.24558: K(40, 22) = 2.23786: K(41, 22) = 2.
2294
9378  K(35, 23) = 2.91299: K(36, 23) = 2.91898: K(37, 23) = 2.92397: K(38
, 23) = 2.92799: K(39, 23) = 2.93107: K(40, 23) = 2.93324: K(41, 23) = 2.
9345
9380  K(35, 24) = 3.22641: K(36, 24) = 3.238: K(37, 24) = 3.24853: K(38,
24) = 3.25803: K(39, 24) = 3.26653: K(40, 24) = 3.27404: K(41, 24) = 3.28
```

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06
9382  K(35, 25) = 4.22473: K(36, 25) = 4.25569: K(37, 25) = 4.28545: K(38
, 25) = 4.31403: K(39, 25) = 4.34147: K(40, 25) = 4.36777: K(41, 25) = 4.
39296
9384  K(35, 26) = 5.25291: K(36, 26) = 5.30559: K(37, 26) = 5.35698: K(38
, 26) = 5.40711: K(39, 26) = 5.45598: K(40, 26) = 5.50362: K(41, 26) =
55005
9386  K(35, 27) = 6.64627: K(36, 27) = 6.73032: K(37, 27) = 6.81301: K(38
, 27) = 6.89435: K(39, 27) = 6.97435: K(40, 27) = 7.05304: K(41, 27) = 7.
13043
9388  K(35, 28) = 7.72024: K(36, 28) = 7.82954: K(37, 28) = 7.93744: K(38
, 28) = 8.04395: K(39, 28) = 8.1491: K(40, 28) = 8.25289: K(41, 28) = 8.3
5534
9390  K(35, 29) = 8.80779: K(36, 29) = 8.94335: K(37, 29) = 9.077501: K(3
8, 29) = 9.21023: K(39, 29) = 9.341581: K(40, 29) = 9.471541: K(41, 29) =
 9.60013
9392  K(35, 30) = 11.37334: K(36, 30) = 11.57284: K(37, 30) = 11.77092: K
(38, 30) = 11.96757: K(39, 30) = 12.1628: K(40, 30) = 12.35663: K(41, 30)
 = 12.54906


9394 FOR G = 42 TO 83
9396    FOR P = 0 TO 30
9398        K(G, P) = -K(G - 41, 30 - P)
9400    NEXT P
9402 NEXT G
9404 LOCATE 20, 20
9406 COLOR 12
9408 INPUT "PRESS ENTER TO PROCEED ..............", ERM
9410 CHAIN "FANINP"
9412 END


10000 REM   *************************************************************
10002 REM                            QUIT PROGRAM
10004 REM   *************************************************************
10006 OPEN "QUIT" FOR OUTPUT AS #3
10008 CLS
10010 COLOR 12
10012 FOR I = 1 TO 10: PRINT : NEXT I
10014 PRINT "                          PROGRAM TERMINATED BY USER"
10016 COLOR 7
10020 SYSTEM
11111 END
```

```
1   REM
2   REM   FFFFFFFF  AAA         NNN      NNN IIIIIIIII NNN      NNN PPPPPPPP
3   REM   FFF        AAAAA      NNNN     NNN    III    NNNN     NNN PPP     PPP
4   REM   FFF       AAA  AAA    NNNNNN   NNN    III    NNNNNN   NNN PPP     PPP
5   REM   FFFFFF AAA      AAA   NNN NNN NNN    III    NNN NNN NNN PPPPPPPP
6   REM   FFF    AAAAAAAAAAAA   NNN   NNNNNN    III    NNN   NNNNNN PPP
7   REM   FFF  AAA          AAA NNN     NNNN    III    NNN     NNNN PPP
8   REM   FFF  AAA          AAA NNN      NNN IIIIIIIII NNN      NNN PPP
9   REM
10 REM
11 REM Portions (c) Copyright
12 REM Microsoft Corporation
13 REM 1982-1988
14 REM All Rights Reserved
15 REM
16 REM Alluvial Fan - Michael Baker Jr., Inc. - JULY 1990
17 REM
18 REM
19 COMMON SHARED P(), K(), KG(), KO()
20 REM
30 ON KEY(1) GOSUB 10000
31 KEY(1) ON
35 REM
74 REM
99 REM


100 REM       ****************************************************************
102 REM                                INPUT DATA
104 REM       ****************************************************************
105 REM
106 CLS
107 COLOR 11: PRINT "Press F1 and then press ENTER to exit": COLOR 2
108 PRINT : PRINT
109 PRINT "                    ENTER THE NAME OF THE ALLUVIAL FAN    "
110 INPUT "                    ", B$
114 CLS : MULT = 0
115 COLOR 11: PRINT "Press F1 and then press ENTER to exit": COLOR 2
116 FOR I = 1 TO 4: PRINT : NEXT I
```

```
120 REM      ********************************************************************
122 REM                          INPUT MULTIPLE-CHANNEL DATA
124 REM      *******************************************************************
126 REM
128 PRINT "DO YOU WISH TO COMPUTE ZONE BOUNDARIES"
130 INPUT "FOR MULTIPLE CHANNELS (Y/N)"; MULT$
132 IF INSTR(MULT$, "Y") = 0 AND INSTR(MULT$, "y") = 0 THEN GOTO 179
134 MULT = 1
136 CLS : COLOR 11: PRINT "Press F1 and then press ENTER to exit": COLOR
2
137 FOR I = 1 TO 4: PRINT : NEXT I
138 INPUT "ENTER SLOPE OF ALLUVIAL FAN     ", SLOPE
140 CLS : COLOR 11: PRINT "Press F1 and then press ENTER to exit": COLOR
2
141 FOR I = 1 TO 4: PRINT : NEXT I
142 IF SLOPE > 1 THEN PRINT "SLOPE ("; SLOPE; ") IS TOO LARGE" ELSE GOTO
146
144 GOTO 148
146 IF SLOPE < .000001 THEN PRINT "SLOPE ("; SLOPE; ") IS TOO SMALL" ELSE
 GOTO 152
148 FOR I = 1 TO 4: PRINT : NEXT I
150 GOTO 138
152 CLS : COLOR 11: PRINT "Press F1 and then press ENTER to exit": COLOR
2
153 FOR I = 1 TO 4: PRINT : NEXT I
154 PRINT "SLOPE ="; SLOPE
156 FOR I = 1 TO 4: PRINT : NEXT I
158 INPUT "ENTER ROUGHNESS COEFFICIENT (N-VALUE)     ", NVALUE
160 CLS : COLOR 11: PRINT "Press F1 and then press ENTER to exit": COLOR
2
161 FOR I = 1 TO 4: PRINT : NEXT I
162 IF NVALUE > 1 THEN PRINT "N-VALUE ("; NVALUE; ") IS TOO LARGE" ELSE G
OTO 166
164 GOTO 168
166 IF NVALUE < .001 THEN PRINT "N-VALUE ("; NVALUE; ") IS TOO SMALL" ELS
E GOTO 172
168 FOR I = 1 TO 4: PRINT : NEXT I
170 GOTO 158
172 CLS : COLOR 11: PRINT "Press F1 and then press ENTER to exit": COLOR
2
173 FOR I = 1 TO 4: PRINT : NEXT I
174 PRINT "MULTIPLE CHANNEL PARAMETERS :"
176 PRINT "                          SLOPE ="; SLOPE
178 PRINT "                        N-VALUE ="; NVALUE
179 COLOR 11
180 IF INSTR(MULT$, "Y") = 0 AND INSTR(MULT$, "y") = 0 THEN CLS
181 IF INSTR(MULT$, "Y") = 0 AND INSTR(MULT$, "y") = 0 THEN PRINT "Press
F1 and then press ENTER to exit"
182 COLOR 2
183 FOR I = 1 TO 4: PRINT : NEXT I
```

```
184 REM
186 REM      ***************************************************************
188 REM                          INPUT AVULSION FACTOR
190 REM      ***************************************************************
192 REM
194 INPUT "ENTER AVULSION FACTOR    ", AVUL
196 IF AVUL = 0 THEN AVUL = 1


199 REM
200 REM      ***************************************************************
202 REM            CHOOSE OPTION FOR DEFINING FLOOD FREQUENCY CURVE
204 REM      ***************************************************************
205 REM
206 CLS : COLOR 11: PRINT "Press F1 and then press ENTER to exit": COLOR
2
207 FOR I = 1 TO 4: PRINT : NEXT I
208 PRINT "YOU MAY DEFINE THE FLOOD FREQENCY CURVE BY:"
210 PRINT : PRINT
212 PRINT "(1)....ENTERING THE MEAN, STANDARD DEVIATION, AND SKEW COEFFIC
IENT"
214 PRINT "        OF THE PEARSON TYPE-III DISTRIBUTION"
216 PRINT
218 PRINT "(2)....ENTERING (AT LEAST THREE) PAIRS OF RETURN INTERVALS AND
 DISCHARGES"
220 PRINT : PRINT
222 INPUT "PLEASE ENTER OPTION NUMBER (1 OR 2)  ", PDFOPT
224 IF PDFOPT = 1 THEN GOTO 300
226 IF PDFOPT = 2 THEN GOTO 400
228 CLS : COLOR 11: PRINT "Press F1 and then press ENTER to exit": COLOR
2
229 FOR I = 1 TO 4: PRINT : NEXT I
230 PRINT "SORRY, THERE ARE ONLY TWO OPTIONS."
232 PRINT PDFOPT; " IS NOT ONE OF THEM."
234 FOR I = 1 TO 4: PRINT : NEXT I
236 GOTO 208
```

```
299 REM
300 REM     *****************************************************************
302 REM                    OPTION (1)  ENTER STATISTICS
304 REM     *****************************************************************
305 REM
306 CLS : COLOR 11: PRINT "Press F1 and then press ENTER to exit": COLOR
2: FOR I = 1 TO 4: PRINT : NEXT I
307 INPUT "                ENTER MEAN  ", MU
308 PRINT
309 INPUT "  ENTER STANDARD DEVIATION  ", SIGMA
310 PRINT
311 IF SIGMA < .1 THEN GOTO 330
312 INPUT "    ENTER SKEW COEFFICIENT  ", SKEW
313 IF SKEW > 4.1 THEN GOTO 340
314 IF SKEW < -4.1 THEN GOTO 350
318 IF SKEW < 0 THEN SK = 4.1 - SKEW ELSE SK = SKEW
320 G = INT(10 * (SK + .05))
322 SKEW = G / 10
324 IF SKEW > 4.1 THEN SKEW = 4.1 - SKEW
326 GOTO 700
330 PRINT "SORRY, STANDARD DEVIATION MUST BE GREATER THAN 0.1"
332 INPUT "RE-ENTER STANDARD DEVIATION  ", SIGMA
334 GOTO 310
340 PRINT
341 PRINT "SORRY, SKEW CANNOT BE GREATER THAN 4.1"
342 INPUT " RE-ENTER SKEW COEFFICIENT  ", SKEW
343 GOTO 313
350 PRINT
351 PRINT "SORRY, SKEW CANNOT BE LESS THAN -4.1"
352 GOTO 342
```

```
399 REM
400 REM     ****************************************************************
402 REM                       OPTION (2)  ENTER PAIRS OF DATA
404 REM     ****************************************************************
405 REM
406 CLS : COLOR 11: PRINT "Press F1 and then press ENTER to exit": COLOR
2
408 FOR I = 1 TO 4: PRINT : NEXT I
410 PRINT "HOW MANY PAIRS OF DISCHARGES AND"
412 INPUT "RECURRENCE INTERVALS DO YOU WISH TO ENTER"; NOF: NOF = INT(NOF
)
414 IF NOF >= 3 THEN GOTO 424
416 CLS : COLOR 11:   PRINT "Press F1 and then press ENTER to exit": PRIN
T : PRINT : COLOR 2
418 PRINT "SORRY, YOU MUST ENTER AT LEAST THREE PAIRS OF DATA."
420 PRINT NOF; " IS LESS THAN THREE."
422 GOTO 408
423 REM
424 PRINT : PRINT
426 DIM RET(NOF), Q(NOF), Y(NOF), KK(NOF), PIN(NOF)
428 FOR I = 1 TO NOF
430     PRINT "ENTER RECURRENCE INTERVAL NUMBER"; I;
432     INPUT "    ", RET(I): GOSUB 1000
434     PRINT : PRINT
436     PRINT "      ENTER"; RET(I); "- YEAR DISCHARGE";
438     INPUT "                          ", Q(I): GOSUB 2000: GOSUB 3000
440     GOSUB 480
442 NEXT
443 I = NOF
444 INPUT "                DO YOU WISH TO CHANGE ANY DATA (Y/N)"; CHGDAT$
446 IF INSTR(CHGDAT$, "Y") = 0 AND INSTR(CHGDAT$, "y") = 0 GOTO 500
448 PRINT : PRINT
450 PRINT "WHICH PAIR OF DATA ( 1 -"; NOF; ") DO YOU WISH TO";
452 INPUT " CHANGE"; CHGDAT: CHGDAT = INT(CHGDAT)
454 IF (CHGDAT >= 1) AND (CHGDAT <= NOF) THEN GOTO 464
456 CLS : COLOR 11:   PRINT "Press F1 and then press ENTER to exit": PRIN
T : PRINT : COLOR 2
458 PRINT "SORRY, THERE IS NO DATA PAIR NUMBER "; CHGDAT
460 PRINT : PRINT
462 GOSUB 482: GOTO 444
464 GOSUB 3066
476 GOSUB 480
478 GOTO 444
480 CLS : COLOR 11: PRINT "Press F1 and then press ENTER to exit": COLOR
2
481 FOR K = 1 TO 4: PRINT : NEXT K
482 PRINT "             DATA PAIR    RECURRENCE INTERVAL  DISCHARGE"
484 PRINT
486 FOR J = 1 TO I
488     PRINT USING "##################"; J; RET(J); Q(J)
490 NEXT
492 PRINT : PRINT : PRINT
494 RETURN
```

```
499 REM
500 REM      ***************************************************************
502 REM                           FIND SKEW THAT GIVES BEST FIT
504 REM      ***************************************************************
505 REM
506 LEFT = -4.1: RIGHT = 4.1
508 IF RIGHT - LEFT < .12 GOTO 528
510 MID = INT(10 * (RIGHT + LEFT) / 2 + .001) / 10
512 RMID = MID + .1
514 IF MID < 0 THEN G = 10 * (4.1 - MID) ELSE G = 10 * MID
516 GOSUB 600: MIDR = R
518 IF RMID < 0 THEN G = 10 * (4.1 - RMID) ELSE G = 10 * RMID
520 GOSUB 600: RMIDR = R
522 IF RMIDR < MIDR GOTO 526
524 LEFT = MID: GOTO 508
526 RIGHT = MID: GOTO 508
528 IF RMIDR > MIDR THEN SKEW = RMID ELSE SKEW = MID
530 IF SKEW < 0 THEN G = 10 * (4.1 - SKEW) ELSE G = 10 * SKEW
532 GOSUB 600
534 RMAX = R: MU = MEAN: SIGMA = STDV
550 GOTO 700


600 REM
602 REM      ***************************************************************
604 REM                      GIVEN INPUT DATA (OPTION 2), COMPUTE
606 REM                      LOG(J) AND CORRESPONDING DEVIATE KK(J)
608 REM      ***************************************************************
610 REM
612 FOR J = 1 TO NOF
614    Y(J) = LOG(Q(J)) / LOG(10)
616    PIN(J) = 1 / RET(J)
618    FOR I = 1 TO 30
620        IF PIN(J) < P(I) THEN GOTO 626
622        N = I: M = N - 1
624        GOTO 628
626    NEXT I
628    KK(J) = K(G, M) + (PIN(J) - P(M)) * (K(G, N) - K(G, M)) / (P(N) -
P(M))
630 NEXT J
```

```
632 REM
634 REM       ****************************************************************
636 REM            GIVEN NOF PAIRS OF DATA, Y(J) AND KK(J), COMPUTE
638 REM            MEAN (Y-INTERCEPT), STANDARD DEVIATION (SLOPE),
640 REM            AND CORRELATION COEFICIENT BY METHOD OF LEAST SQUARES
642 REM       ****************************************************************
644 REM
646 MK = 0: MY = 0: A = 0: B = 0: C = 0
648 FOR I = 1 TO NOF
650    MK = MK + KK(I)
652    MY = MY + Y(I)
654 NEXT
656 MEANK = MK / NOF
658 MEANY = MY / NOF
660 FOR I = 1 TO NOF
662    A = A + (KK(I) - MEANK) ^ 2
664    B = B + (Y(I) - MEANY) ^ 2
666    C = C + (KK(I) - MEANK) * (Y(I) - MEANY)
668 NEXT
670 SIGK = SQR(A / NOF): SIGY = SQR(B / NOF)
672 R = C / NOF / SIGK / SIGY
674 STDV = R * SIGY / SIGK
676 MEAN = MEANY - STDV * MEANK
678 RETURN


700 REM
702 REM       ****************************************************************
704 REM                   CHECK VALUE OF 100-YEAR FLOOD DISCHARGE
706 REM       ****************************************************************
708 REM
709 IF (SIGMA * K(G, 25) + MU) > 6 THEN GOTO 714
710 Q100 = 10 ^ (SIGMA * K(G, 25) + MU)
712 IF Q100 < 500000! THEN GOTO 730
714 CLS : COLOR 12
716 FOR I = 1 TO 8: PRINT : NEXT I
718 PRINT "                    Q100 > 500000 cfs... PROGRAM TERMINATED":
PRINT : PRINT : PRINT
720 COLOR 15, 0: PRINT "                             Press ENTER to start over
": PRINT
721 INPUT "                    or press F1 and then ENTER to exit", MJM
722 COLOR 12
723 GOTO 100
730 IF Q100 > 50 THEN GOTO 800
732 CLS : COLOR 12
734 FOR I = 1 TO 8: PRINT : NEXT I
736 PRINT "                    Q100 < 50 cfs ....... PROGRAM TERMINATED":
 PRINT : PRINT : PRINT
738 GOTO 720
```

```
800 REM
802 REM      ***************************************************************
804 REM                         WRITE INPUT TO INPUT FILE
806 REM      ***************************************************************
808 REM
812 PRINT #1, B$
814 PRINT #1, MULT, SLOPE, NVALUE
816 PRINT #1, PDFOPT, AVUL
818 PRINT #1, NOF
820 FOR I = 1 TO NOF
822     PRINT #1, RET(I), Q(I), KK(I)
824 NEXT I
826 PRINT #1, MU, SIGMA, SKEW, RMAX
828 FOR I = 0 TO 30
830     PRINT #1, P(I)
832 NEXT I
834 FOR I = 0 TO 30
836     KO(I) = K(0, I): PRINT #1, KO(I)
838     KG(I) = K(G, I): PRINT #1, KG(I)
840 NEXT I
842 PRINT #1, Q100
850 SYSTEM
```

```
1000 REM
1001 REM      ****************************************************************
1002 REM                            CHECK RECURRENCE INTERVAL
1004 REM      ****************************************************************
1005 REM
1006 IF RET(I) < 1.001 GOTO 1012
1008 IF RET(I) > 1000 GOTO 1024
1010 GOTO 1030
1012 PRINT
1014 PRINT "SORRY, RECURRENCE INTERVAL CANNOT BE LESS THAN 1.001 YEAR"
1016 PRINT
1018 PRINT "RE-ENTER RECURRENCE INTERVAL";
1020 INPUT "  ", RET(I)
1022 GOTO 1000
1024 PRINT
1026 PRINT "SORRY, RECURRENCE INTERVAL CANNOT BE GREATER THAN 1000 YEARS"
1028 GOTO 1016
1030 IF I = 1 THEN RETURN
1032 FOR J = 1 TO I - 1
1034     IF RET(I) = RET(J) GOTO 1040
1036 NEXT J
1038 RETURN
1040 PRINT
1042 PRINT "SORRY, EACH RECURRENCE INTERVAL MAY BE ENTERED ONLY ONCE"
1044 PRINT RET(I); "IS ALSO RECURRENCE INTERVAL NUMBER "; J
1046 GOTO 1016


2000 REM
2001 REM      ****************************************************************
2002 REM                   CHECK THAT DISCHARGE IS GREATER THAN ZERO
2004 REM      ****************************************************************
2005 REM
2006 IF Q(I) > 0 THEN RETURN
2008 PRINT
2010 PRINT "DISCHARGE MUST BE GREATER THAN ZERO"
2012 PRINT
2014 PRINT "RE-ENTER"; RET(I); "- YEAR DISCHARGE";
2016 INPUT "                          ", Q(I)
2018 GOTO 2000
```

```
3000 REM
3001 REM     ************************************************************
3002 REM                    ORDER DATA BY RECURRENCE INTERVAL
3003 REM               AND CHECK THAT DISCHARGES DO NOT DECREASE
3004 REM     ************************************************************
3005 IF I = 1 THEN RETURN
3006 FOR J = 1 TO I - 1
3008     IF RET(I) < RET(J) GOTO 3014
3010 NEXT J
3012 GOTO 3024
3014 RET = RET(I): Q = Q(I)
3016 FOR K = I TO J + 1 STEP -1
3018     RET(K) = RET(K - 1): Q(K) = Q(K - 1)
3020 NEXT K
3022 RET(J) = RET: Q(J) = Q
3024 IF Q(J) < Q(J - 1) GOTO 3030
3025 IF J = I THEN RETURN
3026 IF J < I AND Q(J) > Q(J + 1) GOTO 3030
3028 RETURN
3030 CLS : COLOR 11:   PRINT "Press F1 and then press ENTER to exit": PRI
NT : PRINT : COLOR 2
3032 PRINT "SORRY, DISCHARGE VALUES CANNOT DECREASE WITH INCREASING RECUR
RENCE INTERVALS"
3034 PRINT
3036 IF Q(J) < Q(J - 1) THEN L = J ELSE L = J + 1
3038 PRINT "THE"; RET(L); "- YEAR DISCHARGE ("; Q(L); " CFS ) IS LESS THA
N"
3039 PRINT "THE"; RET(L - 1); "- YEAR DISCHARGE ("; Q(L - 1); " CFS )"
3040 PRINT
3042 PRINT "             DATA PAIR   RECURRENCE INTERVAL  DISCHARGE"
3044 PRINT
3046 PRINT USING "##################"; L - 1; RET(L - 1); Q(L - 1)
3048 PRINT USING "##################"; L; RET(L); Q(L)
3050 PRINT : PRINT
3052 PRINT "WHICH PAIR OF DATA DO YOU WISH TO CHANGE -- "
3054 PRINT "DATA PAIR NUMBER"; L - 1; "OR DATA PAIR NUMBER"; L; "?";
3056 INPUT "  ", CHGDAT
3058 IF CHGDAT = L - 1 OR CHGDAT = L GOTO 3066
3060 CLS : COLOR 11:   PRINT "Press F1 and then press ENTER to exit": PRI
NT : PRINT : COLOR 2
3062 PRINT "SORRY, YOU ONLY HAVE TWO CHOICES. "; CHGDAT; "IS NOT ONE OF T
HEM."
3064 GOTO 3040
3066 IF CHGDAT = I GOTO 3074
3068 FOR K = CHGDAT TO I - 1
3070     RET(K) = RET(K + 1): Q(K) = Q(K + 1)
3072 NEXT K
3074 PRINT
3076 PRINT "RE-ENTER RECURRENCE INTERVAL NUMBER"; CHGDAT;
3078 INPUT "  ", RET(I): GOSUB 1000
3080 PRINT
3082 PRINT "ENTER"; RET(I); "- YEAR DISCHARGE";
3084 INPUT "                         ", Q(I)
3085 GOSUB 2000
3086 GOTO 3000
```

```
9999  REM
10000 REM    ****************************************************************
10002 REM                                  QUIT PROGRAM
10004 REM    ****************************************************************
10006 OPEN "QUIT" FOR OUTPUT AS #3
10008 CLS
10010 COLOR 12
10012 FOR I = 1 TO 10: PRINT : NEXT I
10014 PRINT "                                 PROGRAM TERMINATED BY USER"
10016 COLOR 7
10020 SYSTEM
11111 END
```

```
1 REM
2 REM    FFFFFFF  AAA       NNN      NNN   RRRRRRRR   UUU      UUU NNN       NN
3 REM    FFF       AAAAA     NNNN     NNN   RRR      RRR UUU      UUU NNNN      NNN
4 REM    FFF      AAA  AAA   NNNNNN   NNN   RRR      RRR UUU      UUU NNN NNN NNN
5 REM    FFFFFF AAA      AAA NNN NNN NNN   RRRRRRRR   UUU      UUU NNN   NNNNNN
7 REM    FFF    AAAAAAAAAAAA NNN     NNNN   RRR      RRR UUU      UUU NNN     NNNN
8 REM    FFF    AAA       AAA NNN      NNN   RRR      RRR  UUUUUUU   NNN      NNN
9 REM
10 REM
11 REM     ************************************************************
12 REM                SET UP CHECK FOR GRAPHICS CAPABILITIES
13 REM     ************************************************************
14 ON ERROR GOTO 17
15 SCREEN 2:   PRESET (1, 1)
16 GOTO 19
17 GRCHK = 99
18 RESUME NEXT
19 ON ERROR GOTO 0
20 KEY OFF
100 REM
102 REM    ************************************************************
104 REM                GET INPUT FROM INPUT FILE - FAN.IN
106 REM    ************************************************************
108 REM
120 DIM P(30), KG(30), KO(30)
122 OPEN "FAN.IN" FOR INPUT AS #1
124 OPEN "FAN.OUT" FOR OUTPUT AS #2
126 INPUT #1, B$
128 A$ = SPACE$(INT((71 - LEN(B$)) / 2)) + B$
130 INPUT #1, MULT, SLOPE, NVALUE
132 INPUT #1, PDFOPT, AVUL
134 INPUT #1, NOF
136 DIM RET(NOF), Q(NOF), KK(NOF)
138 FOR I = 1 TO NOF
140      INPUT #1, RET(I), Q(I), KK(I)
142 NEXT I
144 INPUT #1, MU, SIGMA, SKEW, RMAX
146 FOR I = 0 TO 30
148      INPUT #1, P(I)
150 NEXT I
152 FOR I = 0 TO 30
154      INPUT #1, KO(I), KG(I)
156 NEXT I
158 INPUT #1, Q100
850 GOSUB 6200
852 IF GRCHK < 44 THEN GOSUB 8200
854 IF GRCHK < 44 THEN GOTO 1000
900 CLS
904 LOCATE 4, 16
906 PRINT "SYSTEM NOT COMPATIBLE WITH GRAPHICS SUBROUTINES"
910 LOCATE 20, 30
911 COLOR 18
912 PRINT "PROGRAM RUNNING ..."
```

```
1000 REM
1002 REM     *************************************************************
1004 REM     *************************************************************
1006 REM     ********************    MAIN PROGRAM    *********************
1008 REM     *************************************************************
1010 REM     *************************************************************
1012 REM
1014 REM
1016 REM     *************************************************************
1018 REM     TRANSFORM RANDOM VARIABLE (Y TO Z):
1019                                              MUY = MU: SIGMAY = SIGMA
1020                                                           GOSUB 2000
1022 REM     *************************************************************
1024 REM
1026                                                           GOSUB 6400
1028 REM
1030 REM     *************************************************************
1032 REM     ASSIGN DISCHARGES AND CALCULATE ALLUVIAL FAN
1034 REM     WIDTHS FOR DEPTH ZONE BOUNDARIES:
1035                                                           GOSUB 2200
1036 REM     *************************************************************
1038 REM
1040 REM     *************************************************************
1042 REM     ASSIGN DISCHARGES AND CALCULATE ALLUVIAL FAN
1044 REM     WIDTHS FOR VELOCITY ZONE BOUNDARIES:
1045                                                           GOSUB 2400
1046 REM     *************************************************************
1048 REM
1050                                                           GOSUB 6600
1051                                       IF GRCHK < 44 THEN GOSUB 8400
1052 REM
1054 REM     *************************************************************
1056 REM     *************************************************************
1058 REM     ********************    END OF RUN    ***********************
1060 REM     *************************************************************
1062 REM     *************************************************************
1064 REM
1066 REM
1068 REM     *************************************************************
1069 REM     OPTION TO VIEW AND/OR PRINT OUTPUT DATA:
1070                                                           GOSUB 9000
1072 REM     *************************************************************
1074 REM
1098 SYSTEM
```

```
1999 REM
2000 REM      ***************************************************************
2002 REM                       TRANSFORM RANDOM VARIABLE
2004 REM     *****************************************************************
2005 REM
2006 IF SKEW = 0 THEN GOTO 2022
2008 SHAPE = 4 / SKEW / SKEW
2010 SCALE = 2 / SKEW / SIGMAY
2012 TRANS = MUY - 2 * SIGMAY / SKEW
2014 MUZ = TRANS + SHAPE / (SCALE - .92)
2016 SIGMAZ = SQR(SHAPE / (SCALE - .92) / (SCALE - .92))
2018 CNST = EXP(.92 * TRANS) * (SCALE / (SCALE - .92)) ^ SHAPE
2020 GOTO 2028
2022 MUZ = MUY + .92 * SIGMAY * SIGMAY
2024 SIGMAZ = SIGMAY
2026 CNST = EXP(.92 * MUY + .42 * SIGMAY * SIGMAY)
2028 RETURN


2200 REM
2202 REM      ****************************************************************
2204 REM          SUBROUTINE TO COMPUTE CONTOUR WIDTHS FOR DEPTH ZONES
2206 REM      ****************************************************************
2208 REM
2210 DH = INT((Q100 / 274) ^ .4 + 1)
2212 DIM PHYSING(DH), PHZSING(DH), HSING(DH), QHSING(DH), WHSING(DH)
2214 DIM PHYMULT(DH), PHZMULT(DH), HMULT(DH), QHMULT(DH), WHMULT(DH)
2216 H = .5: NH = 0
2218 IF MULT = 2 THEN GOSUB 2300 ELSE Q = 274.3902 * H ^ 2.5
2220 IF Q > Q100 THEN GOTO 2262
2222 IF MULT = 2 THEN NHMULT = NH ELSE NHSING = NH
2224 NH = NH + 1
2226 Y = LOG(Q) / LOG(10)
2228 MU = MUY: SIGMA = SIGMAY: GOSUB 4000
2230 IF MULT = 2 THEN PHYMULT(NH) = P ELSE PHYSING(NH) = P
2232 MU = MUZ: SIGMA = SIGMAZ: GOSUB 4000
2234 IF MULT = 2 THEN PHZMULT(NH) = P ELSE PHZSING(NH) = P
2236 IF MULT = 2 THEN GOTO 2244
2238 HSING(NH) = H: QHSING(NH) = INT((INT(Q * 10) + 5) / 10)
2240 WHSING(NH) = AVUL * CNST * PHZSING(NH) * 940.8059
2242 GOTO 2248
2244 HMULT(NH) = H: QHMULT(NH) = INT((INT(Q * 10) + 5) / 10)
2246 WHMULT(NH) = AVUL * CNST * PHZMULT(NH) * 3575.0624#
2248 IF MULT = 2 THEN SORM = 35.750624# ELSE SORM = 9.408059
2250 IF MULT = 2 THEN W = WHMULT(NH) ELSE W = WHSING(NH)
2252 GOSUB 4200
2254 IF MULT = 2 THEN WHMULT(NH) = W ELSE WHSING(NH) = W
2256 H = H + 1
2258 IF MULT = 2 THEN NHMULT = NH ELSE NHSING = NH
2260 GOTO 2218
2262 MULT = MULT + 1
2264 IF MULT = 2 THEN GOTO 2216
2266 RETURN
```

```
2300 REM
2302 REM      ************************************************************
2304 REM            SUBROUTINE TO COMPUTE Q(H) FOR MULTIPLE CHANNELS
2306 REM      ************************************************************
2308 REM
2310 QL = 0: QH = Q100: QG = Q100: HL = 0
2312 HG = 9.220001E-02 * NVALUE ^ .6 * QG ^ .36 / SLOPE ^ .3 + .00143 * S
LOPE ^ .6 * QG ^ .48 / NVALUE ^ 1.2
2314 IF QH - QL < .01 THEN GOTO 2330
2315 IF QL / QH > .99999 THEN GOTO 2330
2316 IF (QG = Q100) AND (HG < H) THEN GOTO 2328
2318 IF QG = Q100 THEN HH = HG
2320 IF HG > H THEN HH = HG ELSE HL = HG
2322 IF HG > H THEN QH = QG ELSE QL = QG
2324 QG = (QH + QL) / 2
2326 GOTO 2312
2328 Q = 2 * Q100: GOTO 2332
2330 Q = (QH + QL) / 2
2332 RETURN
```

```
2400 REM
2402 REM      ****************************************************************
2404 REM        SUBROUTINE TO COMPUTE CONTOUR WIDTHS FOR VELOCITY ZONES
2406 REM      ****************************************************************
2408 REM
2410 DV = INT((Q100 / .12) ^ .2 - 2)
2412 DIM PVYSING(DV), PVZSING(DV), VSING(DV), QVSING(DV), WVSING(DV)
2414 DIM PVYMULT(DV), PVZMULT(DV), VMULT(DV), QVMULT(DV), WVMULT(DV)
2416 GOTO 2430
2417 IF MULT = 4 AND QHMULT(1) < 1 THEN GOTO 2474
2418 VMAX = .3033 * SLOPE ^ .3 * Q100 ^ .24 / NVALUE ^ .6
2420 VMIN = .3033 * SLOPE ^ .3 * QHMULT(1) ^ .24 / NVALUE ^ .6
2422 VT = .5
2424 IF VT > VMIN THEN GOTO 2430
2426 VT = VT + 1
2428 GOTO 2424
2430 NV = 0: IF MULT = 4 THEN V = VT ELSE V = 3.5
2432 IF MULT = 4 THEN Q = 144.1315 * NVALUE ^ 2.5 * V ^ (25 / 6) / SLOPE
^ 1.25 ELSE Q = .1289 * V ^ 5
2434 IF Q > Q100 THEN GOTO 2474
2436 NV = NV + 1
2438 Y = LOG(Q) / LOG(10)
2440 MU = MUY: SIGMA = SIGMAY: GOSUB 4000
2442 IF MULT = 4 THEN PVYMULT(NV) = P ELSE PVYSING(NV) = P
2444 MU = MUZ: SIGMA = SIGMAZ: GOSUB 4000
2446 IF MULT = 4 THEN PVZMULT(NV) = P ELSE PVZSING(NV) = P
2448 IF MULT = 4 THEN GOTO 2456
2450 VSING(NV) = V: QVSING(NV) = INT((INT(Q * 10) + 5) / 10)
2452 WVSING(NV) = AVUL * CNST * PVZSING(NV) * 940.8059
2454 GOTO 2460
2456 VMULT(NV) = V: QVMULT(NV) = INT((INT(Q * 10) + 5) / 10)
2458 WVMULT(NV) = AVUL * CNST * PVZMULT(NV) * 3575.0624#
2460 IF MULT = 4 THEN SORM = 35.750624# ELSE SORM = 9.408059
2462 IF MULT = 4 THEN W = WVMULT(NV) ELSE W = WVSING(NV)
2464 GOSUB 4200
2466 IF MULT = 4 THEN WVMULT(NV) = W ELSE WVSING(NV) = W
2468 V = V + 1
2470 IF MULT = 4 THEN NVMULT = NV ELSE NVSING = NV
2472 GOTO 2432
2474 MULT = MULT + 1
2476 IF MULT = 4 THEN GOTO 2417
2478 RETURN
```

```
4000 REM
4002 REM      *******************************************************************
4004 REM                   SUBROUTINE TO COMPUTE PROBABILITY
4006 REM           GIVEN LOG(Q), MEAN, STANDARD DEVIATION, AND SKEW
4008 REM      *******************************************************************
4010 REM
4012 K = (Y - MU) / SIGMA
4014 IF K > KG(0) THEN GOTO 4018
4016 P = 1: GOTO 4032
4018 FOR I = 1 TO 30
4020    IF K > KG(I) THEN GOTO 4026
4022    N = I: M = N - 1
4024    GOTO 4030
4026 NEXT
4028 P = 0: GOTO 4032
4030 P = P(N) + (P(M) - P(N)) * (K - KG(N)) / (KG(M) - KG(N))
4032 RETURN


4200 REM
4202 REM      *******************************************************************
4204 REM        SUBROUTINE TO ADJUST WIDTH FOR CHANNEL WIDTH > FAN WIDTH
4206 REM      *******************************************************************
4208 REM
4209 WI = W: NA = 0
4210 PROB = P
4212 QW = (W / SORM) ^ 2.5
4214 Y = LOG(QW) / LOG(10)
4216 MU = MUY: SIGMA = SIGMAY: GOSUB 4000
4218 PYQW = P
4220 MU = MUZ: SIGMA = SIGMAZ: GOSUB 4000
4222 PZQW = P
4224 PRB = AVUL * CNST * SORM / W * (PROB - PZQW) + PYQW
4225 IF PRB > .01 THEN NA = W
4226 IF PRB < .01 THEN WI = W
4227 WNEW = 100 * PRB * W
4229 IF ABS(WNEW - W) < 1 OR WI - NA < 1 THEN GOTO 4234
4230 IF ABS(WNEW - W) >= WI - NA THEN W = (WI + NA) / 2 ELSE W = WNEW
4232 GOTO 4212
4234 RETURN
```

```
6200 REM
6202 REM        ******************************************************************
6204 REM                           FLOOD FREQUENCY OUTPUT
6206 REM        ******************************************************************
6208 REM
6210 PRINT #2, CHR$(12)
6212 PRINT #2, A$
6214 FOR I = 1 TO 2: PRINT #2, : NEXT
6216 PRINT #2, USING "                         AVULSION FACTOR = #.####"; A
VUL
6218 FOR I = 1 TO 4: PRINT #2, : NEXT
6220 IF PDFOPT = 2 GOTO 6226
6222 PRINT #2, "    FLOOD FREQUENCY CURVE DEFINED BY MEAN, STANDARD DEVIA
TION, AND SKEW"
6224 GOTO 6242
6226 PRINT #2, "    FLOOD FREQUENCY CURVE DEFINED BY LEAST-SQUARES FIT OF
 DATA"
6228 FOR I = 1 TO 2: PRINT #2, : NEXT
6230 PRINT #2, "    RETURN INTERVAL     INPUT DISCHARGE   BEST FIT DISCHA
RGE"
6232 PRINT #2, "        (YEARS)               (CFS)               (CFS)"
6234 PRINT #2,
6236 FOR K = 1 TO NOF
6238    PRINT #2, USING "        ####               ######
######"; RET(K); Q(K); 10 ^ (SIGMA * KK(K) + MU)
6240 NEXT
6242 FOR I = 1 TO 2: PRINT #2, : NEXT
6244 PRINT #2, USING "                                  MEAN = ##.####"
; MU
6246 PRINT #2, USING "                        STANDARD DEVIATION = ##.######"
; SIGMA
6248 PRINT #2, USING "                                  SKEW = ##.#"; SKE
W
6250 FOR I = 1 TO 4: PRINT #2, : NEXT
6252 PRINT #2, "    SUMMARY OF DISCHARGES:"
6254 PRINT #2,
6256 PRINT #2, USING "                     10-YEAR DISCHARGE = ######"; 1
0 ^ (SIGMA * KG(20) + MU)
6258 PRINT #2, USING "                     50-YEAR DISCHARGE = ######"; 1
0 ^ (SIGMA * KG(24) + MU)
6260 PRINT #2, USING "                    100-YEAR DISCHARGE = ######"; 1
0 ^ (SIGMA * KG(25) + MU)
6262 PRINT #2, USING "                    500-YEAR DISCHARGE = ######"; 1
0 ^ (SIGMA * KG(27) + MU)
6264 FOR I = 1 TO 4: PRINT #2, : NEXT
6265 IF SIGMA * SKEW > 2.1 THEN GOTO 6270
6266 RETURN
6270 PRINT #2, "                STANDARD DEVIATION TIMES SKEW GREATER T
HAN 2.1"
6272 PRINT #2, "                       WIDTHS CANNOT BE COMPUTED"
6274 PRINT #2,
6276 PRINT #2, "                          PROGRAM TERMINATED"
6278 COLOR 10
```

```
6279 FOR I = 1 TO 4: PRINT : NEXT I
6280 PRINT "                    STANDARD DEVIATION TIMES SKEW GREATER THAN
2.1"
6282 PRINT "                                  WIDTHS CANNOT BE COMPUTED"
6284 PRINT
6286 PRINT : PRINT : PRINT : COLOR 12
6290 PRINT "                                    PROGRAM TERMINATED"
6292 FOR I = 1 TO 4: PRINT : NEXT I
6294 INPUT "               DO YOU WISH TO VIEW THE FLOOD FREQENCY DATA (Y/N
)?  ", FFD$
6295 IF INSTR(FFD$, "Y") = 0 AND INSTR(FFD$, "y") = 0 THEN GOTO 9900
6296 MUY = MU: SIGMAY = SIGMA
6298 COLOR 10: GOTO 9100


6400 REM
6402 REM      ****************************************************************
6404 REM                               TRANSFORMATION OUTPUT
6406 REM      ****************************************************************
6408 REM
6412 IF SKEW = 0 GOTO 6420
6414 AAAA = -.92 * TRANS / (SCALE - .92): BBBB = SCALE / (SCALE - .92)
6416 PRINT #2, USING "      STATISTICS AFTER TRANSFORMATION OF Y=LOG(Q) T
O Z=#.####+#.#### LOG(Q)"; -.92 * TRANS / (SCALE - .92); SCALE / (SCALE -
 .92)
6418 GOTO 6424
6420 AAAA = .92 * SIGMAY * SIGMAY: BBBB = 1!
6422 PRINT #2, USING "      STATISTICS AFTER TRANSFORMATION OF Y=LOG(Q) T
O Z=#.####+LOG(Q)"; .92 * SIGMAY * SIGMAY
6424 PRINT #2,
6426 PRINT #2, USING "                              MEAN OF Z = ##.######"
; MUZ
6428 PRINT #2, USING "                       STANDARD DEVIATION = ##.######"
; SIGMAZ
6430 PRINT #2, USING "                                     SKEW = ##.######"
; SKEW
6432 PRINT #2, USING "                  TRANSFORMATION CONSTANT = ##.######"
; CNST
6434 PRINT #2, CHR$(12)
6436 RETURN
```

```
6600 REM
6602 REM      ***************************************************************
6604 REM                           DEPTH- AND VELOCITY-ZONE OUTPUT
6606 REM      ***************************************************************
6608 REM
6610 PRINT #2, B$; SPC(66 - LEN(B$)); "PAGE 2"
6612 FOR I = 1 TO 4: PRINT #2, : NEXT
6614 PRINT #2, "                          SINGLE-CHANNEL REGION"
6616 PRINT #2, " _______________________________________________________________
_____________"
6618 PRINT #2, : PRINT #2,
6620 PRINT #2, "                                               PROBABILITY OF DISC
HARGE"
6622 PRINT #2, "                                                BEING EXCEEDED AT
THE"
6624 PRINT #2, " ENERGY      DEPTH      DISCHARGE          APEX BY:
        WIDTH"
6626 PRINT #2, USING "  (FT)          (FT)          (CFS)
     #.####     (FT)"; BBBB
6628 PRINT #2, USING "                                                   Q    ###.
#### Q"; 10 ^ AAAA
6630 PRINT #2,
6632 NN = NHSING
6634 FOR I = 1 TO NN
6636 DEPTH = 2 * HSING(I) / 3
6638 PRINT #2, USING "  ##.#        ##.#        #######        #.#####
#.#####      #########"; HSING(I); DEPTH; QHSING(I); PHYSING(I); PHZSING(I
); WHSING(I)
6640 NEXT
6642 FOR I = 1 TO 4: PRINT #2, : NEXT
6644 PRINT #2, " _______________________________________________________________
_____________"
6646 PRINT #2, : PRINT #2,
6648 PRINT #2, "                                               PROBABILITY OF DISC
HARGE"
6650 PRINT #2, "                                                BEING EXCEEDED AT
THE"
6652 PRINT #2, " VELOCITY    DEPTH      DISCHARGE          APEX BY:
        WIDTH"
6654 PRINT #2, USING " (FT/SEC)      (FT)          (CFS)
     #.####     (FT)"; BBBB
6656 PRINT #2, USING "                                                   Q    ###.
#### Q"; 10 ^ AAAA
6658 PRINT #2,
6660 NN = NVSING
6662 FOR I = 1 TO NN
6664 DSING = VSING(I) ^ 2 / 32.16
6666 PRINT #2, USING "  ##.#        ##.#        #######        #.#####
#.#####      #########"; VSING(I); DSING; QVSING(I); PVYSING(I); PVZSING(I
); WVSING(I)
6668 NEXT
6670 IF MULT < 5 GOTO 6742
6672 PRINT #2, CHR$(12)
6674 PRINT #2, B$; SPC(66 - LEN(B$)); "PAGE 3"
6676 FOR I = 1 TO 4: PRINT #2, : NEXT
6678 PRINT #2, "                          MULTIPLE-CHANNEL REGION"
6680 PRINT #2,
```

```
6682 PRINT #2, USING "                                        SLOPE = #.#######"; SLO
PE
6684 PRINT #2, USING "                                      N-VALUE = #.#######"; NVA
LUE
6685 IF QHMULT(1) < 1 THEN GOTO 6744
6686 PRINT #2, " ____________________________________________________________
_______________"
6688 PRINT #2, : PRINT #2,
6690 PRINT #2, "                                              PROBABILITY OF DISC
HARGE"
6692 PRINT #2, "                                               BEING EXCEEDED AT
THE"
6694 PRINT #2, " ENERGY      DEPTH      DISCHARGE            APEX BY:
        WIDTH"
6696 PRINT #2, USING "  (FT)         (FT)          (CFS)
     #.####     (FT)"; BBBB
6698 PRINT #2, USING "                                                     Q    ###.
#### Q"; 10 ^ AAAA
6700 PRINT #2,
6702 NN = NHMULT
6704 FOR I = 1 TO NN
6706 DEPTH = .09168 * NVALUE ^ .6 * QHMULT(I) ^ .36 / SLOPE ^ .3
6708 PRINT #2, USING "  ##.#         ##.#          #######          #.#####
#.#####     #########"; HMULT(I); DEPTH; QHMULT(I); PHYMULT(I); PHZMULT(I
); WHMULT(I)
6710 NEXT
6712 FOR I = 1 TO 4: PRINT #2, : NEXT
6714 PRINT #2, " ____________________________________________________________
_______________"
6716 PRINT #2, : PRINT #2,
6717 IF NVMULT = 0 THEN PRINT #2, USING "                 VELOCITIES BETW
EEN ##.# AND ##.# FT/SEC"; VMIN; VMAX
6718 IF NVMULT = 0 THEN GOTO 6740
6719 PRINT #2, "                                              PROBABILITY OF DISC
HARGE"
6720 PRINT #2, "                                               BEING EXCEEDED AT
THE"
6722 PRINT #2, "VELOCITY     DEPTH      DISCHARGE            APEX BY:
        WIDTH"
6724 PRINT #2, USING " (FT/SEC)      (FT)          (CFS)
     #.####     (FT)"; BBBB
6726 PRINT #2, USING "                                                     Q    ###.
#### Q"; 10 ^ AAAA
6728 PRINT #2,
6730 NN = NVMULT
6732 FOR I = 1 TO NN
6734 DMULT = .09168 * NVALUE ^ .6 * QVMULT(I) ^ .36 / SLOPE ^ .3
6736 PRINT #2, USING "  ##.#         ##.#          #######          #.#####
#.#####     #########"; VMULT(I); DMULT; QVMULT(I); PVYMULT(I); PVZMULT(I
); WVMULT(I)
6738 NEXT
6740 PRINT #2, CHR$(12)
6742 RETURN
6744 PRINT #2,
6745 PRINT #2, "     DEPTHS GREATER THAN 0.5 FOOT HAVE PROBABILITIES LESS
 THAN .01"
6746 RETURN
```

```
8200 REM
8202 REM      ****************************************************************
8204 REM                          DRAW FLOOD FREQUENCY CURVE
8206 REM      ****************************************************************
8208 REM
8209 CLS
8212 LOCATE 1
8214 PRINT "                              FLOOD FREQUENCY CURVE"
8215 NL$ = "NL10;"
8216 NR$ = "NR10;"
8217 BOT$ = "R25;NU5;R31;NU5;R42;NU5;R52;NU5;R52;NU5;R42;NU5;R31;NU5;R25;
"
8218 TOP$ = "L25;ND5;L31;ND5;L42;ND5;L52;ND5;L52;ND5;L42;ND5;L31;ND5;L25;
"
8220 MAG = INT(MU + KG(28) * SIGMA)
8222 PSET (170, 160)
8224 DRAW "X" + VARPTR$(BOT$)
8226 FOR J = 1 TO MAG
8228     FOR I = 2 TO 10
8230         UP = INT(144 * LOG(I / (I - 1)) / LOG(10) / MAG + .5)
8232         DRAW "U=" + VARPTR$(UP): DRAW "X" + VARPTR$(NL$)
8234     NEXT I
8236 NEXT J
8238 DRAW "X" + VARPTR$(TOP$)
8240 FOR J = 1 TO MAG
8242     FOR I = 2 TO 10
8244         DWN = INT(144 * LOG((12 - I) / (11 - I)) / LOG(10) / MAG + .
5)
8246         DRAW "D=" + VARPTR$(DWN): DRAW "X" + VARPTR$(NR$)
8248     NEXT I
8250 NEXT J
8252 LOCATE 22
8254 PRINT "                     .999 .99   .9    .5    .1   .01 .001"
8256 FOR I = 1 TO 30
8258     XX = 170 + INT(300 * (K0(30) + K0(I)) / K0(30) / 2)
8260     YY = 160 - INT(144 * (MU + KG(I) * SIGMA - 1) / MAG)
8262     IF YY > 160 OR YY < 16 THEN GOTO 8266
8264     PSET (XX, YY)
8266 NEXT I
8268 LOCATE 20, 18
8270 PRINT 10
8272 LOCATE 3, 18 - MAG
8274 PRINT 10 ^ (MAG + 1)
8276 LOCATE 5, 28: PRINT "MEAN ="; MU
8278 LOCATE 6, 25: PRINT "STD DEV ="; SIGMA
8280 LOCATE 7, 28: PRINT "SKEW ="; SKEW
8282 IF PDFOPT = 1 THEN GOTO 8286
8284 LOCATE 8, 24: PRINT "COR COEF ="; RMAX
8286 RETURN
```

```
8400 REM
8402 REM      *****************************************************************
8404 REM                                    DRAW FAN
8406 REM      *****************************************************************
8408 REM
8410 LOCATE 25
8412 INPUT "                  ********     PRESS ENTER TO CONTINUE    *******
*", KFM
8414 CLS
8418 FOR I = 1 TO 4: PRINT : NEXT I
8420 PRINT B$
8422 PRINT
8424 IF MULT = 6 THEN PRINT "MULTIPLE-CHANNEL REGION" ELSE PRINT "SINGLE-
CHANNEL REGION"
8426 PI = 3.141593
8427 IF MULT = 6 THEN NH = NHMULT ELSE NH = NHSING
8428 FOR K = 1 TO NH
8430    IF MULT = 6 THEN W = WHMULT(K) / WHMULT(1) ELSE W = WHSING(K) / W
HSING(1)
8432    R = W * 600
8433    X = INT(R * COS(-PI / 10) / 8): Y = INT((R * SIN(PI / 24) + 102)
/ 8)
8434    CIRCLE (0, 90), R, 1, -19 * PI / 10, -PI / 10
8435    LOCATE Y, X: PRINT K - .5
8436 NEXT K
8437 IF MULT = 6 THEN NV = NVMULT ELSE NV = NVSING
8438 FOR K = 1 TO NV
8439    IF MULT = 6 THEN W = WVMULT(K) / WHMULT(1) ELSE W = WVSING(K) / W
HSING(1)
8440    R = W * 600
8441    X = INT(R * COS(-PI / 10) / 8) - 1
8442    IF X < 0 OR X = 0 THEN X = 1
8443    Y = INT((-R * SIN(PI / 24) + 98) / 8): IF Y < 1 THEN Y = 1
8444    A = 19 * PI / 10
8446    B = A + 3 * PI / R
8448    IF A > 2 * PI THEN A = A - 2 * PI
8450    IF B > 2 * PI THEN B = B - 2 * PI
8452    IF A > PI / 10 AND A < PI THEN GOTO 8461
8454    IF B > PI / 10 AND B < PI THEN B = PI / 10
8456    CIRCLE (0, 90), R, 1, A, B
8458    A = B + 4 * PI / R
8460    GOTO 8446
8461    LOCATE Y, X
8462    IF MULT = 6 THEN PRINT K + VMULT(1) - 1 ELSE PRINT K + 2.5
8463 NEXT K
8464 LOCATE 18
8466 PRINT "____ DEPTH"
8468 PRINT
8470 PRINT "---- VELOCITY"
8472 MULT = MULT + 1
8474 IF MULT = 6 AND QHMULT(1) > 1 THEN GOTO 8410
8476 LOCATE 25
8478 INPUT "                  ********     PRESS ENTER TO CONTINUE    *******
*", EMM
8480 RETURN
```

```
9000 REM      ****************************************************************
9002 REM                  OPTION TO VIEW AND/OR PRINT OUTPUT DATA
9006 REM      *************************************************************
9008 A$ = "     " + A$
9010 SCREEN 0: COLOR 12
9020 CLS
9022 FOR I = 1 TO 8: PRINT : NEXT I
9024 INPUT "                 DO YOU WISH TO VIEW SOME OUTPUT DATA (Y/N)?
", V$
9026 CLS : COLOR 10
9027 IF INSTR(V$, "Y") = 0 AND INSTR(V$, "y") = 0 THEN GOTO 9900
9028 FOR I = 1 TO 5: PRINT : NEXT
9030 PRINT
9032 PRINT : PRINT : COLOR 10
9034 PRINT "                (1)....FLOOD FREQUENCY DATA"
9036 PRINT
9038 PRINT "                (2)....TRANSFORMATION DATA"
9040 PRINT
9042 PRINT "                (3)....100-YEAR DEPTH-ZONE DATA -- SINGLE-CHANN
EL REGION"
9044 PRINT
9046 PRINT "                (4)....100-YEAR VELOCITY-ZONE DATA -- SINGLE-CH
ANNEL REGION"
9048 IF MULT < 5.5 THEN GOTO 9056 ELSE PRINT
9050 PRINT "                (5)....100-YEAR DEPTH-ZONE DATA -- MULTIPLE-CHA
NNEL REGION"
9052 PRINT
9054 PRINT "                (6)....100-YEAR VELOCITY-ZONE DATA -- MULTIPLE-
CHANNEL REGION"
9056 PRINT CHR$(11): FOR I = 1 TO 2: PRINT CHR$(31): NEXT I
9057 COLOR 12
9058 INPUT "       PLEASE SELECT, BY NUMBER, THE DATA THAT YOU WISH TO VI
EW....", SEL
9059 COLOR 10
9060 IF SEL = 1 THEN GOTO 9100
9062 IF SEL = 2 THEN GOTO 9200
9064 IF SEL = 3 THEN GOTO 9300
9066 IF SEL = 4 THEN GOTO 9400
9067 IF MULT < 5.5 THEN GOTO 9072
9068 IF SEL = 5 THEN GOTO 9500
9070 IF SEL = 6 THEN GOTO 9600
9072 CLS : PRINT : PRINT
9074 PRINT "                 SORRY, THERE IS NO DATA SELECTION NUMBER "; S
EL
9076 FOR I = 1 TO 5: PRINT : NEXT I
9077 COLOR 12
9078 GOTO 9024
```

```
9100 REM
9102 REM      ****************************************************************
9104 REM                          FLOOD FREQUENCY OUTPUT
9106 REM      ****************************************************************
9107 REM
9108 MU = MUY: SIGMA = SIGMAY
9109 CLS : PRINT
9110 PRINT A$
9116 FOR I = 1 TO 2: PRINT : NEXT
9118 IF PDFOPT = 2 GOTO 9124
9120 PRINT "          FLOOD FREQUENCY CURVE DEFINED BY MEAN, STANDARD DEVIA
TION, AND SKEW"
9122 GOTO 9140
9124 PRINT "          FLOOD FREQUENCY CURVE DEFINED BY LEAST-SQUARES FIT OF
 DATA"
9126 FOR I = 1 TO 2: PRINT : NEXT
9128 PRINT "          RETURN INTERVAL     INPUT DISCHARGE   BEST FIT DISCHA
RGE"
9130 PRINT "              (YEARS)                 (CFS)               (CFS)"
9132 PRINT
9134 FOR K = 1 TO NOF
9136    PRINT USING "             ####                ######
######"; RET(K); Q(K); 10 ^ (SIGMA * KK(K) + MU)
9138 NEXT
9139 GOTO 9180
9140 PRINT
9142 PRINT USING "                                          MEAN = ##.######"
; MUY
9144 PRINT USING "                            STANDARD DEVIATION = ##.######"
; SIGMAY
9146 PRINT USING "                                          SKEW = ##.#"; SKE
W
9148 FOR I = 1 TO 2: PRINT : NEXT
9150 PRINT "          SUMMARY OF DISCHARGES:"
9152 PRINT
9154 PRINT USING "                            10-YEAR DISCHARGE = ######"; 1
0 ^ (SIGMA * KG(20) + MU)
9156 PRINT USING "                            50-YEAR DISCHARGE = ######"; 1
0 ^ (SIGMA * KG(24) + MU)
9158 PRINT USING "                           100-YEAR DISCHARGE = ######"; 1
0 ^ (SIGMA * KG(25) + MU)
9160 PRINT USING "                           500-YEAR DISCHARGE = ######"; 1
0 ^ (SIGMA * KG(27) + MU)
9161 IF INSTR(FFD$, "Y") = 1 OR INSTR(FFD$, "y") = 1 THEN GOTO 9192
9162 FOR I = 1 TO 3: PRINT : NEXT I
9163 COLOR 12
9164 INPUT "  DO YOU WISH TO VIEW MORE OUTPUT DATA (Y/N)?  ", V$
9168 GOTO 9026
9180 FOR I = 1 TO 3: PRINT : NEXT I
9181 COLOR 12
9182 INPUT "              ********    PRESS ENTER TO CONTINUE    *******
*", EMM
9183 CLS : COLOR 10
9184 FOR I = 1 TO 4: PRINT : NEXT I
9185 PRINT A$
9186 FOR I = 1 TO 3: PRINT : NEXT I
```

```
9188 PRINT "    FLOOD FREQUENCY CURVE DEFINED BY LEAST-SQUARES FIT OF DAT
A"
9190 GOTO 9140
9192 FOR I = 1 TO 5: PRINT : NEXT I
9194 PRINT "                        STANDARD DEVIATION TIMES SKEW GREATER THAN
2.1"
9195 PRINT "                                     WIDTHS CANNOT BE COMPUTED"
9196 PRINT : COLOR 12
9197 PRINT "                                          PROGRAM TERMINATED"
9198 PRINT : INPUT "                                       PRESS ENTER TO CONTINU
E", RTP
9199 GOTO 9900


9200 REM     ************************************************************
9201 REM                         TRANSFORMATION OUTPUT
9202 REM     ************************************************************
9203 CLS
9204 FOR I = 1 TO 4: PRINT : NEXT
9205 PRINT A$
9206 REM
9208 FOR I = 1 TO 2: PRINT : NEXT
9210 IF SKEW = 0 GOTO 9218
9212 AAAA = -.92 * TRANS / (SCALE - .92): BBBB = SCALE / (SCALE - .92)
9214 PRINT USING "      STATISTICS AFTER TRANSFORMATION OF Y=LOG(Q) TO Z=
#.####+#.#### LOG(Q)"; -.92 * TRANS / (SCALE - .92); SCALE / (SCALE - .92
)
9216 GOTO 9222
9218 AAAA = .92 * SIGMAY * SIGMAY: BBBB = 1!
9220 PRINT USING "      STATISTICS AFTER TRANSFORMATION OF Y=LOG(Q) TO
#.####+LOG(Q)"; .92 * SIGMAY * SIGMAY
9222 PRINT
9224 PRINT USING "                              MEAN OF Z = ##.######"; MU
Z
9226 PRINT USING "                     STANDARD DEVIATION = ##.######"; SI
GMAZ
9228 PRINT USING "                                   SKEW = ##.######"; SK
EW
9230 PRINT USING "                TRANSFORMATION CONSTANT = ##.######"; CN
ST
9232 FOR I = 1 TO 5: PRINT : NEXT I
9233 COLOR 12
9236 INPUT "   DO YOU WISH TO VIEW MORE OUTPUT DATA (Y/N)?  ", V$
9238 GOTO 9026
```

```
9300 REM
9301 REM          ****************************************************************
9302 REM                                DEPTH-ZONE OUTPUT DATA
9304 REM          ****************************************************************
9306 CLS
9308 FOR I = 1 TO 2: PRINT : NEXT I
9310 PRINT A$
9312 FOR I = 1 TO 2: PRINT : NEXT
9314 PRINT "                                  SINGLE-CHANNEL REGION"
9316 PRINT "        ________________________________________________________________
          "
________________
9318 PRINT : PRINT
9320 PRINT "                                                    PROBABILITY OF DISCH
ARGE"
9322 PRINT "                                                     BEING EXCEEDED AT
THE"
9324 PRINT "     ENERGY       DEPTH       DISCHARGE              APEX BY:
        WIDTH"
9326 PRINT USING "      (FT)          (FT)          (CFS)
     #.####      (FT)"; BBBB
9328 PRINT USING "                                                          Q    ###.
#### Q"; 10 ^ AAAA
9330 PRINT
9332 NN = NHSING
9334 FOR I = 1 TO NN
9336 DEPTH = 2 * HSING(I) / 3
9338 PRINT USING "      ##.#        ##.#        #######        #.#####
#.#####     #########"; HSING(I); DEPTH; QHSING(I); PHYSING(I); PHZSING(I
); WHSING(I)
9340 NEXT
9341 PRINT
9342 PRINT "                               AVULSION FACTOR ="; AVUL
9343 FOR I = 1 TO 3: PRINT : NEXT I
9344 COLOR 12
9346 INPUT "   DO YOU WISH TO VIEW MORE OUTPUT DATA (Y/N)?  ", V$
9348 GOTO 9026
```

```
9400 REM
9401 REM      *****************************************************************
9402 REM                         VELOCITY-ZONE OUTPUT DATA
9404 REM      ****************************************************************
9406 CLS
9408 FOR I = 1 TO 2: PRINT : NEXT I
9410 PRINT A$
9412 FOR I = 1 TO 2: PRINT : NEXT I
9420 PRINT "                              SINGLE-CHANNEL REGION"
9422 PRINT "    ________________________________________________________
_____________"
9424 PRINT : PRINT
9426 PRINT "                                                  PROBABILITY OF DISC
HARGE"
9428 PRINT "                                                   BEING EXCEEDED AT
THE"
9430 PRINT "     VELOCITY    DEPTH       DISCHARGE               APEX BY:
     WIDTH"
9432 PRINT USING "      (FT/SEC)      (FT)          (CFS)
      #.####      (FT)"; BBBB
9434 PRINT USING "                                                        Q    ###.
#### Q"; 10 ^ AAAA
9436 PRINT
9438 NN = NVSING
9440 FOR I = 1 TO NN
9442 DSING = VSING(I) ^ 2 / 32.16
9444 PRINT USING "       ##.#        ##.#        #######         #.#####
#.#####     #########"; VSING(I); DSING; QVSING(I); PVYSING(I); PVZSING(I
); WVSING(I)
9446 NEXT
9447 PRINT
9448 PRINT "                                 AVULSION FACTOR ="; AVUL
9449 FOR I = 1 TO 3: PRINT : NEXT I
9450 COLOR 12
9452 INPUT "   DO YOU WISH TO VIEW MORE OUTPUT DATA (Y/N)?  ", V$
9454 GOTO 9026
```

```
9500 REM
9501 REM        ************************************************************
9502 REM                          DEPTH-ZONE OUTPUT DATA
9504 REM        ************************************************************
9506 CLS
9508 FOR I = 1 TO 2: PRINT : NEXT I
9510 PRINT A$
9512 FOR I = 1 TO 2: PRINT : NEXT I
9520 PRINT "                          MULTIPLE-CHANNEL REGION"
9530 PRINT
9532 PRINT USING "                                  SLOPE = #.#######"; SLO
PE
9534 PRINT USING "                                N-VALUE = #.#######"; NVA
LUE
9536 IF QHMULT(1) < 1 THEN GOTO 9574
9538 PRINT "  ________________________________________________________________
________________"
9540 PRINT : PRINT
9542 PRINT "                                        PROBABILITY OF DISC
HARGE"
9544 PRINT "                                         BEING EXCEEDED AT
THE"
9546 PRINT "     ENERGY       DEPTH       DISCHARGE            APEX BY:
        WIDTH"
9548 PRINT USING "       (FT)          (FT)          (CFS)
      #.####      (FT)"; BBBB
9550 PRINT USING "                                                  Q    ###.
#### Q"; 10 ^ AAAA
9552 PRINT
9554 NN = NHMULT
9556 FOR I = 1 TO NN
9558 DEPTH = .09168 * NVALUE ^ .6 * QHMULT(I) ^ .36 / SLOPE ^ .3
9560 PRINT USING "      ##.#        ##.#        #######        #.#####
#.#####      #########"; HMULT(I); DEPTH; QHMULT(I); PHYMULT(I); PHZMULT(I
); WHMULT(I)
9562 NEXT
9564 GOTO 9577
9574 PRINT
9576 PRINT "       DEPTHS GREATER THAN 0.5 FOOT HAVE PROBABILITIES LESS
 THAN .01"
9577 PRINT
9578 PRINT "                           AVULSION FACTOR ="; AVUL
9580 FOR I = 1 TO 3: PRINT : NEXT I
9584 COLOR 12
9586 INPUT "  DO YOU WISH TO VIEW MORE OUTPUT DATA (Y/N)?  ", V$
9588 GOTO 9026
```

```
9600 REM
9601 REM        ***************************************************************
9602 REM                          VELOCITY-ZONE OUTPUT DATA
9604 REM        ****************************************************************
9606 CLS
9608 FOR I = 1 TO 2: PRINT : NEXT I
9610 PRINT A$
9612 FOR I = 1 TO 2: PRINT : NEXT I
9620 PRINT "                              MULTIPLE-CHANNEL REGION"
9622 PRINT : PRINT
9624 IF NVMULT = 0 THEN PRINT USING "                              VELOCITIES BETW
EEN ##.# AND ##.# FT/SEC"; VMIN; VMAX
9626 IF NVMULT = 0 THEN GOTO 9650
9628 PRINT "                                                    PROBABILITY OF DISC
HARGE"
9630 PRINT "                                                     BEING EXCEEDED AT
THE"
9632 PRINT "   VELOCITY     DEPTH      DISCHARGE             APEX BY:
        WIDTH"
9634 PRINT USING "     (FT/SEC)      (FT)          (CFS)
    #.####      (FT)"; BBBB
9636 PRINT USING "                                                          Q    ###.
#### Q"; 10 ^ AAAA
9638 PRINT
9640 NN = NVMULT
9642 FOR I = 1 TO NN
9644 DMULT = .09168 * NVALUE ^ .6 * QVMULT(I) ^ .36 / SLOPE ^ .3
9646 PRINT USING "      ##.#       ##.#       #######        #.#####
#.#####    #########"; VMULT(I); DMULT; QVMULT(I); PVYMULT(I); PVZMULT(I
); WVMULT(I)
9648 NEXT
9649 PRINT
9650 PRINT "                         AVULSION FACTOR ="; AVUL
9651 FOR I = 1 TO 3: PRINT : NEXT I
9652 COLOR 12
9654 INPUT "   DO YOU WISH TO VIEW MORE OUTPUT DATA (Y/N)?  ", V$
9656 GOTO 9026


9900 REM       ****************************************************************
*****
9902 REM                         OPTION TO PRINT OUTPUT
9904 REM       ****************************************************************
*****
9906 REM
9908 REM
9910 COLOR 12
9912 FOR I = 1 TO 4: PRINT : NEXT I
9914 INPUT "                   DO YOU WISH TO PRINT THE OUTPUT (Y/N)?  ",
 PRNT$
9916 IF INSTR(PRNT$, "Y") = 0 AND INSTR(PRNT$, "y") = 0 THEN GOTO 9920
9918 OPEN "FANN" FOR OUTPUT AS #3
9920 CLS
9923 SYSTEM
```

```
1 REM
2 REM          AAA          GGGGGGGGG          AAA          IIIIIIIII  NNN       NNN
3 REM         AAAAA        GGG     GGG        AAAAA            III     NNNN      NNN
4 REM        AAA AAA       GGG               AAA AAA           III     NNNNNN    NNN
5 REM       AAA   AAA      GGG              AAA   AAA          III     NNN NNN NNN
6 REM      AAAAAAAAAAAA    GGG   GGGGGG    AAAAAAAAAAAA        III     NNN   NNNNN
7 REM     AAA        AAA   GGG      GGG   AAA        AAA       III     NNN    NNNN
8 REM     AAA        AAA    GGGGGGGGG     AAA        AAA   IIIIIIIII   NNN     NNN
9 REM
10 CLS
20 COLOR 12
30 FOR I=1 TO 6 : PRINT : NEXT I
40 INPUT "                         DO YOU WISH TO MAKE ANOTHER RUN (Y/N)?  ",NWR
N$
50 IF INSTR(NWRN$,"Y")=0 AND INSTR(NWRN$,"y")=0 THEN GOTO 70
60 OPEN "FANN" FOR OUTPUT AS #3
70 SYSTEM
```

ATTACHMENT WR-1B

Evaluating Scour at Bridges

ATTACHMENT WR-1B

U.S. Department of Transportation
Federal Highway Administration

Publication No. FHWA NHI 01-001
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Evaluating Scour At Bridges
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16. This document is the fourth edition of HEC-18. It presents the state of knowledge and practice for the design, evaluation and inspection of bridges for scour. There are two companion documents, HEC-20 entitled "Stream Stability at Highway Structures," and HEC-23 entitled "Bridge Scour and Stream Instability Countermeasures." These three documents contain updated material from previous editions and the publication, "Interim Procedures for Evaluating Scour at Bridges," issued in September 1988 as part of the FHWA Technical Advisory T 5140.20, "Scour at Bridges." T5140.20 has since been superseded by T 5140.23, "Evaluating Scour at Bridges" dated October 28, 1991. This fourth edition of HEC-18 contains revisions obtained from further scour-related developments and the use of the 1995 edition by the highway community.

The major changes in this fourth edition of HEC-18 are: change in nomenclature to using General Scour to include both contraction scour and other general scour components, changing the order of Chapters 2 and 3 so that the policy chapter entitled "Designing Bridges to Resist Scour" comes before the chapter entitled "Basic Concepts and Definitions of Scour;" and separating Chapter 4 into separate chapters dealing with each of the major scour components. In addition, a new K_4, to account for coarse bed material in the pier scour equation, improved methods to compute scour for complex pier configurations, example problems, and additional information on computer programs for modeling tidal hydraulics are given. There is no change in the recommendations regarding abutment scour.

In addition to minor editorial revisions, the following substantive changes have been made in this revised edition of HEC-18: revised definition of grain roughness k_s, (p. xii); changed guidance on determining the magnitude of the 500-year flood (p. 2.9). Note 3 definition of K_3 factor corrected (p. 6.5); revised guidance on minimum value of K_4 factor (p. 6.6); revised definition of grain roughness k_s (p. 6.13); and corrected multiplier for kinematic viscosity in Appendix A (Table A.7).

17. Key Words

scour design, contraction scour, local scour, pier scour, abutment scour, scour susceptible, scour critical, clear-water scour, live-bed scour, superflood, bridge inspection, countermeasures, tidal scour

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LIST OF SYMBOLS

a	=	Pier width, m (ft)
A	=	Maximum amplitude of elevation of the tide or storm surge, m (ft)
A_e	=	Flow area of the approach cross section obstructed by the embankment, m² (ft^2)
A_c	=	Cross-sectional area of the waterway at mean tide elevation--half between high and low tide, m² (ft^2)
	=	Net cross-sectional area in the inlet at the crossing, at mean water surface elevation, m^2 (ft^2)
C_d	=	Coefficient of discharge
D	=	Diameter of the bed material, m (ft)
	=	Diameter of smallest nontransportable particle in the bed material, m (ft)
D_m	=	Effective mean diameter of the bed material in the bridge, mm or m
	=	1.25 D_{50}
D_{50}	=	Median diameter of the bed material, diameter which 50% of the sizes are smaller, mm or m
D_{84}	=	Diameter of the bed material of which 84% are smaller, mm or m
D_{90}	=	Diameter of the bed material of which 90% are smaller, mm or m
Fr	=	Froude Number [V/(gy)½]
	=	Froude Number of approach flow upstream of the abutment
	=	Froude Number based on the velocity and depth adjacent to and upstream of the abutment
Fr_1	=	Froude Number directly upstream of a pier
g	=	Acceleration of gravity, m/s^2 (ft/s²)
h_{1-2}	=	Head loss between sections 1 and 2, m (ft)
h_c	=	Average depth of flow in the waterway at mean water elevation, m (ft)
H	=	Height (i.e., height of a dune), m (ft)
H_b	=	Distance from the low chord of the bridge to the average elevation of the stream bed before scour, m (ft)
K	=	Various coefficients in equations as described below
	=	Conveyance in Manning's equation $\frac{(AR^{2/3})}{n}$, m^3/s (ft^3/s)
	=	Bottom width of the scour hole as a fraction of scour depth, m (ft)
K_o	=	Velocity head loss coefficient on the ocean side or downstream side of the waterway
K_b	=	Velocity head loss coefficient on the bay or upstream side of the waterway
K_s	=	Shields coefficient
K_1	=	Correction factor for pier nose shape
	=	Coefficient for abutment shape

K_2	=	Correction factor for angle of attack of flow
	=	Coefficient for angle of embankment to flow
K_3	=	Correction factor for increase in equilibrium pier scour depth for bed condition
K_4	=	Correction factor for armoring in pier scour equation
k_1 & k_2	=	Exponents determined in Laursen live-bed contraction equation, depends on the mode of bed material transport
k_s	=	Grain roughness of the bed, m (ft)
L	=	Length of pier, m (ft)
L_c	=	Length of the waterway, m (ft)
L′ or L	=	Length of abutment (embankment) projected normal to flow, m (ft)
n	=	Manning's n
n_1	=	Manning's n for upstream main channel
n_2	=	Manning's n for contracted section
Q	=	Discharge through the bridge or on the overbank at the bridge, m^3/s (ft^3/s)
Q_e	=	Flow obstructed by the abutment and approach embankment, m^3/s (ft^3/s)
Q_{max}	=	Maximum discharge in the tidal cycle, m^3/s (ft^3/s)
	=	Maximum discharge in the inlet, m^3/s (ft^3/s)
Q_t	=	Discharge at any time, t, in the tidal cycle, m^3/s (ft^3/s)
Q_1	=	Flow in the upstream main channel transporting sediment, m^3/s (ft^3/s)
Q_2	=	Flow in the contracted channel, m^3/s (ft^3/s). Often this is equal to the total discharge unless the total flood flow is reduced by relief bridges or water overtopping the approach roadway
Q_{100}	=	Storm-event having a probability of occurrence of one every 100 years, m^3/s (ft^3/s)
Q_{500}	=	Storm-event having a probability of occurrence of one every 500 years, m^3/s (ft^3/s)
q	=	Discharge per unit width, $m^3/s/m$ ($ft^3/s/ft$)
	=	Discharge in conveyance tube, m^3/s (ft^3/s)
R	=	Hydraulic radius
	=	Coefficient of resistance
SBR	=	Set-back ratio of each abutment
S_1	=	Slope of energy grade line of main channel, m/m (ft/ft)
S_f	=	Slope of the energy grade line, m/m (ft/ft)
S_o	=	Average bed slope, m/m (ft/ft)
S_s	=	Specific gravity of bed material. For most bed material this is equal to 2.65
t	=	Time from the beginning of total cycle, min
T	=	Total time for one complete tidal cycle, min
	=	Tidal period between successive high or low tides, s

V	=	Average velocity, m/s (ft/s)
	=	Characteristic average velocity in the contracted section for estimating a median stone diameter, D_{50}, m/s (ft/s)
V_{max}	=	Q_{max}/A', or maximum velocity in the inlet, m/s (ft/s)
V_1	=	Average velocity at upstream main channel, m/s (ft/s)
	=	Mean velocity of flow directly upstream of the pier, m/s (ft/s)
V_2	=	Average velocity in the contracted section, m/s (ft/s)
V_c	=	Critical velocity, m/s (ft/s), above which the bed material of size D, D_{50}, etc. and smaller will be transported
V_{c50}	=	Critical velocity for D_{50} bed material size, m/s (ft/s)
V_{c90}	=	Critical velocity for D_{90} bed material size, m/s (ft/s)
V_e	=	Q_e/A_e, m/s (ft/s)
V_f	=	Average velocity of flow zone below the top of the footing, m/s (ft/s)
V_i	=	Approach velocity when particles at a pier begin to move, m/s (ft/s)
V_{max}	=	Maximum average velocity in the cross section at Q_{max}, m/s (ft/s)
V_R	=	Velocity ratio
V_*	=	Shear velocity in the upstream section, m/s (ft/s)
	=	$(\tau_o/\rho) = (gy_1S_1)^{1/2}$
VOL	=	Volume of water in the tidal prism between high and low tide levels, m^3 (ft^3)
W	=	Bottom width of the bridge less pier widths, or overbank width (set back distance less pier widths, m (ft)
	=	Topwidth of the scour hole from each side of the pier of footing, m (ft)
W_1	=	Bottom width of the upstream main channel, m (ft)
W_2	=	Bottom width of the main channel in the contracted section less pier widths, m (ft)
ω	=	Fall velocity of the bed material of a given size, m/s (ft/s)
y	=	Depth of flow, m (ft). This depth is used in the Neill's and Larson's equation as the upstream channel depth to determine V_c.
	=	Depth of flow in the contracted bridge opening for estimating a median stone diameter, D_{50}, m (ft)
	=	Amplitude or elevation of the tide above mean water level, m (ft), at time t
y_a	=	Average depth of flow on the floodplain, m (ft)
y_f	=	Distance from the bed to the top of the footing, m (ft)
y_o	=	Existing depth of flow, m (ft)
y_{ps}	=	Depth of pier scour, m (ft)
y_s	=	Average contraction scour depth, m (ft)
y_s	=	Local scour depth, m (ft)
y_s	=	Depth of vertical contraction scour relative to mean bed elevation, m (ft)
y_{sc}	=	Depth of contraction scour, m (ft)
y_1	=	Average depth in the upstream main channel or on the floodplain prior to contraction scour, m (ft)

	=	Depth of flow directly upstream of the pier, m (ft)
	=	Depth of flow at the abutment, on the overbank or in the main channel for abutment scour, m (ft)
y_2	=	Average depth in the contracted section (bridge opening) or on the overbank at the bridge, m (ft)
	=	Average depth under lower cord, m (ft)
Z	=	Vertical offset to datum, m (ft)
τ_2, τ_o	=	Average bed shear stress at the contracted section, Pa or N/m^2 (lbs/ft^2)
τ_c	=	Critical bed shear stress at incipient motion, N/m^2 (lbs/ft^2)
γ	=	Specific weight of water, N/m^3 (lbs/ft^3)
ρ	=	Density of water, kg/m^3 ($slugs/ft^3$)
ρ_s	=	Density of sediment, kg/m^3 ($slugs/ft^3$)
θ	=	Angle of repose of the bed material (ranges from about 30° to 44°)
	=	Skew angle of flow with respect to pier
	=	Skew angle of abutment (embankment) with respect to flow
	=	Angle, in degrees, subdividing the tidal cycle
ΔH	=	Maximum difference in water surface elevation between the bay and ocean side of the inlet or channel, m (ft)

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GLOSSARY

abrasion: Removal of streambank material due to entrained sediment, ice, or debris rubbing against the bank.

aggradation: General and progressive buildup of the longitudinal profile of a channel bed due to sediment deposition.

alluvial channel: Channel wholly in alluvium; no bedrock is exposed in channel at low flow or likely to be exposed by erosion.

alluvial fan: A fan-shaped deposit of material at the place where a stream issues from a narrow valley of high slope onto a plain or broad valley of low slope. An alluvial cone is made up of the finer materials suspended in flow while a debris cone is a mixture of all sizes and kinds of materials.

alluvial stream: A stream which has formed its channel in cohesive or noncohesive materials that have been and can be transported by the stream.

alluvium: Unconsolidated material deposited by a stream in a channel, floodplain, alluvial fan, or delta.

alternating bars: Elongated deposits found alternately near the right and left banks of a channel.

anabranch: Individual channel of an anabranched stream.

anabranched stream: A stream whose flow is divided at normal and lower stages by large islands or, more rarely, by large bars; individual islands or bars are wider than about three times water width; channels are more widely and distinctly separated than in a braided stream.

anastomosing stream: An anabranched stream.

angle of repose: The maximum angle (as measured from the horizontal) at which gravel or sand particles can stand.

annual flood: The maximum flow in one year (may be daily or instantaneous).

apron: Protective material placed on a streambed to resist scour.

apron, launching: An apron designed to settle and protect the side slopes of a scour hole after settlement.

armor (armoring): Surfacing of channel bed, banks, or embankment slope to resist erosion and scour. (a) Natural process whereby an erosion- resistant layer of relatively large particles is formed on a streambed due to the removal of finer particles by streamflow; (b) placement of a covering to resist erosion.

articulated concrete mattress:	Rigid concrete slabs which can move without separating as scour occurs; usually hinged together with corrosion-resistant cable fasteners; primarily placed for lower bank protection.
average velocity:	Velocity at a given cross section determined by dividing discharge by cross sectional area.
avulsion:	A sudden change in the channel course that usually occurs when a stream breaks through its banks; usually associated with a flood or a catastrophic event.
backfill:	The material used to refill a ditch or other excavation, or the process of doing so.
backwater:	The increase in water surface elevation relative to the elevation occurring under natural channel and floodplain conditions. It is induced by a bridge or other structure that obstructs or constricts the free flow of water in a channel.
backwater area:	The low-lying lands adjacent to a stream that may become flooded due to backwater.
bank:	The sides of a channel between which the flow is normally confined.
bank, left (right):	The side of a channel as viewed in a downstream direction.
bankfull discharge:	Discharge that, on the average, fills a channel to the point of overflowing.
bank protection:	Engineering works for the purpose of protecting streambanks from erosion.
bank revetment:	Erosion-resistant materials placed directly on a streambank to protect the bank from erosion.
bar:	An elongated deposit of alluvium within a channel, not permanently vegetated.
base floodplain:	The floodplain associated with the flood with a 100-year recurrence interval.
bay:	A body of water connected to the ocean with an inlet.
bed:	The bottom of a channel bounded by banks.
bed form:	A recognizable relief feature on the bed of a channel, such as a ripple, dune, plane bed, antidune, or bar. Bed forms are a consequence of the interaction between hydraulic forces (boundary shear stress) and the bed sediment.

bed layer:	A flow layer, several grain diameters thick (usually two) immediately above the bed.
bed load:	Sediment that is transported in a stream by rolling, sliding, or skipping along the bed or very close to it; considered to be within the bed layer (contact load).
bed load discharge (or bed load):	The quantity of bed load passing a cross section of a stream in a unit of time.
bed material:	Material found in and on the bed of a stream (May be transported as bed load or in suspension).
bedrock:	The solid rock exposed at the surface of the earth or overlain by soils and unconsolidated material.
bed sediment discharge:	The part of the total sediment discharge that is composed of grain sizes found in the bed and is equal to the transport capability of the flow.
bed shear (tractive force):	The force per unit area exerted by a fluid flowing past a stationary boundary.
bed slope:	The inclination of the channel bottom.
blanket:	Material covering all or a portion of a streambank to prevent erosion.
boulder:	A rock fragment whose diameter is greater than 250 mm.
braid:	A subordinate channel of a braided stream.
braided stream:	A stream whose flow is divided at normal stage by small mid-channel bars or small islands; the individual width of bars and islands is less than about three times water width; a braided stream has the aspect of a single large channel within which are subordinate channels.
bridge opening:	The cross-sectional area beneath a bridge that is available for conveyance of water.
bridge waterway:	The area of a bridge opening available for flow, as measured below a specified stage and normal to the principal direction of flow.
bulk density:	Density of the water sediment mixture (mass per unit volume), including both water and sediment.
bulkhead:	A vertical, or near vertical, wall that supports a bank or an embankment; also may serve to protect against erosion.
bulking:	Increasing the water discharge to account for high concentrations of sediment in the flow.

catchment: See drainage basin.

causeway: Rock or earth embankment carrying a roadway across water.

caving: The collapse of a bank caused by undermining due to the action of flowing water.

cellular-block mattress: Interconnected concrete blocks with regular cavities placed directly on a streambank or filter to resist erosion. The cavities can permit bank drainage and the growth of vegetation where synthetic filter fabric is not used between the bank and mattress.

channel: The bed and banks that confine the surface flow of a stream.

channelization: Straightening or deepening of a natural channel by artificial cutoffs, grading, flow-control measures, or diversion of flow into an engineered channel.

channel diversion: The removal of flows by natural or artificial means from a natural length of channel.

channel pattern: The aspect of a stream channel in plan view, with particular reference to the degree of sinuosity, braiding, and anabranching.

channel process: Behavior of a channel with respect to shifting, erosion and sedimentation.

check dam: A low dam or weir across a channel used to control stage or degradation.

choking (of flow): Excessive constriction of flow which may cause severe backwater effect.

clay (mineral): A particle whose diameter is in the range of 0.00024 to 0.004 mm.

clay plug: A cutoff meander bend filled with fine grained cohesive sediments.

clear-water scour: Scour at a pier or abutment (or contraction scour) when there is no movement of the bed material upstream of the bridge crossing at the flow causing bridge scour.

cobble: A fragment of rock whose diameter is in the range of 64 to 250 mm.

concrete revetment: Unreinforced or reinforced concrete slabs placed on the channel bed or banks to protect it from erosion.

confluence: The junction of two or more streams.

constriction: A natural or artificial control section, such as a bridge crossing, channel reach or dam, with limited flow capacity in which the upstream water surface elevation is related to discharge.

contact load: Sediment particles that roll or slide along in almost continuous contact with the streambed (bed load).

contraction: The effect of channel or bridge constriction on flow streamlines.

contraction scour: Contraction scour, in a natural channel or at a bridge crossing, involves the removal of material from the bed and banks across all or most of the channel width. This component of scour results from a contraction of the flow area at the bridge which causes an increase in velocity and shear stress on the bed at the bridge. The contraction can be caused by the bridge or from a natural narrowing of the stream channel.

Coriolis force: The inertial force caused by the Earth's rotation that deflects a moving body to the right in the Northern Hemisphere.

countermeasure: A measure intended to prevent, delay or reduce the severity of hydraulic problems.

crib: A frame structure filled with earth or stone ballast, designed to reduce energy and to deflect streamflow away from a bank or embankment.

critical shear stress: The minimum amount of shear stress required to initiate soil particle motion.

crossing: The relatively short and shallow reach of a stream between bends; also crossover or riffle.

cross section: A section normal to the trend of a channel or flow.

current: Water flowing through a channel.

current meter: An instrument used to measure flow velocity.

cut bank: The concave wall of a meandering stream.

cutoff: (a) A direct channel, either natural or artificial, connecting two points on a stream, thereby shortening the original length of the channel and increasing its slope; (b) A natural or artificial channel which develops across the neck of a meander loop (neck cutoff) or across a point bar (chute cutoff).

cutoff wall: A wall, usually of sheet piling or concrete, that extends down to scour-resistant material or below the expected scour depth.

daily discharge:	Discharge averaged over one day (24 hours).
debris:	Floating or submerged material, such as logs, vegetation, or trash, transported by a stream.
degradation (bed):	A general and progressive (long-term) lowering of the channel bed due to erosion, over a relatively long channel length.
deep water (for waves):	Water of such a depth that surface waves are little affected by bottom conditions; customarily, water deeper than half the wavelength.
depth of scour:	The vertical distance a streambed is lowered by scour below a reference elevation.
design flow (design flood):	The discharge that is selected as the basis for the design or evaluation of a hydraulic structure.
dike:	An impermeable linear structure for the control or containment of overbank flow. A dike-trending parallel with a streambank differs from a levee in that it extends for a much shorter distance along the bank, and it may be surrounded by water during floods.
dike (groin, spur, jetty):	A structure extending from a bank into a channel that is designed to: (a) reduce the stream velocity as the current passes through the dike, thus encouraging sediment deposition along the bank (permeable dike); or (b) deflect erosive current away from the streambank (impermeable dike).
diurnal tide	Tides with an approximate tidal period of 24 hours.
discharge:	Volume of water passing through a channel during a given time.
dominant discharge:	(a) The discharge of water which is of sufficient magnitude and frequency to have a dominating effect in determining the characteristics and size of the stream course, channel, and bed; (b) That discharge which determines the principal dimensions and characteristics of a natural channel. The dominant formative discharge depends on the maximum and mean discharge, duration of flow, and flood frequency. For hydraulic geometry relationships, it is taken to be the bankfull discharge which has a return period of approximately 1.5 years in many natural channels.
drainage basin:	An area confined by drainage divides, often having only one outlet for discharge (catchment, watershed).
drift:	Alternative term for vegetative "debris."

ebb tide:	Flow of water from the bay or estuary to the ocean.
eddy current:	A vortex-type motion of a fluid flowing contrary to the main current, such as the circular water movement that occurs when the main flow becomes separated from the bank.
entrenched stream:	Stream cut into bedrock or consolidated deposits.
ephemeral stream:	A stream or reach of stream that does not flow for parts of the year. As used here, the term includes intermittent streams with flow less than perennial.
equilibrium scour:	Scour depth in sand-bed stream with dune bed about which live bed pier scour level fluctuates due to variability in bed material transport in the approach flow.
erosion:	Displacement of soil particles due to water or wind action.
erosion control matting:	Fibrous matting (e.g., jute, paper, etc.) placed or sprayed on a stream- bank for the purpose of resisting erosion or providing temporary stabilization until vegetation is established.
estuary:	Tidal reach at the mouth of a river.
fabric mattress:	Grout-filled mattress used for streambank protection.
fall velocity:	The velocity at which a sediment particle falls through a column of still water.
fascine:	A matrix of willow or other natural material woven in bundles and used as a filter. Also, a streambank protection technique consisting of wire mesh or timber attached to a series of posts, sometimes in double rows; the space between the rows may be filled with rock, brush, or other materials.
fetch:	The area in which waves are generated by wind having a rather constant direction and speed; sometimes used synonymously with fetch length.
fetch length:	The horizontal distance (in the direction of the wind) over which wind generates waves and wind setup.
fill slope:	Side or end slope of an earth-fill embankment. Where a fill-slope forms the streamward face of a spill-through abutment, it is regarded as part of the abutment.
filter:	Layer of fabric (geotextile) or granular material (sand, gravel, or graded rock) placed between bank revetment (or bed protection) and soil for the following purposes: (1) to prevent the soil from moving through the revetment by piping, extrusion, or erosion; (2) to prevent the revetment from sinking into the soil; and (3) to permit natural seepage from the streambank, thus preventing the buildup of excessive hydrostatic pressure.
filter blanket:	A layer of graded sand and gravel laid between fine-grained material and riprap to serve as a filter.

filter fabric (cloth): Geosynthetic fabric that serves the same purpose as a granular filter blanket.

fine sediment load: That part of the total sediment load that is composed of particle sizes finer than those represented in the bed (wash load). Normally, the fine-sediment load is finer than 0.062 mm for sand-bed channels. Silts, clays and sand could be considered wash load in coarse gravel and cobble-bed channels.

flanking: Erosion around the landward end of a stream stabilization countermeasure.

flashy stream: Stream characterized by rapidly rising and falling stages, as indicated by a sharply peaked hydrograph. Typically associated with mountain streams or highly disturbed urbanized catchments. Most flashy streams are ephemeral, but some are perennial.

flood tide: Flow of water from the ocean to the bay or estuary.

flood-frequency curve: A graph indicating the probability that the annual flood discharge will exceed a given magnitude, or the recurrence interval corresponding to a given magnitude.

floodplain: A nearly flat, alluvial lowland bordering a stream, that is subject to frequent inundation by floods.

flow-control structure: A structure either within or outside a channel that acts as a countermeasure by controlling the direction, depth, or velocity of flowing water.

flow hazard: Flow characteristics (discharge, stage, velocity, or duration) that are associated with a hydraulic problem or that can reasonably be considered of sufficient magnitude to cause a hydraulic problem or to test the effectiveness of a countermeasure.

flow slide: Saturated soil materials which behave more like a liquid than a solid. A flow slide on a channel bank can result in a bank failure.

fluvial geomorphology: The science dealing with the morphology (form) and dynamics of streams and rivers.

fluvial system: The natural river system consisting of (1) the drainage basin, watershed, or sediment source area, (2) tributary and mainstem river channels or sediment transfer zone, and (3) alluvial fans, valley fills and deltas, or the sediment deposition zone.

freeboard: The vertical distance above a design stage that is allowed for waves, surges, drift, and other contingencies.

fresh water:	Water that is not salty as compared to sea water which generally has a salinity of 35 000 parts per million.
Froude Number:	A dimensionless number that represents the ratio of inertial to gravitational forces in open channel flow.
gabion:	A basket or compartmented rectangular container made of wire mesh. When filled with cobbles or other rock of suitable size, the gabion becomes a flexible and permeable unit with which flow- and erosion-control structures can be built.
general scour:	General scour is a lowering of the streambed across the stream or waterway at the bridge. This lowering may be uniform across the bed or non-uniform. That is, the depth of scour may be deeper in some parts of the cross section. General scour may result from contraction of the flow or other general scour conditions such as flow around a bend.
geomorphology/morphology:	That science that deals with the form of the Earth, the general configuration of its surface, and the changes that take place due to erosion and deposition.
grade-control structure (sill, check dam):	Structure placed bank to bank across a stream channel (usually with its central axis perpendicular to flow) for the purpose of controlling bed slope and preventing scour or headcutting.
graded stream:	A geomorphic term used for streams that have apparently achieved a state of equilibrium between the rate of sediment transport and the rate of sediment supply throughout long reaches.
gravel:	A rock fragment whose diameter ranges from 2 to 64 mm.
groin:	A structure built from the bank of a stream in a direction transverse to the current to redirect the flow or reduce flow velocity. Many names are given to this structure, the most common being "spur," "spur dike," "transverse dike," "jetty," etc. Groins may be permeable, semi-permeable, or impermeable.
grout:	A fluid mixture of cement and water or of cement, sand, and water used to fill joints and voids.
guide bank:	A dike extending upstream from the approach embankment at either or both sides of the bridge opening to direct the flow through the opening. Some guidebanks extend downstream from the bridge (also spur dike).
hardpoint:	A streambank protection structure whereby "soft" or erodible materials are removed from a bank and replaced by stone or compacted clay. Some hard points protrude a short distance into the channel to direct erosive currents away from the bank. Hard points also occur naturally along streambanks as passing currents remove erodible materials leaving nonerodible materials exposed.

headcutting: Channel degradation associated with abrupt changes in the bed elevation (headcut) that generally migrates in an upstream direction.

helical flow: Three-dimensional movement of water particles along a spiral path in the general direction of flow. These secondary-type currents are of most significance as flow passes through a bend; their net effect is to remove soil particles from the cut bank and deposit this material on a point bar.

hydraulics: The applied science concerned with the behavior and flow of liquids, especially in pipes, channels, structures, and the ground.

hydraulic model: A small-scale physical or mathematical representation of a flow situation.

hydraulic problem: An effect of streamflow, tidal flow, or wave action such that the integrity of the highway facility is destroyed, damaged, or endangered.

hydraulic radius: The cross-sectional area of a stream divided by its wetted perimeter.

hydraulic structures: The facilities used to impound, accommodate, convey or control the flow of water, such as dams, weirs, intakes, culverts, channels, and bridges.

hydrograph: The graph of stage or discharge against time.

hydrology: The science concerned with the occurrence, distribution, and circulation of water on the earth.

imbricated: In reference to stream bed sediment particles, having an overlapping or shingled pattern.

icing: Masses or sheets of ice formed on the frozen surface of a river or floodplain. When shoals in the river are frozen to the bottom or otherwise dammed, water under hydrostatic pressure is forced to the surface where it freezes.

incised reach: A stretch of stream with an incised channel that only rarely overflows its banks.

incised stream: A stream which has deepened its channel through the bed of the valley floor, so that the floodplain is a terrace.

invert: The lowest point in the channel cross section or at flow control devices such as weirs, culverts, or dams.

island: A permanently vegetated area, emergent at normal stage, that divides the flow of a stream. Islands originate by establishment of vegetation on a bar, by channel avulsion, or at the junction of minor tributary with a larger stream.

jack: A device for flow control and protection of banks against lateral erosion consisting of three mutually perpendicular arms rigidly fixed at the center. Kellner jacks are made of steel struts strung with wire, and concrete jacks are made of reinforced concrete beams.

jack field: Rows of jacks tied together with cables, some rows generally parallel with the banks and some perpendicular thereto or at an angle. Jack fields may be placed outside or within a channel.

jetty: (a) An obstruction built of piles, rock, or other material extending from a bank into a stream, so placed as to induce bank building, or to protect against erosion; (b) A similar obstruction to influence stream, lake, or tidal currents, or to protect a harbor (also spur).

lateral erosion: Erosion in which the removal of material is extended horizontally as contrasted with degradation and scour in a vertical direction.

launching: Release of undercut material (stone riprap, rubble, slag, etc.) downslope or into a scoured area.

levee: An embankment, generally landward of top bank, that confines flow during high-water periods, thus preventing overflow into lowlands.

littoral transport or drift: Transport of beach material along a shoreline by wave action. Also, longshore sediment transport.

live-bed scour: Scour at a pier or abutment (or contraction scour) when the bed material in the channel upstream of the bridge is moving at the flow causing bridge scour.

load (or sediment load): Amount of sediment being moved by a stream.

local scour: Removal of material from around piers, abutments, spurs, and embankments caused by an acceleration of flow and resulting vortices induced by obstructions to the flow.

longitudinal profile: The profile of a stream or channel drawn along the length of its centerline. In drawing the profile, elevations of the water surface or the thalweg are plotted against distance as measured from the mouth or from an arbitrary initial point.

lower bank:	That portion of a streambank having an elevation less than the mean water level of the stream.
mathematical model:	A numerical representation of a flow situation using mathematical equations (also computer model).
mattress:	A blanket or revetment of materials interwoven or otherwise lashed together and placed to cover an area subject to scour.
meander or full meander:	A meander in a river consists of two consecutive loops, one flowing clockwise and the other counter-clockwise.
meander amplitude:	The distance between points of maximum curvature of successive meanders of opposite phase in a direction normal to the general course of the meander belt, measured between center lines of channels.
meander belt:	The distance between lines drawn tangent to the extreme limits of successive fully developed meanders.
meander length:	The distance along a stream between corresponding points of successive meanders.
meander loop:	An individual loop of a meandering or sinuous stream lying between inflection points with adjoining loops.
meander ratio:	The ratio of meander width to meander length.
meander radius of curvature:	The radius of a circle inscribed on the centerline of a meander loop.
meander scrolls:	Low, concentric ridges and swales on a floodplain, marking the successive positions of former meander loops.
meander width:	The amplitude of a fully developed meander measured from midstream to midstream.
meandering stream:	A stream having a sinuosity greater than some arbitrary value. The term also implies a moderate degree of pattern symmetry, imparted by regularity of size and repetition of meander loops. The channel generally exhibits a characteristic process of bank erosion and point bar deposition associated with systematically shifting meanders.
median diameter:	The particle diameter of the 50th percentile point on a size distribution curve such that half of the particles (by weight, number, or volume) are larger and half are smaller ($D_{50.}$)
mid-channel bar:	A bar lacking permanent vegetal cover that divides the flow in a channel at normal stage.

middle bank: The portion of a streambank having an elevation approximately the same as that of the mean water level of the stream.

migration: Change in position of a channel by lateral erosion of one bank and simultaneous accretion of the opposite bank.

mud: A soft, saturated mixture mainly of silt and clay.

natural levee: A low ridge that slopes gently away from the channel banks that is formed along streambanks during floods by deposition.

nominal diameter: Equivalent spherical diameter of a hypothetical sphere of the same volume as a given sediment particle.

nonalluvial channel: A channel whose boundary is in bedrock or non-erodible material.

normal stage: The water stage prevailing during the greater part of the year.

overbank flow: Water movement that overtops the bank either due to stream stage or to overland surface water runoff.

oxbow: The abandoned former meander loop that remains after a stream cuts a new, shorter channel across the narrow neck of a meander. Often bow-shaped or horseshoe-shaped.

pavement: Streambank surface covering, usually impermeable, designed to serve as protection against erosion. Common pavements used on streambanks are concrete, compacted asphalt, and soil-cement.

paving: Covering of stones on a channel bed or bank (used with reference to natural covering).

peaked stone dike: Riprap placed parallel to the toe of a streambank (at the natural angle of repose of the stone) to prevent erosion of the toe and induce sediment deposition behind the dike.

perennial stream: A stream or reach of a stream that flows continuously for all or most of the year.

phreatic line: The upper boundary of the seepage water surface landward of a streambank.

pile: An elongated member, usually made of timber, concrete, or steel, that serves as a structural component of a river-training structure.

pile dike: A type of permeable structure for the protection of banks against caving; consists of a cluster of piles driven into the stream, braced and lashed together.

piping:	Removal of soil material through subsurface flow of seepage water that develops channels or "pipes" within the soil bank.
point bar:	An alluvial deposit of sand or gravel lacking permanent vegetal cover occurring in a channel at the inside of a meander loop, usually somewhat downstream from the apex of the loop.
poised stream:	A stream which, as a whole, maintains its slope, depths, and channel dimensions without any noticeable raising or lowering of its bed (stable stream). Such condition may be temporary from a geological point of view, but for practical engineering purposes, the stream may be considered stable.
probable maximum flood:	A very rare flood discharge value computed by hydro-meteorological methods, usually in connection with major hydraulic structures.
quarry-run stone:	Stone as received from a quarry without regard to gradation requirements.
railbank protection:	A type of countermeasure composed of rock-filled wire fabric supported by steel rails or posts driven into streambed.
rapid drawdown:	Lowering the water against a bank more quickly than the bank can drain without becoming unstable.
reach:	A segment of stream length that is arbitrarily bounded for purposes of study.
recurrence interval:	The reciprocal of the annual probability of exceedance of a hydrologic event (also return period, exceedance interval).
regime:	The condition of a stream or its channel with regard to stability. A stream is in regime if its channel has reached an equilibrium form as a result of its flow characteristics. Also, the general pattern of variation around a mean condition, as in flow regime, tidal regime, channel regime, sediment regime, etc. (used also to mean a set of physical characteristics of a river).
regime change:	A change in channel characteristics resulting from such things as changes in imposed flows, sediment loads, or slope.
regime channel:	Alluvial channel that has attained, more or less, a state of equilibrium with respect to erosion and deposition.
regime formula:	A formula relating stable alluvial channel dimensions or slope to discharge and sediment characteristics.
reinforced-earth bulkhead:	A retaining structure consisting of vertical panels and attached to reinforcing elements embedded in compacted backfill for supporting a streambank.

reinforced revetment:	A streambank protection method consisting of a continuous stone toe-fill along the base of a bank slope with intermittent fillets of stone placed perpendicular to the toe and extending back into the natural bank.
relief bridge:	An opening in an embankment on a floodplain to permit passage of overbank flow.
retard (retarder structure):	A permeable or impermeable linear structure in a channel parallel with the bank and usually at the toe of the bank, intended to reduce flow velocity, induce deposition, or deflect flow from the bank.
revetment:	Rigid or flexible armor placed to inhibit scour and lateral erosion. (See bank revetment).
riffle:	A natural, shallow flow area extending across a streambed in which the surface of flowing water is broken by waves or ripples. Typically, riffles alternate with pools along the length of a stream channel.
riparian:	Pertaining to anything connected with or adjacent to the banks of a stream (corridor, vegetation, zone, etc.).
riprap:	Layer or facing of rock or broken concrete dumped or placed to protect a structure or embankment from erosion; also the rock or broken concrete suitable for such use. Riprap has also been applied to almost all kinds of armor, including wire-enclosed riprap, grouted riprap, sacked concrete, and concrete slabs.
river training:	Engineering works with or without the construction of embankment, built along a stream or reach of stream to direct or to lead the flow into a prescribed channel. Also, any structure configuration constructed in a stream or placed on, adjacent to, or in the vicinity of a streambank that is intended to deflect currents, induce sediment deposition, induce scour, or in some other way alter the flow and sediment regimes of the stream.
rock-and-wire mattress:	A flat wire cage or basket filled with stone or other suitable material and placed as protection against erosion.
roughness coefficient:	Numerical measure of the frictional resistance to flow in a channel, as in the Manning's or Chezy's formulas.
rubble:	Rough, irregular fragments of materials of random size used to retard erosion. The fragments may consist of broken concrete slabs, masonry, or other suitable refuse.
runoff:	That part of precipitation which appears in surface streams of either perennial or intermittent form.
run-up, wave:	Height to which water rises above still-water elevation when waves meet a beach, wall, etc.

sack revetment:	Sacks (e.g., burlap, paper, or nylon) filled with mortar, concrete, sand, stone or other available material used as protection against erosion.
saltation load:	Sediment bounced along the streambed by energy and turbulence of flow, and by other moving particles.
sand:	A rock fragment whose diameter is in the range of 0.062 to 2.0 mm.
scour:	Erosion of streambed or bank material due to flowing water; often considered as being localized (see local scour, contraction scour, total scour).
sediment or fluvial sediment:	Fragmental material transported, suspended, or deposited by water.
sediment concentration:	Weight or volume of sediment relative to the quantity of transporting (or suspending) fluid.
sediment discharge:	The quantity of sediment that is carried past any cross section of a stream in a unit of time. Discharge may be limited to certain sizes of sediment or to a specific part of the cross section.
sediment load:	Amount of sediment being moved by a stream.
sediment yield:	The total sediment outflow from a watershed or a drainage area at a point of reference and in a specified time period. This outflow is equal to the sediment discharge from the drainage area.
seepage:	The slow movement of water through small cracks and pores of the bank material.
seiche:	Long-period oscillation of a lake or similar body of water.
semi-diurnal tide	Tides with an approximate tidal period of 12 hours.
set-up:	Raising of water level due to wind action.
set-up, wave:	Height to which water rises above still-water elevation as a result of storm wind effects.
shallow water (for waves):	Water of such a depth that waves are noticeably affected by bottom conditions; customarily, water shallower than half the wavelength.
shear stress:	See unit shear force.
shoal:	A relatively shallow submerged bank or bar in a body of water.

sill:	(a) A structure built under water, across the deep pools of a stream with the aim of changing the depth of the stream; (b) A low structure built across an effluent stream, diversion channel or outlet to reduce flow or prevent flow until the main stream stage reaches the crest of the structure.
silt:	A particle whose diameter is in the range of 0.004 to 0.062 mm.
sinuosity:	The ratio between the thalweg length and the valley length of a stream.
slope (of channel or stream):	Fall per unit length along the channel centerline or thalweg.
slope protection:	Any measure such as riprap, paving, vegetation, revetment, brush or other material intended to protect a slope from erosion, slipping or caving, or to withstand external hydraulic pressure.
sloughing:	Sliding or collapse of overlying material; same ultimate effect as caving, but usually occurs when a bank or an underlying stratum is saturated.
slope-area method:	A method of estimating unmeasured flood discharges in a uniform channel reach using observed high-water levels.
slump:	A sudden slip or collapse of a bank, generally in the vertical direction and confined to a short distance, probably due to the substratum being washed out or having become unable to bear the weight above it.
soil-cement:	A designed mixture of soil and Portland cement compacted at a proper water content to form a blanket or structure that can resist erosion.
sorting:	Progressive reduction of size (or weight) of particles of the sediment load carried down a stream.
spill-through abutment:	A bridge abutment having a fill slope on the streamward side. The term originally referred to the "spill-through" of fill at an open abutment but is now applied to any abutment having such a slope.
spread footing:	A pier or abutment footing that transfers load directly to the earth.
spur:	A permeable or impermeable linear structure that projects into a channel from the bank to alter flow direction, induce deposition, or reduce flow velocity along the bank.
spur dike:	See guide bank.

stability:	A condition of a channel when, though it may change slightly at different times of the year as the result of varying conditions of flow and sediment charge, there is no appreciable change from year to year; that is, accretion balances erosion over the years.
stable channel:	A condition that exists when a stream has a bed slope and cross section which allows its channel to transport the water and sediment delivered from the upstream watershed without aggradation, degradation, or bank erosion (a graded stream).
stage:	Water-surface elevation of a stream with respect to a reference elevation.
still-water elevation:	Flood height to which water rises as a result of barometric pressure changes occurring during a storm event.
stone riprap:	Natural cobbles, boulders, or rock dumped or placed as protection against erosion.
stream:	A body of water that may range in size from a large river to a small rill flowing in a channel. By extension, the term is sometimes applied to a natural channel or drainage course formed by flowing water whether it is occupied by water or not.
streambank erosion:	Removal of soil particles or a mass of particles from a bank surface due primarily to water action. Other factors such as weathering, ice and debris abrasion, chemical reactions, and land use changes may also directly or indirectly lead to bank erosion.
streambank failure:	Sudden collapse of a bank due to an unstable condition such as removal of material at the toe of the bank by scour.
streambank protection:	Any technique used to prevent erosion or failure of a streambank.
storm surge:	Coastal flooding phenomenon resulting from wind and barometric changes. The storm surge is measured by subtracting the astronomical tide elevation from the total flood elevation (Hurricane surge).
storm tide:	Coastal flooding resulting from combination of storm surge and astronomical tide (often referred to as storm surge)
suspended sediment discharge:	The quantity of sediment passing through a stream cross section above the bed layer in a unit of time suspended by the turbulence of flow (suspended load).
sub-bed material:	Material underlying that portion of the streambed which is subject to direct action of the flow. Also, substrate.

subcritical, supercritical flow:	Open channel flow conditions with Froude Number less than and greater than unity, respectively.
tetrahedron:	Component of river-training works made of six steel or concrete struts fabricated in the shape of a pyramid.
tetrapod:	Bank protection component of precast concrete consisting of four legs joined at a central joint, with each leg making an angle of 109.5° with the other three.
thalweg:	The line extending down a channel that follows the lowest elevation of the bed.
tidal amplitude:	Generally, half of tidal range.
tidal cycle:	One complete rise and fall of the tide.
tidal day:	Time of rotation of the earth with respect to the moon. Assumed to equal approximately 24.84 solar hours in length.
tidal inlet:	A channel connecting a bay or estuary to the ocean.
tidal passage:	A tidal channel connected with the ocean at both ends.
tidal period:	Duration of one complete tidal cycle. When the tidal period equals the tidal day (24.84 hours), the tide exhibits diurnal behavior. Should two complete tidal periods occur during the tidal day, the tide exhibits semi-diurnal behavior.
tidal prism:	Volume of water contained in a tidal bay, inlet or estuary between low and high tide levels.
tidal range:	Vertical distance between specified low and high tide levels.
tidal scour:	Scour at bridges over tidal waterways, i.e., in the coastal zone.
tidal waterways:	A generic term which includes tidal inlets, estuaries, bridge crossings to islands or between islands, inlets to bays, crossings between bays, tidally affected streams, etc.
tides, astronomical:	Rhythmic diurnal or semi-diurnal variations in sea level that result from gravitational attraction of the moon and sun and other astronomical bodies acting on the rotating earth. Also, daily tides.
tieback:	Structure placed between revetment and bank to prevent flanking.
timber or brush mattress:	A revetment made of brush, poles, logs, or lumber interwoven or otherwise lashed together. The completed mattress is then placed on the bank of a stream and weighted with ballast.

toe of bank:	That portion of a stream cross section where the lower bank terminates and the channel bottom or the opposite lower bank begins.
toe protection:	Loose stones laid or dumped at the toe of an embankment, groin, etc., or masonry or concrete wall built at the junction of the bank and the bed in channels or at extremities of hydraulic structures to counteract erosion.
total scour:	The sum of long-term degradation, general (contraction) scour, and local scour.
total sediment load:	The sum of suspended load and bed load or the sum of bed material load and wash load of a stream (total load).
tractive force:	The drag or shear on a streambed or bank caused by passing water which tends to move soil particles along with the streamflow.
trench-fill revetment:	Stone, concrete, or masonry material placed in a trench dug behind and parallel to an eroding streambank. When the erosive action of the stream reaches the trench, the material placed in the trench armors the bank and thus retards further erosion.
tsunami:	Long-period ocean wave resulting from earthquake, other seismic disturbances or submarine landslides.
turbulence:	Motion of fluids in which local velocities and pressures fluctuate irregularly in a random manner as opposed to laminar flow where all particles of the fluid move in distinct and separate lines.
ultimate scour:	The maximum depth of scour attained for a given flow condition. May require multiple flow events and in cemented or cohesive soils may be achieved over a long time period.
uniform flow:	Flow of constant cross section and velocity through a reach of channel at a given time. Both the energy slope and the water slope are equal to the bed slope under conditions of uniform flow.
unit discharge:	Discharge per unit width (may be average over a cross section, or local at a point).
unit shear force (shear stress):	The force or drag developed at the channel bed by flowing water. For uniform flow, this force is equal to a component of the gravity force acting in a direction parallel to the channel bed on a unit wetted area. Usually in units of stress, Pa (N/m^2) or (lb/ft^2).
unsteady flow:	Flow of variable discharge and velocity through a cross section with respect to time.

upper bank:	The portion of a streambank having an elevation greater than the average water level of the stream.
velocity:	The time rate of flow usually expressed in m/s (ft/sec). The average velocity is the velocity at a given cross section determined by dividing discharge by cross-sectional area.
vertical abutment:	An abutment, usually with wingwalls, that has no fill slope on its streamward side.
vortex:	Turbulent eddy in the flow generally caused by an obstruction such as a bridge pier or abutment (e.g., horseshoe vortex).
wandering channel:	A channel exhibiting a more or less non-systematic process of channel shifting, erosion and deposition, with no definite meanders or braided pattern.
wandering thalweg:	A thalweg whose position in the channel shifts during floods and typically serves as an inset channel that conveys all or most of the stream flow at normal or lower stages.
wash load:	Suspended material of very small size (generally clays and colloids) originating primarily from erosion on the land slopes of the drainage area and present to a negligible degree in the bed itself.
watershed:	See drainage basin.
waterway opening width (area):	Width (area) of bridge opening at (below) a specified stage, measured normal to the principal direction of flow.
wave period:	Time interval between arrivals of successive wave crests at a point.
weephole:	A hole in an impermeable wall or revetment to relieve the neutral stress or pore pressure in the soil.
windrow revetment:	A row of stone placed landward of the top of an eroding streambank. As the windrow is undercut, the stone is launched downslope, thus armoring the bank.
wire mesh:	Wire woven to form a mesh; where used as an integral part of a countermeasure, openings are of suitable size and shape to enclose rock or broken concrete or to function on fence-like spurs and retards.

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CHAPTER 1

INTRODUCTION

1.1 PURPOSE

The purpose of this document is to provide guidelines for the following:

1. Designing new and replacement bridges to resist scour
2. Evaluating existing bridges for vulnerability to scour
3. Inspecting bridges for scour
4. Improving the state-of-practice of estimating scour at bridges

1.2 BACKGROUND

The most common cause of bridge failures is from floods scouring bed material from around bridge foundations. Scour is the engineering term for the erosion caused by water of the soil surrounding a bridge foundation (piers and abutments). During the spring floods of 1987, 17 bridges in New York and New England were damaged or destroyed by scour. In 1985, 73 bridges were destroyed by floods in Pennsylvania, Virginia, and West Virginia. A 1973 national study for the Federal Highway Administration (FHWA) of 383 bridge failures caused by catastrophic floods showed that 25 percent involved pier damage and 75 percent involved abutment damage.(1) A second more extensive study in 1978 indicated local scour at bridge piers to be a problem about equal to abutment scour problems.(2) A number of case histories on the causes and consequences of scour at major bridges are presented in Transportation Research Record 950.(3)

From available information, the 1993 flood in the upper Mississippi basin, caused 23 bridge failures for an estimated damage of $15 million. The modes of bridge failures were 14 from abutment scour, two from pier scour, three from pier and abutment scour, two from lateral bank migration, one from debris load, and one from unknown cause.(4)

In the 1994 flooding from storm Alberto in Georgia, there were over 500 state and locally owned bridges with damage attributed to scour. Thirty-one of state-owned bridges experienced from 15 to 20 feet of contraction scour and/or long-term degradation in addition to local scour. These bridges had to be replaced. Of more than 150 bridges identified as scour damaged, the Georgia Department of Transportation (GADOT) also recommended that 73 non-federal aid bridges be repaired or replaced. Total damage to the GADOT highway system was approximately $130 million.(4)

The American Association of State Highway and Transportation Officials (AASHTO) standard specifications for highway bridges has the following requirements to address the problem of stream stability and scour.(5)

- Hydraulic studies are a necessary part of the preliminary design of a bridge and should include. . .estimated scour depths at piers and abutments of proposed structures.

- The probable depth of scour shall be determined by subsurface exploration and hydraulic studies. Refer to Article 1.3.2 and FHWA Hydraulic Engineering Circular (HEC) 18 for general guidance regarding hydraulic studies and design.

- . . .in all cases, the pile length shall be determined such that the design structural load may be safely supported entirely below the probable scour depth.

1.3 COMPREHENSIVE ANALYSIS

This manual is part of a set of HECs issued by FHWA to provide guidance for bridge scour and stream stability analyses. The three manuals in this set are:

HEC-18	Evaluating Scour at Bridges
HEC-20	Stream Stability at Highway Structures[6]
HEC-23	Bridge Scour and Stream Instability Countermeasures[7]

The Flow Chart of Figure 1.1 illustrates graphically the interrelationship between these three documents and emphasizes that they should be used as a set. A comprehensive scour analysis or stability evaluation should be based on information presented in all three documents.

While the flow chart does not attempt to present every detail of a complete stream stability and scour evaluation, it has sufficient detail to show the major elements in a complete analysis, the logical flow of a typical analysis or evaluation, and the most common decision points and feedback loops. It clearly shows how the three documents tie together, and recognizes the differences between design of a new bridge and evaluation of an existing bridge.

The HEC-20 block of the flow chart outlines initial data collection and site reconnaissance activities leading to an understanding of the problem, evaluation of river system stability and potential future response. The HEC-20 procedures include both qualitative and quantitative geomorphic and engineering analysis techniques which help establish the level of analysis necessary to solve the stream instability and scour problem for design of a new bridge, or for the evaluation of an existing bridge that may require rehabilitation or countermeasures. The "Classify Stream," "Evaluate Stability," and "Assess Response" portions of the HEC-20 block are expanded in HEC-20 into a six-step Level 1 and an eight-step Level 2 analysis procedure. In some cases, the HEC-20 analysis may be sufficient to determine that stream instability or scour problems do not exist, i.e., the bridge has a "low risk" of failure regarding scour susceptibility.

In most cases, the analysis or evaluation will progress to the HEC-18 block of the flow chart. Here more detailed hydrologic and hydraulic data are developed, with the specific approach determined by the level of complexity of the problem and waterway characteristics (e.g., tidal or riverine). The "Scour Analysis" portion of the HEC-18 block encompasses a seven-step specific design approach which includes evaluation of the components of total scour (see Chapter 3).

Since bridge scour evaluation requires multidisciplinary inputs, it is often advisable for the hydraulic engineer to involve structural and geotechnical engineers at this stage of the analysis. **Once the total scour prism is plotted, then all three disciplines must be involved in a determination of structural stability**.

For a new bridge design, if the structure is stable the design process can proceed to consideration of environmental impacts, cost, constructability, and maintainability. If the structure is unstable, revise the design and repeat the analysis. For an existing bridge, a finding of structural stability at this stage will result in a "low risk" evaluation, with no further action required. However, a Plan of Action should be developed for an unstable existing bridge (scour critical) to correct the problem as discussed in Chapter 12 and HEC-23.[7]

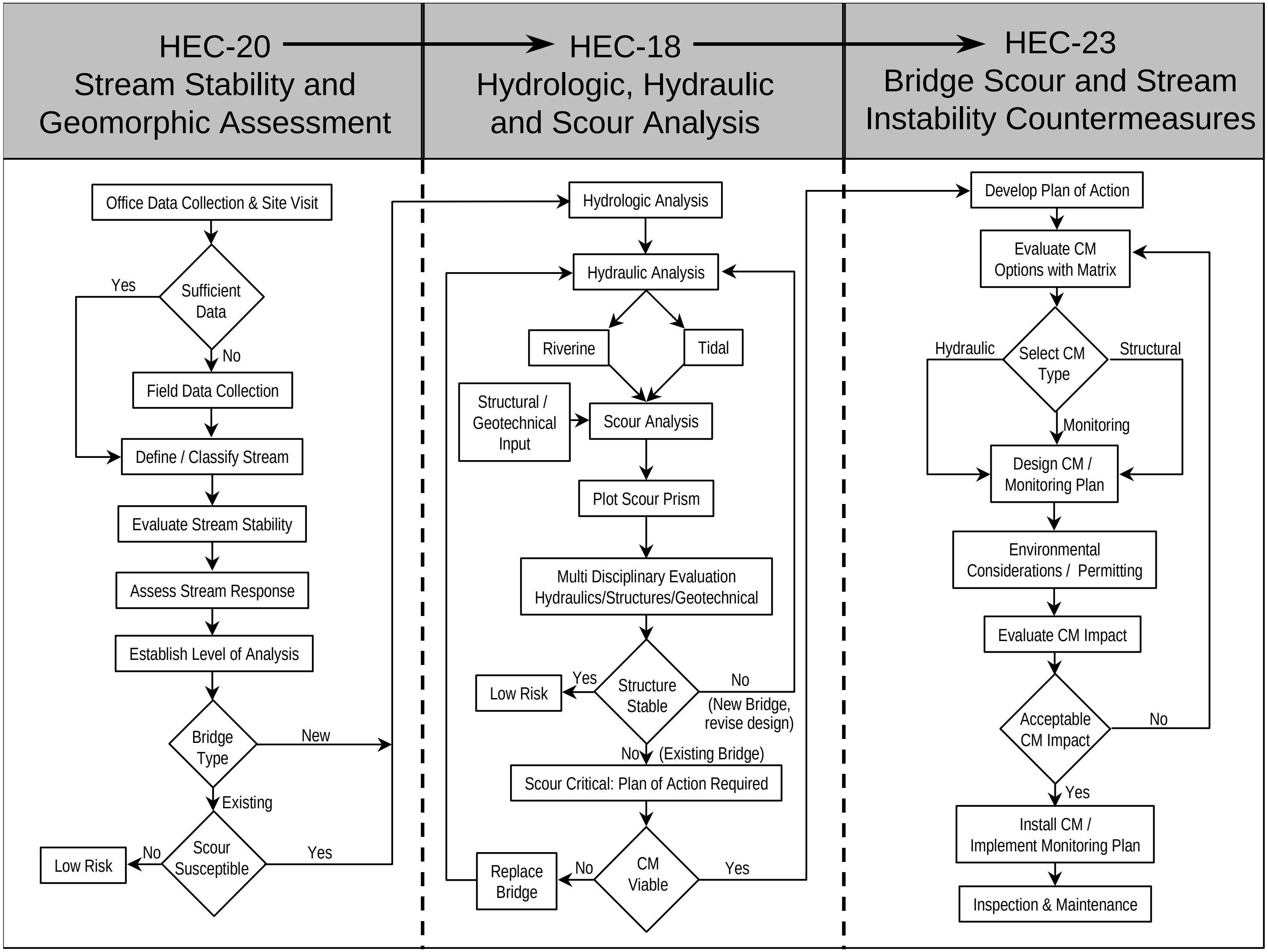

Figure 1.1. Flow chart for scour and stream stability analysis and evaluation.

The scour problem may be so serious that installing countermeasures would not provide a viable solution and a replacement or substantial bridge rehabilitation would be required. If countermeasures would correct the stream instability or scour problem at a reasonable cost and with acceptable environmental impacts, the analysis would progress to the HEC-23 block of the flow chart.

HEC-23 provides a range of resources to support bridge scour or stream instability countermeasure selection and design. A countermeasure matrix in HEC-23 presents a variety of countermeasures that have been used by State departments of transportation (DOTs) to control scour and stream instability at bridges. The matrix is organized to highlight the various groups of countermeasures and identifies distinctive characteristics of each countermeasure. The matrix identifies most countermeasures used and lists information on their functional applicability to a particular problem, their suitability to specific river environments, the general level of maintenance resources required, and which DOTs have experience with specific countermeasures. Finally, a reference source for design guidelines is noted.

HEC-23 includes specific design guidelines for the most common (and some uncommon) countermeasures used by DOTs, or references to sources of design guidance. Inherent in the design of any countermeasure is an evaluation of potential environmental impacts, permitting for countermeasure installation, and redesign, if necessary, to meet environmental requirements. As shown in the flow chart, to be effective most countermeasures will require a monitoring plan, inspection, and maintenance.

1.4 MANUAL ORGANIZATION

The procedures presented in this document contain the state-of-knowledge and practice for dealing with scour at highway bridges.

- Chapter 1 gives the background of the scour problem, a flowchart for a comprehensive analysis using HEC-18, HEC-20, and HEC-23, organization of this manual and improvements needed in the state-of-knowledge of scour.

- Chapter 2 gives recommendations for designing bridges to resist scour.

- Basic concepts and definitions are presented in Chapter 3.

- Methods for estimating long-term aggradation and degradation are given in Chapter 4.

- Chapter 5 provides procedures and equations for determining contraction scour and discusses other general scour conditions.

- Chapter 6 provides equations for calculating and evaluating local scour depths at piers.

- Chapter 7 discusses local scour at abutments and the equations for predicting scour depths at abutments.

- Chapter 8 provides a comprehensive example of scour analysis for a river crossing.

- Chapter 9 provides an introduction to tidal processes and scour analysis methods for bridges over tidal waterways.

- Chapter 10 explains how the National Bridge Scour Evaluation program determines the vulnerability of existing bridges to scour and gives the status of the program.

- Chapter 11 explains how the National Scour Evaluation program relates to the National Bridge Inspection Standards (NBIS). It also presents guidelines for inspecting bridges for scour.

- Chapter 12 explains the need for and details of a Plan of Action to protect a bridge that has been determined to be scour critical.

1.5 OBJECTIVES OF A BRIDGE SCOUR EVALUATION PROGRAM

The need to minimize future flood damage to the nation's bridges requires that additional attention be devoted to developing and implementing improved procedures for designing and inspecting bridges for scour.(8) Approximately 83 percent of the 583,000 bridges in the National Bridge Inventory are built over waterways. Statistically, we can expect hundreds of these bridges to experience floods in the magnitude of a 100-year flood or greater each year. Because it is not economically feasible to construct all bridges to resist all conceivable floods, or to install scour countermeasures at all existing bridges to ensure absolute invulnerability from scour damage, some risks of failure from future floods may have to be accepted. **However, every bridge over water, whether existing or under design, should be assessed as to its vulnerability to floods in order to determine the prudent measures to be taken.** The added cost of making a bridge less vulnerable to scour is small when compared to the total cost of a failure which can easily be two to ten times the cost of the bridge itself. Moreover, the need to ensure public safety and minimize the adverse effects resulting from bridge closures requires our best efforts to improve the state-of-practice for designing and maintaining bridge foundations to resist the effects of scour. **The hydraulic design of bridge waterways is typically based on flood frequencies somewhat less than those recommended for scour analysis in this publication.**

The procedures presented in this manual serve as guidance for implementing the recommendations contained in the FHWA Technical Advisory T5140.23 entitled, "Evaluating Scour at Bridges."(9) The recommendations have been developed to summarize the essential elements which should be addressed in developing a comprehensive scour evaluation program. A key element of the program is the identification of scour-critical bridges which will be entered into the National Bridge Inventory using the FHWA document "Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges."(10)

1.6 DUAL SYSTEM OF UNITS

This edition of HEC-18 uses dual units (SI metric and English). The "English" system of units as used throughout this manual refers to U.S. customary units. **In Appendix A, the metric (SI) unit of measurement is explained. The conversion factors, physical properties of water in the SI and English systems of units, sediment particle size grade scale, and some common equivalent hydraulic units are also given**. This edition uses for the unit of length the meter (m) or foot (ft); of mass the kilogram (kg) or slug; of weight/force the newton (N) or pound (lb); of pressure the Pascal (Pa, N/m^2) or (lb/ft^2); and of temperature the degree centigrade (°C) or Fahrenheit (°F). The unit of time is the same in SI as in English system (seconds, s). Sediment particle size is given in millimeters (mm), but in calculations the decimal equivalent of millimeters in meters is used (1 mm = 0.001 m) or for the English system feet (ft). The value of some hydraulic engineering terms used in the text in SI units and their equivalent English units are given in Table 1.1.

Table 1.1. Commonly Used Engineering Terms in SI and English Units.		
Term	SI Units	English Units
Length	1 m	3.28 ft
Volume	1 m^3	35.31 ft^3
Discharge	1 m^3/s	35.31 ft^3/s
Acceleration of Gravity	9.81 m/s^2	32.2 ft/s^2
Unit Weight of Water	9800 N/m^3	62.4 lb/ft^3
Density of Water	1000 kg/m^3	1.94 $slugs/ft^3$
Density of Quartz	2647 kg/m^3	5.14 $slugs/ft^3$
Specific Gravity of Quartz	2.65	2.65
Specific Gravity of Water	1	1
Temperature	°C = 5/9 (°F - 32)	°F

1.7 STATE-OF-KNOWLEDGE AND PRACTICE FOR ESTIMATING SCOUR AT BRIDGES

Some of the problems associated with estimating scour and providing cost-effective and safe designs are being addressed in research and development programs of the FHWA and individual DOTs. The following sections detail the most pressing research needs.

1. **Field Measurements of Scour**. The current equations and methods for estimating scour at bridges are based primarily on laboratory research. Very little field data have been collected to verify the applicability and accuracy of the various design procedures for the range of soil conditions, stream flow conditions, and bridge designs encountered throughout the United States. In particular, DOTs are encouraged to initiate studies for the purpose of obtaining field measurements of scour and related hydraulic conditions at bridges for evaluating, verifying, and improving existing scour prediction methods. In excess of 20 states have initiated cooperative studies with the Water Resources Division of the U.S. Geological Survey (USGS) to collect scour data at existing bridges. A model cooperative agreement with the USGS for purposes of conducting a scour study was included in the FHWA guidance "Interim Procedures for Evaluating Scour at Bridges," which accompanied the September 1988 FHWA Technical Advisory.[11, 9]

2. **Scour Monitoring and Measurement Equipment**. Many bridges in the United States were constructed prior to the development of scour estimation procedures. Some of these bridges have foundations which are vulnerable to scour; however, it is not economically feasible to repair or replace all of these bridges. Therefore, these bridges need to be monitored during floods and closed before they fail. The FHWA, in cooperation with DOTs and the Transportation Research Board, has conducted research to develop scour monitoring and measuring instruments.[12] This research has developed several instruments for scour monitoring and measurement (see Chapter 7, HEC-23).[7] However, there is a need for additional research to develop additional instrumentation and equipment to measure scour for research and to indicate when a bridge is in danger of collapsing due to scour.

3. **Equipment and Methods to Determine Unknown Foundations**. Many of the 575,000 bridges have Unknown foundations. Research sponsored by FHWA, in cooperation with DOTs and the Transportation Research Board has investigated techniques and instruments to identify the type and depth of unknown foundations for most existing

bridges. Additional research is needed to perfect the methods and instruments and to develop alternative methods and equipment (Appendix L).

4. **Hydraulic Variables for Scour Computations**. Advances have been made in developing computational software to establish hydraulic variables for scour computations, including 1- and 2-dimensional, steady and unsteady models. Recent research has provided guidance for applying these models to estimating scour for coastal (tidal) bridges.(13) Most, if not all, of the commonly used scour prediction equations have been incorporated into these models. However, applications methodologies are required to facilitate the use of more appropriate hydraulic variables that can be obtained from more sophisticated computer models. World wide web sites providing hydraulic models applicable to scour computations include:

   - www.fhwa.dot.gov/bridge/hydsoft.htm

   - www.hec.usace.army.mil/software/index.html

5. **Pressure Flow**. Research sponsored by FHWA has developed equations and methods to determine pier and abutment local scour depths when a bridge is submerged (pressure flow).(14) A regression equation for vertical contraction scour is available, but combinations of vertical and lateral contraction scour need to be investigated.

6. **Field and Laboratory Studies of Scour**. Laboratory studies are needed to better understand certain elements of the scour processes and develop alternate and improved scour countermeasures. Only through controlled experiments can the effect of the variables and parameters associated with scour be determined. Through these efforts, scour prediction equations can be improved and additional design methods for countermeasures developed. Results from these laboratory experiments must be verified by ongoing field measurements of scour.

   Laboratory and field research is needed to:

   a. Improve methods to predict scour depths associated with pressure flow,

   b. Improve equations for abutment scour,

   c. Improve methods for estimating scour when abutments are set back from the channel with overbank flow,

   d. Conduct fundamental research on the mechanics of riverine and tidal scour,

   e. Determine methods to predict scour depths when there is ice or debris buildup at a pier or abutment,

   f. Improve our knowledge of the influence of graded, armored, or cohesive bed material on maximum local scour at piers and abutments,

   g. Improve methods for determining the size and placement (elevation, width, and location) of riprap in the scour hole to protect piers and abutments,

   h. Determine the width of scour hole as a function of scour depth and bed material size,

i. Improve our knowledge of the effects of flow depth and velocity on scour depths,

j. Improve our understanding of the bridge scour failure mechanism which would combine the various scour components (pier, abutment, contraction, lateral migration, degradation) into an estimate of the scoured cross section under the bridge,

k. Improve methods to predict the effect of flow angle of attack against a pier or abutment on scour depth,

l. Determine the effect of wide piers and variable pier widths on scour depths,

m. Determine the impact of overlapping scour holes, and

n. Determine scour depths in structures designed as bottomless culverts, that is culverts founded on spread footings and placed on erodible soil.

CHAPTER 2

DESIGNING BRIDGES TO RESIST SCOUR

2.1 DESIGN PHILOSOPHY AND CONCEPTS

Bridge foundations should be designed to withstand the effects of scour without failing for the worst conditions resulting from floods equal to the 100-year flood, or a smaller flood if it will cause scour depths deeper than the 100-year flood. Bridge foundations should be checked to ensure that they will not fail due to scour resulting from the occurrence of a superflood in order of magnitude of a 500-year flood. This requires careful evaluation of the hydraulic, structural, and geotechnical aspects of bridge foundation design.

Guidance in this chapter is based on the following concepts:

1. The foundation should be designed by an interdisciplinary team of engineers with expertise in hydraulic, geotechnical, and structural design.

2. Hydraulic studies of bridge sites are a necessary part of a bridge design. These studies should address both the sizing of the bridge waterway opening and the design of the foundations to be safe from scour. The scope of the analysis should be commensurate with the importance of the highway and consequences of failure.

3. Consideration must be given to the limitations and gaps in existing knowledge when using currently available formulas for estimating scour. **The designer needs to apply engineering judgment in comparing results obtained from scour computations with available hydrologic and hydraulic data to achieve a reasonable and prudent design.** Such data should include:

    a. Performance of existing structures during past floods

    b. Effects of regulation and control of flood discharges

    c. Hydrologic characteristics and flood history of the stream and similar streams

    d. Whether the bridge is structurally continuous

4. The principles of economic analysis and experience with actual flood damage indicate that it is almost always cost-effective to provide a foundation that will not fail, even from a very large flood event or superflood. Generally, occasional damage to highway approaches from rare floods can be repaired quickly to restore traffic service. On the other hand, a bridge which collapses or suffers major structural damage from scour can create safety hazards to motorists as well as significant social impacts and economic losses over a long period of time. Aside from the costs to the DOTs of replacing or repairing the bridge and constructing and maintaining detours, there can be significant costs to communities or entire regions due to additional detour travel time, inconvenience, and lost business opportunities. Therefore, a higher hydraulic standard is warranted for the design of bridge foundations to resist scour than is usually required for sizing of the bridge waterway. This concept is reflected in the following design procedure.

2.2 GENERAL DESIGN PROCEDURE

The general design procedure for scour outlined in the following steps is recommended for determining bridge type, size, and location (TS&L) of substructure units:

Step 1. Select the flood event(s) that are expected to produce the most severe scour conditions. Experience indicates that this is likely to be the 100-year flood or the overtopping flood when it is less than the 100-year flood. Check the 100-year flood or the overtopping flood (if less than the 100-year flood) and other flood events if there is evidence that such events would create deeper scour than the 100-year or overtopping floods. Overtopping refers to flow over the approach embankment(s), the bridge itself, or both. See Appendix B for a discussion of extreme event combinations.

Step 2. Develop water surface profiles for the flood flows in Step 1, taking care to evaluate the range of potential tailwater conditions downstream of the bridge which could occur during these floods. The FHWA microcomputer software WSPRO, is recommended for this task.[15] The U.S. Army Corps of Engineers (USACE) Hydrologic Engineering Center's River Analysis System (HEC-RAS) can also be used.[16, 17]

Step 3. Using the seven-step Specific Design Approach in Section 2.4, estimate total scour for the worst condition from Steps 1 and 2 above. The resulting scour from the selected flood event should be considered in the design of a foundation. For this condition, minimum geotechnical safety factors commonly accepted by DOTs should be applied. For example, for pile design in friction, a commonly applied factor of safety ranges from two to three, for the 100-year or overtopping flood.

Step 4. Plot the total scour depths obtained in Step 3 on a cross section of the stream channel and floodplain at the bridge site.

Step 5. Evaluate the results obtained in Steps 3 and 4. Are they reasonable, considering the limitations in current scour estimating procedures? The scour depth(s) adopted may differ from the equation value(s) based on engineering judgment.

Step 6. Evaluate the bridge TS&L on the basis of the scour analysis performed in Steps 3 through 5. Modify the TS&L as necessary.

a. Visualize the overall flood flow pattern at the bridge site for the design conditions. Use this mental picture to identify those bridge elements most vulnerable to flood flows and resulting scour.

b. The extent of protection to be provided should be determined by:

- Degree of uncertainty in the scour prediction method

- Potential for and consequences of failure

Step 7. Perform the bridge foundation analysis on the basis that all streambed material in the scour prism above the total scour line (Step 4) has been removed and is not available for bearing or lateral support. All foundations should be designed in accordance with the AASHTO Standard Specifications for Highway Bridges.[5] In the case of a pile foundation, the piling should be designed for additional lateral restraint and column action because of the increase in unsupported pile length after scour. In areas where the local scour is confined to the proximity of the footing, the lateral ground stresses on the pile length which remains embedded may not be significantly reduced from the pre-local scour conditions.

a. Spread Footings On Soil

- Insure that the top of the footing is below the sum of the long-term degradation, contraction scour, and lateral migration

- Place the bottom of the footing below the total scour line from Step 4

- The top of the footing can act as a local scour arrester

b. Spread Footings On Rock Highly Resistant To Scour

Place the bottom of the footing directly on the cleaned rock surface for massive rock formations (such as granite) that are highly resistant to scour. Small embedments (keying) should be avoided since blasting to achieve keying frequently damages the sub-footing rock structure and makes it more susceptible to scour. If footings on smooth massive rock surfaces require lateral constraint, steel dowels should be drilled and grouted into the rock below the footing level.

c. Spread Footings On Erodible Rock

Weathered or other potentially erodible rock formations need to be carefully assessed for scour. An engineering geologist familiar with the area geology should be consulted to determine if rock or soil or other criteria should be used to calculate the support for the spread footing foundation. The decision should be based on an analysis of intact rock cores, including rock quality designations and local geology, as well as hydraulic data and anticipated structure life. An important consideration may be the existence of a high quality rock formation below a thin weathered zone. For deep deposits of weathered rock, the potential scour depth should be estimated (Steps 4 and 5) and the footing base placed below that depth. Excavation into weathered rock should be made with care. If blasting is required, light, closely spaced charges should be used to minimize overbreak beneath the footing level. Loose rock pieces should be removed and the zone filled with clean concrete. In any event, the final footing should be poured in contact with the sides of the excavation for the full designed footing thickness to minimize water intrusion below footing level. Guidance on scourability of rock formations is given in FHWA memorandum "Scourability of Rock Formations" dated July 19, 1991[18] (see Appendix L).

d. Spread Footings Placed On Tremie Seals And Supported On Soil

- Insure that the top of the footing is below the sum of the long-term degradation, contraction scour, and lateral migration

- Place the bottom of the footing below the total scour line from Step 4

e. <u>For Deep Foundations (Drilled Shaft And Driven Piling) With Footings Or Caps</u>

   Placing the top of the footing or pile cap below the streambed a depth equal to the estimated long-term degradation and contraction scour depth will minimize obstruction to flood flows and resulting local scour. Even lower footing elevations may be desirable for pile supported footings when the piles could be damaged by erosion and corrosion from exposure to river or tidal currents. For more discussion on pile and drilled shaft foundations, see the manuals on Design and Construction of Driven Pile Foundations and Drilled Shafts.[19, 20]

f. <u>Stub Abutments on Piling</u>

   Stub abutments positioned in the embankment should be founded on piling driven below the elevation of the thalweg including long term degradation and contraction scour in the bridge waterway to assure structural integrity in the event the thalweg shifts and the bed material around the piling scours to the thalweg elevation.

Step 8. Repeat the procedure in Steps 2 through 6 above and calculate the scour for a superflood. It is recommended that this superflood (or check flood) be on the order of a 500-year event. However, flows greater or less than these suggested floods may be appropriate depending upon hydrologic considerations and the consequences associated with damage to the bridge. An overtopping flood less than the 500-year flood may produce the worst-case situation for checking the foundation design. The foundation design determined under Step 7 should be reevaluated for the superflood condition and design modifications made where required.

a. Check to make sure that the bottom of spread footings on soil or weathered rock is below the total scour depth for the superflood.

b. **All foundations should have a minimum factor of safety of 1.0 (ultimate load) under the superflood conditions.** Note that in actual practice, the calculations for step 8 would be performed concurrently with steps 1 through 7 for efficiency of operation.

2.3 DESIGN CONSIDERATIONS

2.3.1 General

1. Raise the bridge superstructure elevation above the general elevation of the approach roadways wherever practicable. This provides for overtopping of approach embankments and relief from the hydraulic forces acting at the bridge. This is particularly important for streams carrying large amounts of debris which could clog the waterway at the bridge.

It is recommended that the elevation of the lower cord of the bridge be increased a minimum of 0.9 m (3 ft) above the normal freeboard for the 100-year flood for streams that carry a large amount of debris.

2. Superstructures should be securely anchored to the substructure if buoyant, or if debris and ice forces are probable. Further, the superstructure should be shallow and open to minimize resistance to the flow where overtopping is likely.

3. Continuous span bridges withstand forces due to scour and resultant foundation movement better than simple span bridges. Continuous spans provide alternate load paths (redundancy) for unbalanced forces caused by settlement and/or rotation of the foundations. This type of structural design is recommended for bridges where there is a significant scour potential.

4. Local scour holes at piers and abutments may overlap one another in some instances. If local scour holes do overlap, the scour is indeterminate and may be deeper. The topwidth of a local scour hole on each side of the pier ranges from 1.0 to 2.8 times the depth of local scour. A topwidth value of 2.0 times the depth of local scour on each side of a pier is suggested for practical applications.

5. For pile and drilled shaft supported substructures subjected to scour, a reevaluation of the foundation design may require a change in the pile or shaft length, number, cross-sectional dimension and type based on the loading and performance requirements and site-specific conditions.

6. At some bridge sites, hydraulics and traffic conditions may necessitate consideration of a bridge that will be partially or even totally inundated during high flows. This consideration results in pressure flow through the bridge waterway. Chapter 6 has a discussion on pressure flow scour for these cases.

2.3.2 Piers

1. Pier foundations on floodplains should be designed to the same elevation as pier foundations in the stream channel if there is a likelihood that the channel will shift its location over the life of the bridge.

2. Align piers with the direction of flood flows. Assess the hydraulic advantages of round piers, particularly where there are complex flow patterns during flood events.

3. Streamline piers to decrease scour and minimize potential for buildup of ice and debris. Use ice and debris deflectors where appropriate.

4. Evaluate the hazards of ice and debris buildup when considering use of multiple pile bents in stream channels. Where ice and debris buildup is a problem, consider the bent a solid pier for purposes of estimating scour. Consider use of other pier types where clogging of the waterway area could be a major problem.

5. Scour analyses of piers near abutments need to consider the potential of larger velocities and skew angles from the flow coming around the abutment.

2.3.3 Abutments

1. The equations used to estimate the magnitude of abutment scour were developed in a laboratory under ideal conditions and for the most part lack field verification. Because conditions in the field are different from those in the laboratory, these equations tend to over predict the magnitude of scour that may be expected to develop. Recognizing this, it is recommended that the abutment scour equations be used to develop insight as to the scour potential at an abutment. Engineering judgment must be used to determine if the abutment foundation should be designed to resist the computed local scour. As an alternate, abutment foundations should be designed for the estimated long-term degradation and contraction scour. Riprap and/or guide banks should be used to protect the abutment for this alternative. In summary, riprap or some other protection should always be used to protect the abutment from erosion. Proper design techniques and placement procedures for rock riprap and guide banks are discussed in HEC-23.[7]

2. Relief bridges, guide banks, and river training works should be used, where needed, to minimize the effects of adverse flow conditions at abutments.

3. Where ice build-up is likely to be a problem, set the toe of spill-through slopes or vertical abutments back from the edge of the channel bank to facilitate passage of the ice.

4. Wherever possible, use spill-through (sloping) abutments. Scour at spill-through abutments is about 50 percent of that of vertical wall abutments.

5. Riprap or a guide bank 15 m (50 ft) or longer, or other bank protection methods should be used on the downstream side of an abutment and approach embankment to protect them from erosion by the wake vortex.

2.3.4 Superstructures

The design of the superstructure has a significant impact on the scour of the foundations. Hydraulic forces that should be considered in the design of a bridge superstructure include buoyancy, drag, and impact from ice and floating debris. The configuration of the superstructure should be influenced by the highway profile, the probability of submergence, expected problems with ice and debris, and flow velocities, as well as the usual economic, structural and geometric considerations. Superstructures over waterways should provide structural redundancy, such as continuous spans (rather than simple spans).

<u>Buoyancy</u>. The weight of a submerged or partially submerged bridge superstructure is the weight of the superstructure less the weight of the volume of water displaced. The volume of water displaced may be much greater than the volume of the superstructure components if air is trapped between girders. Also, solid parapet rails and curbs on the bridge deck can increase the volume of water displaced and increase buoyant forces. The volume of air trapped under the superstructure can be reduced by providing holes (vents) through the deck between structural members. Superstructures should be anchored to piers to counter buoyant forces and to resist drag forces. Continuous span designs are also less susceptible to failure from buoyancy than simple span designs.

<u>Drag Forces</u>. Drag forces on a submerged or partially submerged superstructure can be calculated by Equation 2.1:

$$F_d = C_d \rho H \frac{V^2}{2} \tag{2.1}$$

where:

F_d = Drag force per unit of length of bridge, N/m (lb/ft)
C_d = Coefficient of drag (2.0 to 2.2)
ρ = Density of water, 1000 kg/m^3 (1.94 slugs/ft^3)
H = Depth of submergence, m (ft)
V = Velocity of flow, m/s (ft/s)

Floating Debris and Ice. Where bridges are destroyed by debris and ice, it usually is due to accumulations against bridge components. Waterways may be partially or totally blocked by ice and debris, creating hydraulic conditions that cause or increase scour at pier foundations and bridge abutments, structural damage from impact and uplift, and overtopping of roadways and bridges. Floating debris is a common hydraulic problem at highway stream crossings nation-wide. Debris hazards occur more frequently in unstable streams where bank erosion is active and in streams with mild to moderate slopes, as contrasted with headwater streams. Debris hazards are often associated with large floods, and most debris is derived locally along the streambanks upstream from the bridge. After being mobilized, debris typically moves as individual logs which tend to concentrate in the thalweg of the stream. It is possible to evaluate the abundance of debris upstream of a bridge crossing and then to implement mitigation measures, such as removal and or containment, to minimize potential problems during a major flood (see additional discussion in HEC-20, Chapter 4).[(6)]

Ice Forces. Superstructures may be subjected to impact forces from floating ice, static pressure from thermal movements or ice jams, or uplift from adhering ice in water of fluctuating levels. The latter is usually associated with relatively large bodies of water. Superstructures in these locations should normally be high enough to be unaffected. Research is needed to define the static and dynamic loads that can be expected from ice under various conditions of ice strength and streamflow.

In addition to forces imposed on bridge superstructures by ice loads, ice jams at bridges can cause exaggerated backwater and a sluicing action under the ice. There are numerous examples of foundation scour from this orifice flow under ice as well as superstructure damage and failure from ice forces. Accumulations of ice or drift may substantially increase local pier and abutment scour especially if they are allowed to extend down to near the channel bed. Ice also has serious effects on bank stability. For example, ice may form in bank stabilization materials, and large quantities of rock and other material embedded in the ice may be floated downstream and dumped randomly when the ice breaks up. Banks are subjected to piping forces during the drawdown of water surface elevation after the breakup.

Debris Forces. Information regarding methods for computing forces imposed on bridge superstructures by floating debris is also lacking despite the fact that debris causes or contributes to many failures. Floating debris may consist of logs, trees, house trailers, automobiles, storage tanks, lumber, houses, and many other items representative of floodplain usage. This complicates the task of computing impact forces since the mass and the resistance to crushing of the debris contribute to the impact force.

A general equation for computing impact forces is:

$$F = M\,dv / dt = \frac{MV^2}{2S} \tag{2.2}$$

where:

F	=	Impact imparted by the debris, N (lb)
M	=	Mass of the debris, kg (slugs)
S	=	Stopping distance, m (ft)
V	=	Velocity of the floating debris prior to impact, m/s (ft/s)

In addition to impact forces, a buildup of debris increases the effective depth of the superstructure and the drag coefficient may also be increased. Perhaps the most hazardous result of debris buildup is partial or total clogging of the waterway. This can result in a sluicing action of flow under the debris which can result in scour and foundation failure or a shift in the channel location from under the bridge.

2.4 SPECIFIC DESIGN APPROACH

The seven specific steps recommended for estimating scour at bridges are:

Step 1: Determine scour analysis variables

Step 2: Analyze long-term bed elevation change

Step 3: Compute the magnitude of contraction scour

Step 4: Compute other general scour depths.

Step 5: Compute the magnitude of local scour at piers

Step 6: Determine abutment foundation type, protection and elevation. Computation of local scour depths may be used to aid in this determination.

Step 7: Plot and evaluate the total scour depths as outlined in Steps 4 through 6 of the General Design Procedure in Section 2.2.

The engineer should evaluate how reasonable the individual estimates of general scour (contraction and other) and local scour depths are in Steps 3, 4, and 5 and evaluate the reasonableness of the total scour in Step 7. The results from this Specific Design Approach complete Steps 1 through 6 of Section 2.2. The design must now proceed to Steps 7 and 8 of the General Design Procedure in Section 2.2.

The procedures for each of the steps are discussed in the following sections with reference to specific chapters where detailed procedures and equations are given.

2.5 DETAILED PROCEDURES

2.5.1 Step 1: Determine Scour Analysis Variables

1. Determine the magnitude of the discharges for the floods in Steps 1 and 8 of the General Design Procedure in Section 2.2, including the overtopping flood when applicable. For guidance for a particular state in determining the magnitude of the 500-year flood, contact with the U.S. Geological Survey Water Resources District office is suggested. Experience has shown that the incipient overtopping discharge often puts the most stress on a bridge. However, special conditions (angle of attack, pressure flow, decrease in velocity or discharge resulting from high flows overtopping approaches or going through relief bridges, ice jams, etc.) may cause a more severe condition for scour with a flow smaller than the overtopping or 100-year flood.

2. Determine if there are existing or potential future factors that will produce a combination of high discharge and low tailwater control. Are there bedrock or other controls (old diversion structures, erosion control checks, other bridges, etc.) that might be lowered or removed? Are there dams or locks downstream that would control the tailwater elevation seasonally? Are there dams upstream or downstream that could control the elevation of the water surface at the bridge? Select the lowest reasonable downstream water-surface elevation and the largest discharge to estimate the greatest scour potential. Assess the distribution of the velocity and discharge per foot of width for the design flow and other flows through the bridge opening. Also, consider the contraction and expansion of the flow in the bridge waterway, as well as present conditions and anticipated future changes in the river.

3. Determine the water-surface profiles for the discharges judged to produce the most scour from step 1, using WSPRO, or HEC River Analysis System (HEC-RAS).[15, 16, 17] In some instances, the designer may wish to use BRI-STARS.[21] Hydraulic studies by the USACE, USGS, the Federal Emergency Management Agency (FEMA), etc. are potentially useful sources of hydraulic data to calibrate, verify, and evaluate results from WSPRO or HEC-RAS. The engineer should anticipate future conditions at the bridge, in the upstream watershed, and at downstream water-surface elevation controls as outlined in HEC-20.[6] From computer analysis and from other hydraulic studies, determine input variables such as the discharge, velocity and depth needed for the scour calculations.

4. Collect and summarize the following information as appropriate (see HEC-20 for a step-wise analysis procedure[6]).

    a. Boring logs to define geologic substrata at the bridge site

    b. Bed material size, gradation, and distribution in the bridge reach

    c. Existing stream and floodplain cross section through the reach

    d. Stream planform

    e. Watershed characteristics

    f. Scour data on other bridges in the area

g. Slope of energy grade line upstream and downstream of the bridge

h. History of flooding

i. Location of bridge site with respect to other bridges in the area, confluence with tributaries close to the site, bed rock controls, man-made controls (dams, old check structures, river training works, etc.), and confluence with another stream downstream

j. Character of the stream (perennial, flashy, intermittent, gradual peaks, etc.)

k. Geomorphology of the site (floodplain stream; crossing of a delta, youthful, mature or old age stream; crossing of an alluvial fan; meandering, straight or braided stream; etc.) (see HEC-20 and HDS 6)[6, 22]

l. Erosion history of the stream

m. Development history (consider present and future conditions) of the stream and watershed, collect maps, ground photographs, aerial photographs; interview local residents; check for water resource projects planned or contemplated

n. Sand and gravel mining from the streambed or floodplain up- and downstream from site

o. Other factors that could affect the bridge

p. Make a qualitative evaluation of the site with an estimate of the potential for stream movement and its effect on the bridge

2.5.2 Step 2: Analysis of Long-Term Bed Elevation Change

Using the information collected in Step 1, above, and procedures in HEC-20[6] and Chapter 4, determine the long-term trend in the streambed elevation.

2.5.3 Step 3: Compute the Magnitude of Contraction Scour

Using the information collected in Step 1, above, compute the magnitude of the contraction scour using the equations and procedures in Chapter 5.

2.5.4 Step 4: Determine the Magnitude of Other General Scour Components

Using the information collected in Step 1, above, determine the magnitude of other general scour components, if any, using the procedures discussed in Chapter 5.

2.5.5 Step 5: Compute the Magnitude of Local Scour at Piers

Using the information collected in Step 1, above, compute the magnitude of local pier scour using the equations and procedures in Chapter 6.

2.5.6 Step 6: Determine the Foundation Elevation for Abutments

Using the information collected in Step 1, above, compute the magnitude of abutment scour using the information and procedures in Chapter 7.

2.5.7 Step 7: Plot the Total Scour Depths and Evaluate the Design

Plot the Total Scour Depths. On the cross section of the stream channel or other general floodplain at the bridge crossing, plot the estimate of long-term bed elevation change, contraction scour, and local scour at the piers and abutments. Use a distorted scale so that the scour determinations will be easy to evaluate. Make a sketch of any planform changes (lateral stream channel movement due to meander migration, etc.) that might be reasonably expected to occur.

1. Long-term elevation changes may be either aggradation or degradation. However, only degradation is considered in scour computations.

2. Contraction or other general scour is then plotted from and below the long-term degradation line.

3. Local scour is then plotted from and below the contraction scour line.

4. Plot not only the depth of scour at each pier and abutment, but also the scour hole width. Use 2.0 times the depth of local scour, y_s, to estimate scour hole width on each side of the pier.

Evaluate the Total Scour Depths.

1. Evaluate whether the computed scour depths are reasonable and consistent with the design engineer's previous experience, and engineering judgment. If not, carefully review the calculations and design assumption in order to modify the depths. These modifications must reflect sound engineering judgment.

2. Evaluate whether the local scour holes from the piers or abutments overlap between spans. If so, local scour depths can be larger though indeterminate. For new or replacement bridges, the length of the bridge opening should be reevaluated and the opening increased or the number of piers decreased as necessary to avoid overlapping scour holes.

3. Evaluate other factors such as lateral movement of the stream, stream flow hydrograph, velocity and discharge distribution, movement of the thalweg, shifting of the flow direction, channel changes, type of stream, or other factors.

4. Evaluate whether the calculated scour depths appear too deep for the conditions in the field, relative to the laboratory conditions. **Abutment scour equations are for the worst-case conditions.** Rock riprap and/or a guide bank could be a more cost-effective solution than designing the abutment to resist the computed abutment scour depths.

   If the calculated scour depths appear too deep, consider recalculating the hydraulic variables after long-term degradation and/or contraction scour are accounted for. This may decrease the total scour depth.

5. Evaluate cost, safety, etc. Also, account for ice and/or debris effects.

6. In the design of bridge foundations, the bottom foundation elevation(s) should be at or below the total scour elevation(s) as discussed in Section 2.2.

Reevaluate the Bridge Design. Reevaluate the bridge design on the basis of the foregoing scour computations and evaluation. Revise the design as necessary. This evaluation should consider the following questions:

1. Is the waterway area large enough (e.g., is contraction scour too large)?

2. Are the piers too close to each other or to the abutments (i.e., do the scour holes overlap)? Estimate the topwidth of a scour hole on each side of a pier at 2.0 times the depth of scour. If scour holes overlap, local scour can be deeper.

3. Is there a need for relief bridges? Should they or the main bridge be larger?

4. Are bridge abutments properly aligned with the flow and located properly in regard to the stream channel and floodplain?

5. Is the bridge crossing of the stream and floodplain in a desirable location? If the location presents problems:

   a. Can it be changed?

   b. Can river training works, guide banks, abutment setback from the channel, or relief bridges serve to provide for an acceptable flow pattern at the bridge?

6. Is the hydraulic study adequate to provide the necessary information for foundation design?

   a. Are flow patterns complex?
   b. Should a 2-dimensional, water-surface profile model be used for analysis?
   c. Is the foundation design safe and cost-effective?
   d. Is a physical model study needed/warranted?

CHAPTER 3

BASIC CONCEPTS AND DEFINITIONS OF SCOUR

3.1 GENERAL

Scour is the result of the erosive action of flowing water, excavating and carrying away material from the bed and banks of streams and from around the piers and abutments of bridges. Different materials scour at different rates. Loose granular soils are rapidly eroded by flowing water, while cohesive or cemented soils are more scour-resistant. **However, ultimate scour in cohesive or cemented soils can be as deep as scour in sand-bed streams.** Under constant flow conditions, scour will reach maximum depth in sand- and gravel-bed material in hours; cohesive bed material in days; glacial till, sandstones, and shale in months; limestone in years, and dense granite in centuries. Under flow conditions typical of actual bridge crossings, several floods may be needed to attain maximum scour.

Determining the magnitude of scour is complicated by the cyclic nature of the scour process. Scour can be deepest near the peak of a flood, but hardly visible as floodwaters recede and scour holes refill with sediment.

Designers and inspectors need to carefully study site-specific subsurface information in evaluating scour potential at bridges, giving particular attention to foundations on rock. Massive rock formations with few discontinuities are highly resistant to scour during the lifetime of a typical bridge.

All of the equations for estimating contraction and local scour are based on laboratory experiments with limited field verification. However, contraction and local scour depths at piers as deep as computed by these equations have been observed in the field. The equations recommended in this document are considered to be the most applicable for estimating scour depths.

A factor in scour at highway crossings and encroachments is whether it is **clear-water** or **live-bed** scour. Clear-water scour occurs where there is no transport of bed material upstream of the crossing or encroachment or the material being transported from the upstream reach is transported through the downstream reach at less than the capacity of the flow. Live-bed scour occurs where there is transport of bed material from the upstream reach into the crossing or encroachment. This subject is discussed further in Section 3.4.

This document presents procedures, equations, and methods to analyze scour in both riverine and coastal areas. In riverine environments, scour results from flow in one direction (downstream). In coastal areas, highways that cross waterways and/or encroach longitudinally on them are subject to tidal fluctuation and scour may result from flow in two directions. In waterways influenced by tidal fluctuations, flow velocities do not necessarily decrease as scour occurs and the waterway area increases. In tidal waterways as waterway area increases, the discharge may increase. This is in sharp contrast to riverine waterways where the principle of flow continuity and a constant discharge requires that velocity be inversely proportional to the waterway area. **However, the methods and equations for determining stream instability, scour and associated countermeasures can be applied to both riverine and coastal streams.**[23,24] The difficulty in tidal streams is in determining the hydraulic parameters (such as discharge, velocity, and depth) that are to be used in the scour equations. Tidal scour is discussed in Chapter 9.

3.2 TOTAL SCOUR

Total scour at a highway crossing is comprised of three components:

1. Long-term aggradation and degradation of the river bed

2. General scour at the bridge

   a. Contraction scour
   b. Other general scour

3. Local scour at the piers or abutments

These three scour components are added to obtain the total scour at a pier or abutment. This assumes that each component occurs independent of the other. Considering the components additive adds some conservatism to the design. In addition, **lateral migration** of the stream must be assessed when evaluating total scour at bridge piers and abutments.

3.2.1 Aggradation and Degradation

Aggradation and degradation are long-term streambed elevation changes due to natural or man-induced causes which can affect the reach of the river on which the bridge is located. Aggradation involves the deposition of material eroded from the channel or watershed upstream of the bridge; whereas, degradation involves the lowering or scouring of the streambed due to a deficit in sediment supply from upstream.

3.2.2 General Scour

General scour is a lowering of the streambed across the stream or waterway bed at the bridge. This lowering may be uniform across the bed or non-uniform, that is, the depth of scour may be deeper in some parts of the cross section. General scour may result from contraction of the flow, which results in removal of material from the bed across all or most of the channel width, or from other general scour conditions such as flow around a bend where the scour may be concentrated near the outside of the bend. General scour is different from long-term degradation in that general scour may be cyclic and/or related to the passing of a flood.

3.2.3 Local Scour

Local scour involves removal of material from around piers, abutments, spurs, and embankments. It is caused by an acceleration of flow and resulting vortices induced by obstructions to the flow. Local scour can be either clear-water or live-bed scour.

3.2.4 Lateral Stream Migration

In addition to the types of scour mentioned above, naturally occurring lateral migration of the main channel of a stream within a floodplain may affect the stability of piers in a floodplain, erode abutments or the approach roadway, or change the total scour by changing the flow angle of attack at piers and abutments. Factors that affect lateral stream movement also affect the stability of a bridge foundation. These factors are the geomorphology of the stream, location of the crossing on the stream, flood characteristics, and the characteristics of the bed and bank materials (see HEC-20, and HDS 6).[6, 22]

The following sections provide a more detailed discussion of the various components of total scour.

3.3 LONG-TERM STREAMBED ELEVATION CHANGES (AGGRADATION AND DEGRADATION)

Long-term bed elevation changes may be the natural trend of the stream or the result of some modification to the stream or watershed. The streambed may be aggrading, degrading, or in relative equilibrium in the vicinity of the bridge crossing. Long-term aggradation and degradation do not include the cutting and filling of the streambed in the vicinity of the bridge that might occur during a runoff event (general and local scour). A long-term trend may change during the life of the bridge. These long-term changes are the result of modifications to the stream or watershed. Such changes may be the result of natural processes or human activities. The engineer must assess the present state of the stream and watershed and then evaluate potential future changes in the river system. From this assessment, the long-term streambed changes must be estimated. Methods to estimate long-term streambed elevation changes are discussed in Chapter 4.

3.4 CLEAR-WATER AND LIVE-BED SCOUR

There are two conditions for contraction and local scour: **clear-water** and **live-bed** scour. Clear-water scour occurs when there is no movement of the bed material in the flow upstream of the crossing or the bed material being transported in the upstream reach is transported in suspension through the scour hole at the pier or abutment at less than the capacity of the flow. At the pier or abutment the acceleration of the flow and vortices created by these obstructions cause the bed material around them to move. Live-bed scour occurs when there is transport of bed material from the upstream reach into the crossing. Live-bed local scour is cyclic in nature; that is, the scour hole that develops during the rising stage of a flood refills during the falling stage.

Typical clear-water scour situations include (1) coarse-bed material streams, (2) flat gradient streams during low flow, (3) local deposits of larger bed materials that are larger than the biggest fraction being transported by the flow (rock riprap is a special case of this situation), (4) armored streambeds where the only locations that tractive forces are adequate to penetrate the armor layer are at piers and/or abutments, and (5) vegetated channels or overbank areas.

During a flood event, bridges over streams with coarse-bed material are often subjected to clear-water scour at low discharges, live-bed scour at the higher discharges and then clear-water scour at the lower discharges on the falling stages. Clear-water scour reaches its maximum over a longer period of time than live-bed scour (Figure 3.1). This is because clear-water scour occurs mainly in coarse-bed material streams. In fact, local clear-water scour may not reach a maximum until after several floods. Maximum local clear-water pier scour is about 10 percent greater than the equilibrium local live-bed pier scour.

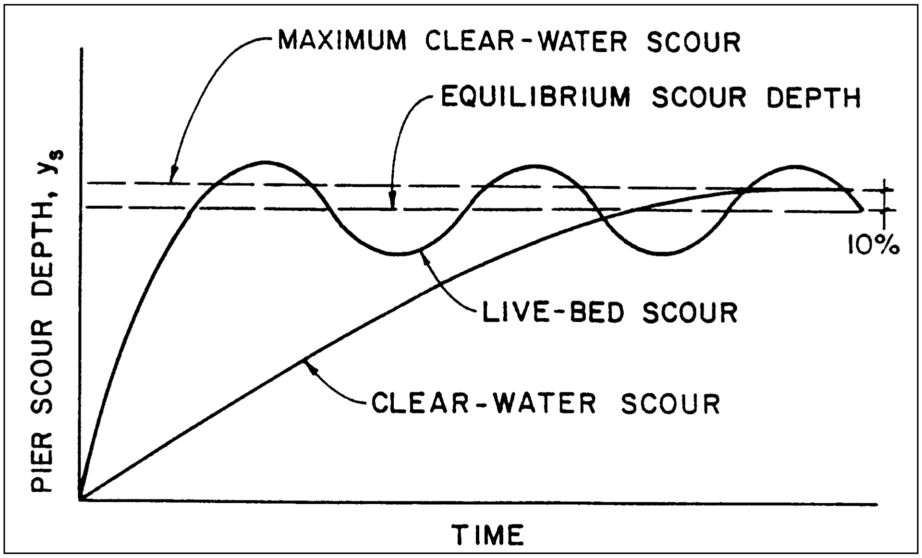

Figure 3.1. Pier scour depth in a sand-bed stream as a function of time.

Critical velocity equations with the reference particle size (D) equal to D_{50} are used to determine the velocity associated with the initiation of motion. They are used as an indicator for clear-water or live-bed scour conditions. If the mean velocity (V) in the upstream reach is equal to or less than the critical velocity (V_c) of the median diameter (D_{50}) of the bed material, then contraction and local scour will be clear-water scour. Also, if the ratio of the shear velocity of the flow to the fall velocity of the D_{50} of the bed material (V_*/ω) is greater than 2, contraction and local scour may be clear-water. If the mean velocity is greater than the critical velocity of the median bed material size, live-bed scour will occur. An equation to determine the critical velocity for a given flow depth and size of bed material is derived in Appendix C and given in Chapter 5.

This technique can be applied to any unvegetated channel or overbank area to determine whether scour is clear-water or live-bed. This procedure should be used with caution for assessing whether or not scour in the overbank will be clear-water or live-bed. For most cases, the presence of vegetation on the overbank will effectively bind and protect the overbank from erosive velocities. Also, in the overbank, generally the velocities are small and the bed material so fine that most overbank areas will experience clear-water scour.

Live-bed pier scour in sand-bed streams with a dune bed configuration fluctuates about the equilibrium scour depth (Figure 3.1). This is due to the variability of the bed material sediment transport in the approach flow when the bed configuration of the stream is dunes. In this case (dune bed configuration in the channel upstream and through the bridge), maximum depth of pier scour is about 30 percent larger than equilibrium depth of scour. However, with the exception of crossings over large rivers (i.e., the Mississippi, Columbia, etc.), the bed configuration in sand-bed streams will plane out during flood flows due to the increase in velocity and shear stress. For general practice, the maximum depth of pier scour is approximately 10 percent greater than equilibrium scour.

For a discussion of bedforms in alluvial channel flow, see Chapter 3 of HDS 6.[22] Equations for estimating local scour at piers or abutments are given in Chapters 6 and 7 of this document. These equations were developed from laboratory experiments and limited field data for both clear-water and live-bed scour.

3.5 GENERAL SCOUR

3.5.1 Contraction Scour

Contraction scour occurs when the flow area of a stream at flood stage is reduced, either by a natural contraction of the stream channel or by a bridge. It also occurs when overbank flow is forced back to the channel by roadway embankments at the approaches to a bridge. From continuity, a decrease in flow area results in an increase in average velocity and bed shear stress through the contraction. Hence, there is an increase in erosive forces in the contraction and more bed material is removed from the contracted reach than is transported into the reach. This increase in transport of bed material from the reach lowers the natural bed elevation. As the bed elevation is lowered, the flow area increases and, in the riverine situation, the velocity and shear stress decrease until relative equilibrium is reached; i.e., the quantity of bed material that is transported into the reach is equal to that removed from the reach, or the bed shear stress is decreased to a value such that no sediment is transported out of the reach. Contraction scour, in a natural channel or at a bridge crossing, involves removal of material from the bed across all or most of the channel width. Methods to estimate live-bed and clear-water contraction scour are presented in Chapter 5.

In coastal waterways which are affected by tides, as the cross-sectional area increases the discharge from the ocean may increase and thus the velocity and shear stress may not decrease. Consequently, relative equilibrium may not be reached. Thus, at tidal inlets contraction scour may result in a continual lowering of the bed (long-term degradation).

Live-bed contraction scour is typically cyclic; for example, the bed scours during the rising stage of a runoff event and fills on the falling stage. The cyclic nature of contraction scour causes difficulties in determining contraction scour depths after a flood. The contraction of flow at a bridge can be caused by either a natural decrease in flow area of the stream channel or by abutments projecting into the channel and/or piers blocking a portion of the flow area. Contraction can also be caused by the approaches to a bridge cutting off floodplain flow. This can cause clear-water scour on a setback portion of a bridge section or a relief bridge because the floodplain flow does not normally transport significant concentrations of bed material sediments. This clear-water picks up additional sediment from the bed upon reaching the bridge opening. In addition, local scour at abutments may well be greater due to the clear-water floodplain flow returning to the main channel at the end of the abutment.

Other factors that can cause contraction scour are (1) natural stream constrictions, (2) long highway approaches to the bridge over the floodplain, (3) ice formations or jams, (4) natural berms along the banks due to sediment deposits, (5) debris, (6) vegetative growth in the channel or floodplain, and (7) pressure flow.

3.5.2 Other General Scour

Other general scour conditions can result from erosion related to the planform characteristics of the stream (meandering, braided or straight), variable downstream control, flow around a bend, or other changes that decrease the bed elevation. General scour conditions can occur at bridges located upstream or downstream of a confluence. These scour conditions are discussed in Section 5.8 and HDS 6.[22]

3.6 LOCAL SCOUR

The basic mechanism causing local scour at piers or abutments is the formation of vortices (known as the horseshoe vortex) at their base (Figure 3.2). The horseshoe vortex results from the pileup of water on the upstream surface of the obstruction and subsequent acceleration of the flow around the nose of the pier or abutment. The action of the vortex removes bed material from around the base of the obstruction. The transport rate of sediment away from the base region is greater than the transport rate into the region, and, consequently, a scour hole develops. As the depth of scour increases, the strength of the horseshoe vortex is reduced, thereby reducing the transport rate from the base region. Eventually, for live-bed local scour, equilibrium is reestablished between bed material inflow and outflow and scouring ceases. For clear-water scour, scouring ceases when the shear stress caused by the horseshoe vortex equals the critical shear stress of the sediment particles at the bottom of the scour hole.

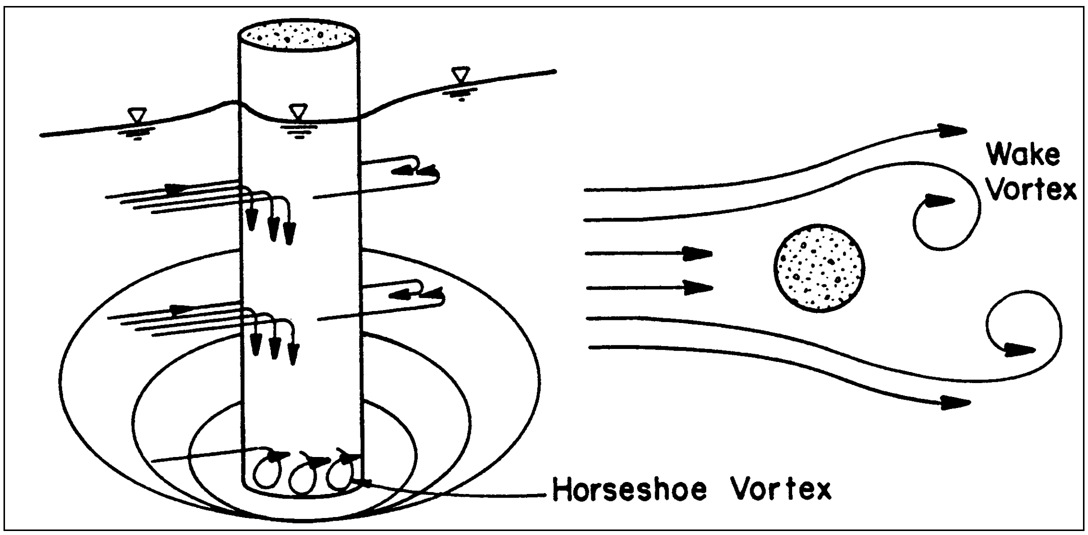

Figure 3.2. Schematic representation of scour at a cylindrical pier.

In addition to the horseshoe vortex around the base of a pier, there are vertical vortices downstream of the pier called the wake vortex (Figure 3.2). Both the horseshoe and wake vortices remove material from the pier base region. However, the intensity of wake vortices diminishes rapidly as the distance downstream of the pier increases. Therefore, immediately downstream of a long pier there is often deposition of material.

Factors which affect the magnitude of local scour depth at piers and abutments are (1) velocity of the approach flow, (2) depth of flow, (3) width of the pier, (4) discharge intercepted by the abutment and returned to the main channel at the abutment (in laboratory flumes this discharge is a function of projected length of an abutment into the flow), (5) length of the pier if skewed to flow, (6) size and gradation of bed material, (7) angle of attack of the approach flow to a pier or abutment, (8) shape of a pier or abutment, (9) bed configuration, and (10) ice formation or jams and debris.

1. Flow velocity affects local scour depth. The greater the velocity, the deeper the scour. There is a high probability that scour is affected by whether the flow is subcritical or supercritical. However, most research and data are for subcritical flow (i.e., flow with a Froude Number less than 1.0, $Fr < 1$).

2. Flow depth also has an influence on the depth of local scour. An increase in flow depth can increase scour depth by a factor of 2 or greater for piers. With abutments, the increase is approximately 1.1 to 2.15 depending on the shape of the abutment.

3. Pier width has a direct influence on depth of local scour. As pier width increases, there is an increase in scour depth. There is a limit to the increase in scour depth as width increases. Very wide piers (see Section 6.3) do not have scour depths as deep as predicted by existing equations.

4. In laboratory flume studies, an increase in the projected length of an abutment (or embankment) into the flow increased scour; whereas, this is not the case in the field. Due to the relatively small scale of a laboratory flume, floodplain flow intercepted by the embankment and returned to the main channel is directly related to the length of the obstruction. However, in the field case the embankment length is not a good measure of the discharge returned to the main channel. This results in "ineffective flow" on the floodplain which can be even more pronounced on wide heavily vegetated floodplains. In order to properly apply laboratory derived abutment scour equations to the field case, an assessment must be made of the location of the boundary between "live flow" and "ineffective flow." The location of this boundary should then be used to establish the length of the abutment or embankment for abutment scour computations (see Section 7.2).

5. Pier length has no appreciable effect on local scour depth as long as the pier is aligned with the flow. When the pier is skewed to the flow, the pier length has a significant influence on scour depth. For example, doubling the length of the pier increases scour depth from 30 to 60 percent (depending on the angle of attack).

6. Bed material characteristics such as size, gradation, and cohesion can affect local scour. Bed material in the sand-size range has little effect on local scour depth. Likewise, larger size bed material that can be moved by the flow or by the vortices and turbulence created by the pier or abutment will not affect the maximum scour, but only the time it takes to attain it. Very large particles in the bed material, such as coarse gravels, cobbles or boulders, may armor the scour hole. Research at the University of Aukland, New Zealand, by the Washington State DOT, and by other researchers developed equations that take into account the decrease in scour due to the armoring of the scour hole.[25, 26, 27, 28] Richardson and Richardson combined these equations into a simplified equation, which accounted for bed material size.[29] However, field data are inadequate to support these equations at this time.

   Molinas in flume experiments sponsored by FHWA, showed for Froude Numbers less than 1.0 ($Fr < 1.0$), and a range of bed material sizes, that when the approach velocity (V_1) of the flow is less than the critical velocity (V_c) of the D_{90} size of the bed material, the D_{90} size will decrease the scour depth.[30]

   The size of the bed material also determines whether the scour at a pier or abutment is clear-water or live-bed scour. This topic is discussed in Section 3.4.

Fine bed material (silts and clays) will have scour depths as deep as sand-bed streams. This is true even if bonded together by cohesion. The effect of cohesion is to influence the time it takes to reach maximum scour. With sand-bed material the time to reach maximum depth of scour is measured in hours and can result from a single flood event. With cohesive bed materials it may take much longer to reach the maximum scour depth, the result of many flood events. Scour in cohesive bed material is discussed in Section 12.9 and Appendix L

7. Angle of attack of the flow to the pier or abutment has a significant effect on local scour, as was pointed out in the discussion of pier length. Abutment scour is reduced when embankments are angled downstream and increased when embankments are angled upstream. According to the work of Ahmad, the maximum depth of scour at an embankment inclined 45 degrees downstream is reduced by 20 percent; whereas, the maximum scour at an embankment inclined 45 degrees upstream is increased about 10 percent.[31]

8. Shape of the nose of a pier or an abutment can have up to a 20 percent influence on scour depth. Streamlining the front end of a pier reduces the strength of the horseshoe vortex, thereby reducing scour depth. Streamlining the downstream end of piers reduces the strength of the wake vortices. A square-nose pier will have maximum scour depths about 20 percent greater than a sharp-nose pier and 10 percent greater than either a cylindrical or round-nose pier. The shape effect is negligible for flow angles in excess of five degrees. Full retaining abutments with vertical walls on the stream side (parallel to the flow) and vertical walls parallel to the roadway will produce scour depths about double that of spill-through (sloping) abutments.

9. Bed configuration of sand-bed channels affects the magnitude of local scour. In streams with sand-bed material, the shape of the bed (bed configuration) as described by Richardson et al. may be ripples, dunes, plane bed, or antidunes.[32] The bed configuration depends on the size distribution of the sand-bed material, hydraulic characteristics, and fluid viscosity. The bed configuration may change from dunes to plane bed or antidunes during an increase in flow for a single flood event. It may change back with a decrease in flow. The bed configuration may also change with a change in water temperature or suspended sediment concentration of silts and clays. The type of bed configuration and change in bed configuration will affect flow velocity, sediment transport, and scour. HDS 6 discusses bed configuration in detail.[22]

10. Potentially, ice and debris can increase the width of the piers, change the shape of piers and abutments, increase the projected length of an abutment, and cause the flow to plunge downward against the bed. This can increase both local and contraction scour. The magnitude of the increase is still largely undetermined. Debris can be taken into account in the scour equations by estimating how much the debris will increase the width of a pier or length of an abutment. Debris and ice effects on contraction scour can also be accounted for by estimating the amount of flow blockage (decrease in width of the bridge opening) in the equations for contraction scour. Limited field measurements of scour at ice jams indicate the scour can be as much as 3 to 10 m (10 to 30 ft).

3.7 LATERAL SHIFTING OF A STREAM

Streams are dynamic. Areas of flow concentration continually shift banklines, and in meandering streams having an "S-shaped" planform, the channel moves both laterally and downstream. A braided stream has numerous channels which are continually changing. In a braided stream, the deepest natural scour occurs when two channels come together or when the flow comes together downstream of an island or bar. This scour depth has been observed to be 1 to 2 times the average flow depth.

A bridge is static. It fixes the stream at one place in time and space. A meandering stream whose channel moves laterally and downstream into the bridge reach can erode the approach embankment and can affect contraction and local scour because of changes in flow direction. A braided stream can shift under a bridge and have two channels come together at a pier or abutment, increasing scour. Descriptions of stream morphology are given in HDS 6 and HEC-20.(22, 6)

Factors that affect lateral shifting of a stream and the stability of a bridge are the geomorphology of the stream, location of the crossing on the stream, flood characteristics, the characteristics of the bed and bank material, and wash load. It is difficult to anticipate when a change in planform may occur. It may be gradual or the result of a single major flood event. Also, the direction and magnitude of the movement of the stream are not easily predicted. While it is difficult to evaluate the vulnerability of a bridge due to changes in planform, it is important to incorporate potential planform changes into the design of new bridges and design of countermeasures for existing bridges. These factors are discussed and analysis techniques are presented in HEC-20.(6)

Countermeasures for lateral shifting and instability of the stream may include changes in the bridge design, construction of river control works, protection of abutments with riprap, or careful monitoring of the river in a bridge inspection program. **Serious consideration should be given to placing footings/foundations located on floodplains at elevations the same as those located in the main channel**. Control of lateral shifting requires river training works, bank stabilizing by riprap, and/or guide banks. The design of these works is beyond the scope of this circular. Design methods are given by FHWA in HEC-23,(7) HDS 6,(22) HEC-11,(33) and similar publications.(34,35) The USACE and AASHTO provide additional guidance.(36,37,38,39)

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CHAPTER 4

LONG-TERM AGGRADATION AND DEGRADATION

4.1 INTRODUCTION

This chapter discusses the factors affecting long-term bed elevation changes, methods available for estimating these changes, and the role of sediment transport computer models that are available to compliment HEC-20 procedures. This chapter links long-term degradation to the other components of scour at a bridge site. In following chapters methods and equations are given for determining the other components of total scour. Procedures for estimating long-term aggradation and degradation at a bridge are presented in HEC-20.[6]

4.2 LONG-TERM BED ELEVATION CHANGES

Long-term bed elevation changes may be the natural trend of the stream or may be the result of some modification to the stream or watershed. The streambed may be aggrading, degrading, or in relative equilibrium in the vicinity of the bridge crossing. In this section, long-term trends are considered. Long-term aggradation and degradation do not include the cutting and filling of the streambed at a bridge that might occur during a runoff event (general and local scour). A stream may cut and fill at specific locations during a runoff event and also have a long-term trend of an increase or decrease in bed elevation over a longer reach of a stream. The problem for the engineer is to estimate the long-term bed elevation changes that will occur during the life of the structure.

A long-term trend may change during the life of the bridge. These long-term changes are the result of modifications to the stream or watershed. Such changes may be the result of natural processes or human activities. The engineer must assess the present state of the stream and watershed and then evaluate potential future changes in the river system. From this assessment, the long-term streambed changes must be estimated.

Factors that affect long-term bed elevation changes are dams and reservoirs (up- or downstream of the bridge), changes in watershed land use (urbanization, deforestation, etc.), channelization, cutoffs of meander bends (natural or man-made), changes in the downstream channel base level (control), gravel mining from the streambed, diversion of water into or out of the stream, natural lowering of the fluvial system, movement of a bend and bridge location with respect to stream planform, and stream movement in relation to the crossing. Tidal ebb and flood may degrade a coastal stream; whereas, littoral drift may result in aggradation. The elevation of the bed under bridges which cross streams tributary to a larger stream will follow the trend of the larger stream unless there are controls. Controls could be bed rock, dams, culverts or other structures. The changes in bed elevation decrease the further upstream the bridge is from the confluence with another stream or from other bed elevation controls.

The USACE, USGS, and other Federal and State agencies should be contacted concerning documented long-term streambed variations. If no data exist or if such data require further evaluation, an assessment of long-term streambed elevation changes for riverine streams should be made using the principles of river mechanics. Such an assessment requires the consideration of all influences upon the bridge crossing, i.e., runoff from the watershed to a stream (hydrology), sediment delivery to the channel (watershed erosion), sediment transport capacity of a stream (hydraulics), and response of a stream to these factors (geomorphology and river mechanics).

With coastal streams, the principles of both river and coastal engineering mechanics are needed. In coastal streams, estuaries or inlets, in addition to the above, consideration must be given to tidal conditions, i.e., the magnitude and period of the storm surge, sediment delivery to the channel by the ebb and flow of the tide, littoral drift, sediment transport capacity of the tidal flows, and response of the stream, estuary, or inlet to these tidal and coastal engineering factors.

Significant morphologic impacts can result from human activities. The assessment of the impact of human activities requires a study of the history of the river, estuary, or tidal inlet, as well as a study of present water and land use and stream control activities. All agencies involved with the river or coastal area should be contacted to determine possible future changes.

4.3 ESTIMATING LONG-TERM AGGRADATION AND DEGRADATION

To organize an assessment of long-term aggradation and degradation, a three-level fluvial system approach can be used. The three level approach consists of (1) a qualitative determination based on general geomorphic and river mechanics relationships, (2) an engineering geomorphic analysis using established qualitative and quantitative relationships to estimate the probable behavior of the stream system to various scenarios or future conditions, and (3) physical models or physical process computer modeling using mathematical models such as BRI-STARS[21] and the USACE HEC-6[40] to make predictions of quantitative changes in streambed elevation due to changes in the stream and watershed. Methods to be used in Levels (1) and (2) are presented in HEC-20 and HDS 6.[6, 22]

For coastal areas, where highway crossings (bridges) and/or longitudinal stream encroachments are subject to tidal influences, the three-level approach used in fluvial systems is also appropriate (Chapter 9). The following sections outline procedures that can assist in identifying long-term trends in vertical stability.

4.3.1 Bridge Inspection Records

The biannual bridge inspection reports for bridges on the stream where a new or replacement bridge is being designed are an excellent source of data on long-term aggradation or degradation trends. Also, inspection reports for bridges crossing streams in the same area or region should be studied. In most states the biannual inspection includes taking the elevation and/or cross section of the streambed under the bridge. These elevations are usually referenced to the bridge, but these relative bed elevations will show trends and can be referenced to sea level elevations. Successive cross sections from a series of bridges in a stream reach can be used to construct longitudinal streambed profiles through the reach.

4.3.2 Gaging Station Records

The USGS and many State Water Resource and Environmental agencies maintain gaging stations to measure stream flow. In the process they maintain records from which the aggradation or degradation of the streambed can be determined. Gaging station records at the bridge site, on the stream to be bridged and in the area or region can be used.

Where an extended historical record is available, one approach to using gaging station records to determine long-term bed elevation change is to plot the change in stage through time for a selected discharge. This approach is often referred to as establishing a "specific gage" record.

Figure 4.1 shows a plot of specific gage data for a discharge of 14 m^3/sec (500 cfs) from about 1910 to 1980 for Cache Creek in California. Cache Creek has experienced significant gravel mining with records of gravel extraction quantities available since about 1940. When the historical record of cumulative gravel mining is compared to the specific gage plot, the potential impacts are apparent. The specific gage record shows more than 3 m (10 ft) of long-term degradation in a 70-year period.

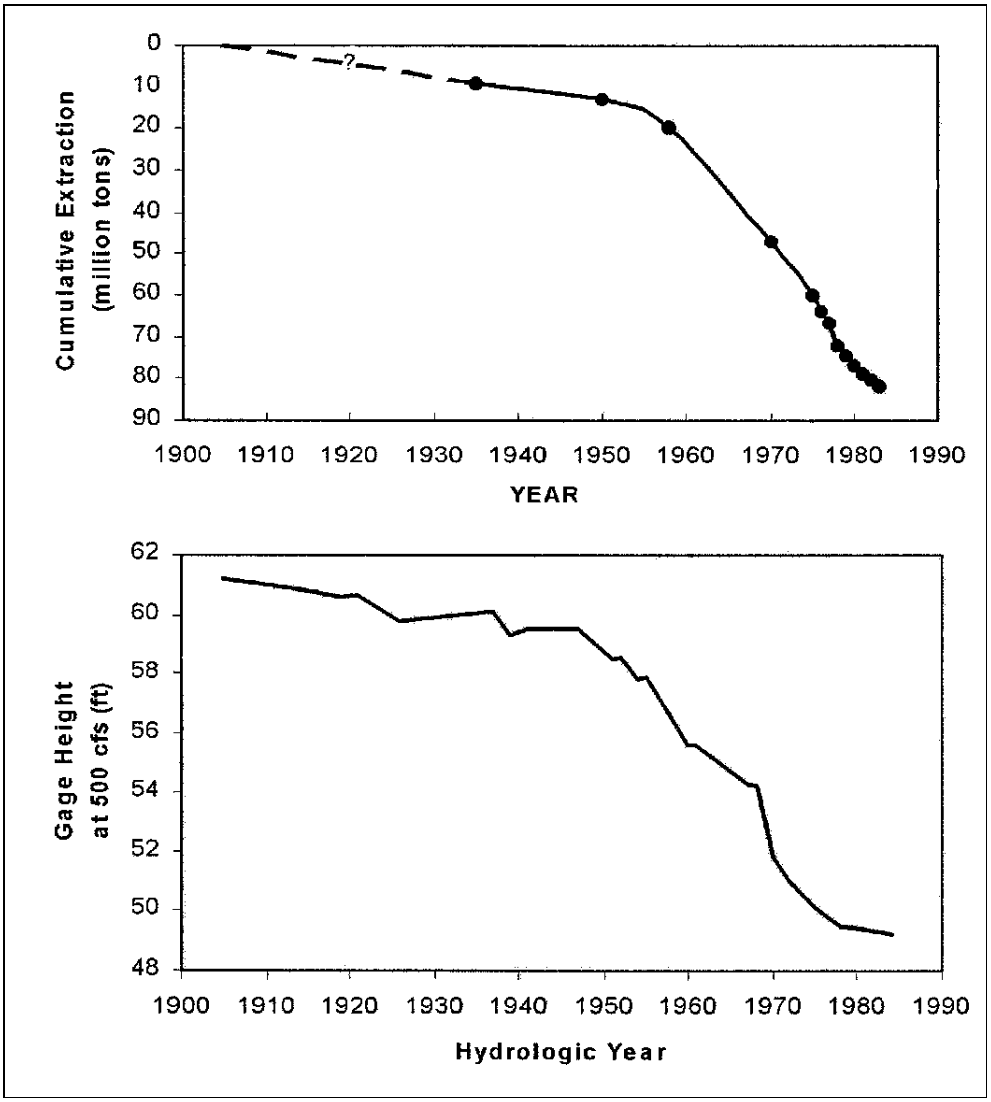

Figure 4.1. Specific gage data for Cache Creek, California.

4.3.3 Geology and Stream Geomorphology

The geology and geomorphology of the site needs to be studied to determine the potential for long-term bed elevation changes at the bridge site. Quantitative techniques for streambed aggradation and degradation analyses are covered in detail in HEC-20.(6) These techniques include:

- Incipient motion analysis
- Analysis of armoring potential
- Equilibrium slope analysis
- Sediment continuity analysis

Sediment transport concepts and equations are discussed in detail in HDS 6.(22)

4.3.4 Computer Models

Sediment transport computer models can be used to determine long-term aggradation or degradation trends. These computer models route sediment down a channel and adjust the channel geometry to reflect imbalances in sediment supply and transport capacity. The BRI-STARS(21) and HEC-6(40) models are examples of sediment transport models that can be used for single event or long-term estimates of changes in bed elevation. The information needed to run these models includes:

- Channel and floodplain geometry
- Structure geometry
- Roughness
- Geologic or structural vertical controls
- Downstream water surface relationship
- Event or long-term inflow hydrographs
- Tributary inflow hydrographs
- Bed material gradations
- Upstream sediment supply
- Tributary sediment supply
- Selection of appropriate sediment transport relationship
- Depth of alluvium

These models perform hydraulic and sediment transport computations on a cross section basis and adjust the channel geometry prior to proceeding with the next time step. The actual flow hydrograph can be used as input. BRI-STARS(21) also has an option where width adjustment can be predicted.

4.3.5 Aggradation, Degradation, and Total Scour

Using all the information available estimate the long-term bed elevation change at the bridge site for the design life of the bridge. Usually, the design life is 100 years. **If the estimate indicates that the stream will degrade, use the elevation after degradation as the base elevation for general and local scour. That is, total scour must include the estimated long-term degradation**. If the estimate indicates that the stream will aggrade, then (1) make note of this fact to inspection and maintenance personnel, and (2) use existing ground elevation as the base for general and local scour.

4.3.6 Inspection, Maintenance, and Countermeasures

The estimate of long-term aggradation or degradation in the final design should be communicated to inspection and maintenance personnel. This information will aid them in tracking long-term trends and provide feedback for future design and evaluation. HEC-23[7] outlines techniques for controlling long-term bed elevation changes and provides design guidance for countermeasures commonly used for vertical stability problems.

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CHAPTER 5

GENERAL SCOUR

5.1 INTRODUCTION

General scour is the general decrease in the elevation of the bed across the bridge opening. It does not include localized scour at the foundations (local scour) or the long-term changes in the stream bed elevation (aggradation or degradation). General scour may not have a uniform depth across the bridge opening. General scour can be cyclic, that is, there can be an increase and decrease of the stream bed elevation (cutting and filling) during the passage of a flood.

The most common general scour is contraction scour. There are several cases and flow conditions for contraction scour. Typically, contraction scour occurs where the bridge opening is smaller than the flow area of the upstream channel and/or floodplain. Other general scour conditions can result from erosion related to planform characteristics of the stream, flow around a bend, variable downstream control, or other changes that decrease the bed elevation at the bridge. In this chapter, methods and equations will be presented to estimate general scour.

5.2 CONTRACTION SCOUR

5.2.1 Contraction Scour Conditions

Contraction scour equations are based on the principle of conservation of sediment transport (continuity). In the case of **live-bed scour**, the fully developed scour in the bridge cross section reaches equilibrium when sediment transported into the contracted section equals sediment transported out. As scour develops, the shear stress in the contracted section decreases as a result of a larger flow area and decreasing average velocity. For **live-bed** scour, maximum scour occurs when the shear stress reduces to the point that sediment transported in equals the bed sediment transported out and the conditions for sediment continuity are in balance. For **clear-water** scour, the transport into the contracted section is essentially zero and maximum scour occurs when the shear stress reduces to the critical shear stress of the bed material in the section. Normally, for both live-bed and clear-water scour the width of the contracted section is constrained and depth increases until the limiting conditions are reached.

Live-bed contraction scour occurs at a bridge when there is transport of bed material in the upstream reach into the bridge cross section. With live-bed contraction scour the area of the contracted section increases until, in the limit, the transport of sediment out of the contracted section equals the sediment transported in.

Clear-water contraction scour occurs when (1) there is no bed material transport from the upstream reach into the downstream reach, or (2) the material being transported in the upstream reach is transported through the downstream reach mostly in suspension and at less than capacity of the flow. With clear-water contraction scour the area of the contracted section increases until, in the limit, the velocity of the flow (V) or the shear stress (τ_o) on the bed is equal to the critical velocity (V_c) or the critical shear stress (τ_c) of a certain particle size (D) in the bed material.

There are four conditions (cases) of contraction scour at bridge sites depending on the type of contraction, and whether there is overbank flow or relief bridges. Regardless of the case, contraction scour can be evaluated using two basic equations: (1) **live-bed** scour, and (2) **clear-water** scour. For any case or condition, it is only necessary to determine if the flow in the main channel or overbank area upstream of the bridge, or approaching a relief bridge, is transporting bed material (live-bed) or is not (clear-water), and then apply the appropriate equation with the variables defined according to the location of contraction scour (channel or overbank).

To determine if the flow upstream of the bridge is transporting bed material, calculate the critical velocity for beginning of motion V_c of the D_{50} size of the bed material being considered for movement and compare it with the mean velocity V of the flow in the main channel or overbank area upstream of the bridge opening. If the critical velocity of the bed material is larger than the mean velocity ($V_c > V$), then clear-water contraction scour will exist. If the critical velocity is less than the mean velocity ($V_c < V$), then live-bed contraction scour will exist. To calculate the critical velocity use the equation derived in the Appendix C. This equation is:

$$V_c = K_u \, y^{1/6} \, D^{1/3} \qquad (5.1)$$

where:

V_c	=	Critical velocity above which bed material of size D and smaller will be transported, m/s (ft/s)
y	=	Average depth of flow upstream of the bridge, m (ft)
D	=	Particle size for V_c, m (ft)
D_{50}	=	Particle size in a mixture of which 50 percent are smaller, m (ft)
K_u	=	6.19 SI units
K_u	=	11.17 English units

The D_{50} is taken as an average of the bed material size in the reach of the stream upstream of the bridge. It is a characteristic size of the material that will be transported by the stream. Normally this would be the bed material size in the upper 0.3 m (1 ft) of the stream bed.

Live-bed contraction scour depths may be limited by armoring of the bed by large sediment particles in the bed material or by sediment transport of the bed material into the bridge cross-section. Under these conditions, live-bed contraction scour at a bridge can be determined by calculating the scour depths using both the clear-water and live-bed contraction scour equations and using the smaller of the two depths.

5.2.2 Contraction Scour Cases

Four conditions (cases) of contraction scour are commonly encountered:

Case 1. Involves overbank flow on a floodplain being forced back to the main channel by the approaches to the bridge. Case 1 conditions include:

a. The river channel width becomes narrower either due to the bridge abutments projecting into the channel or the bridge being located at a narrowing reach of the river (Figure 5.1);

b. No contraction of the main channel, but the overbank flow area is completely obstructed by an embankment (Figure 5.2); or

c. Abutments are set back from the stream channel (Figure 5.3).

Case 2. Flow is confined to the main channel (i.e., there is no overbank flow). The normal river channel width becomes narrower due to the bridge itself or the bridge site is located at a narrower reach of the river (Figures 5.4 and 5.5).

Case 3. A relief bridge in the overbank area with little or no bed material transport in the overbank area (i.e., clear-water scour) (Figure 5.6).

Case 4. A relief bridge over a secondary stream in the overbank area with bed material transport (similar to Case 1) (Figure 5.7).

Notes:

1. **Cases 1, 2, and 4** may either be live-bed or clear-water scour depending on whether there is bed material transport from the upstream reach into the bridge reach during flood flows. To determine if there is bed material transport compute the critical velocity at the approach section for the D_{50} of the bed material using the equation given above and compare to the mean velocity at the approach section. To determine if the bed material will be washed through the contraction determine the ratio of the shear velocity (V_*) in the contracted section to the fall velocity (ω) of the D_{50} of the bed material being transported from the upstream reach (see the definition of V_* in the live-bed contraction scour equation). If the ratio is much larger than 2, then the bed material from the upstream reach will be mostly suspended bed material discharge and may wash through the contracted reach (clear-water scour).

2. **Case 1c is very complex.** The depth of contraction scour depends on factors such as (1) how far back from the bank line the abutment is set, (2) the condition of the overbank (is it easily eroded, are there trees on the bank, is it a high bank, etc.), (3) whether the stream is narrower or wider at the bridge than at the upstream section, (4) the magnitude of the overbank flow that is returned to the bridge opening, and (5) the distribution of the flow in the bridge section, and (6) other factors.

   The main channel under the bridge may be live-bed scour; whereas, the set-back overbank area may be clear-water scour.

   WSPRO[15] or HEC-RAS[16,17] can be used to determine the distribution of flow between the main channel and the set-back overbank areas in the contracted bridge opening. However, the distribution of flow needs to be done with care. Studies by Chang[41] and Sturm [42] have shown that conveyance calculations do not properly account for the flow distribution under the bridge.

   If the abutment is set back only a small distance from the bank (less than 3 to 5 times the average depth of flow through the bridge), there is the possibility that the combination of contraction scour and abutment scour may destroy the bank. Also, the two scour mechanisms are not independent. Consideration should be given to using a guide bank and/or protecting the bank and bed under the bridge in the overflow area with rock riprap. See HEC-23[7] for guidance on designing rock riprap.

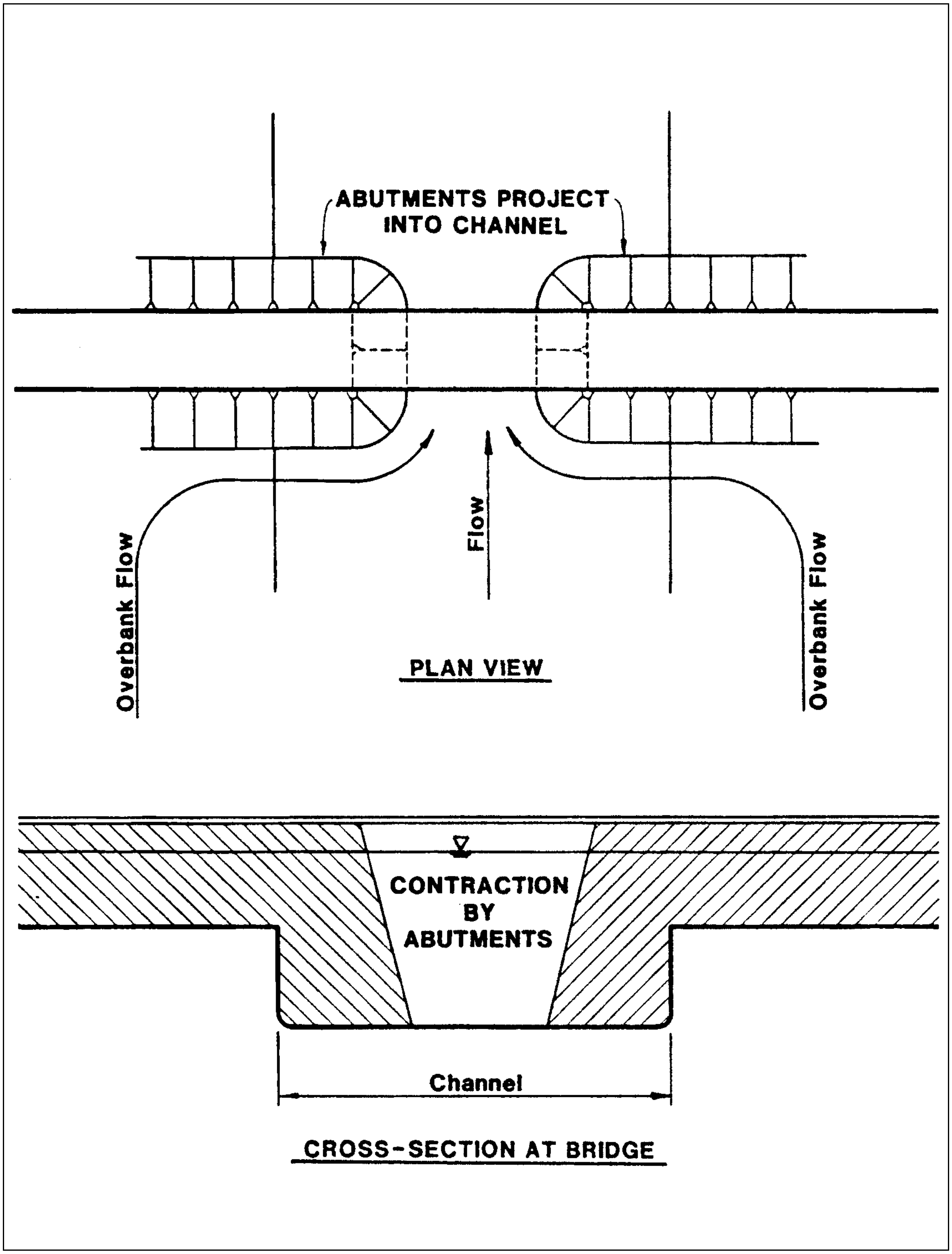


Figure 5.1. Case 1A: Abutments project into channel.

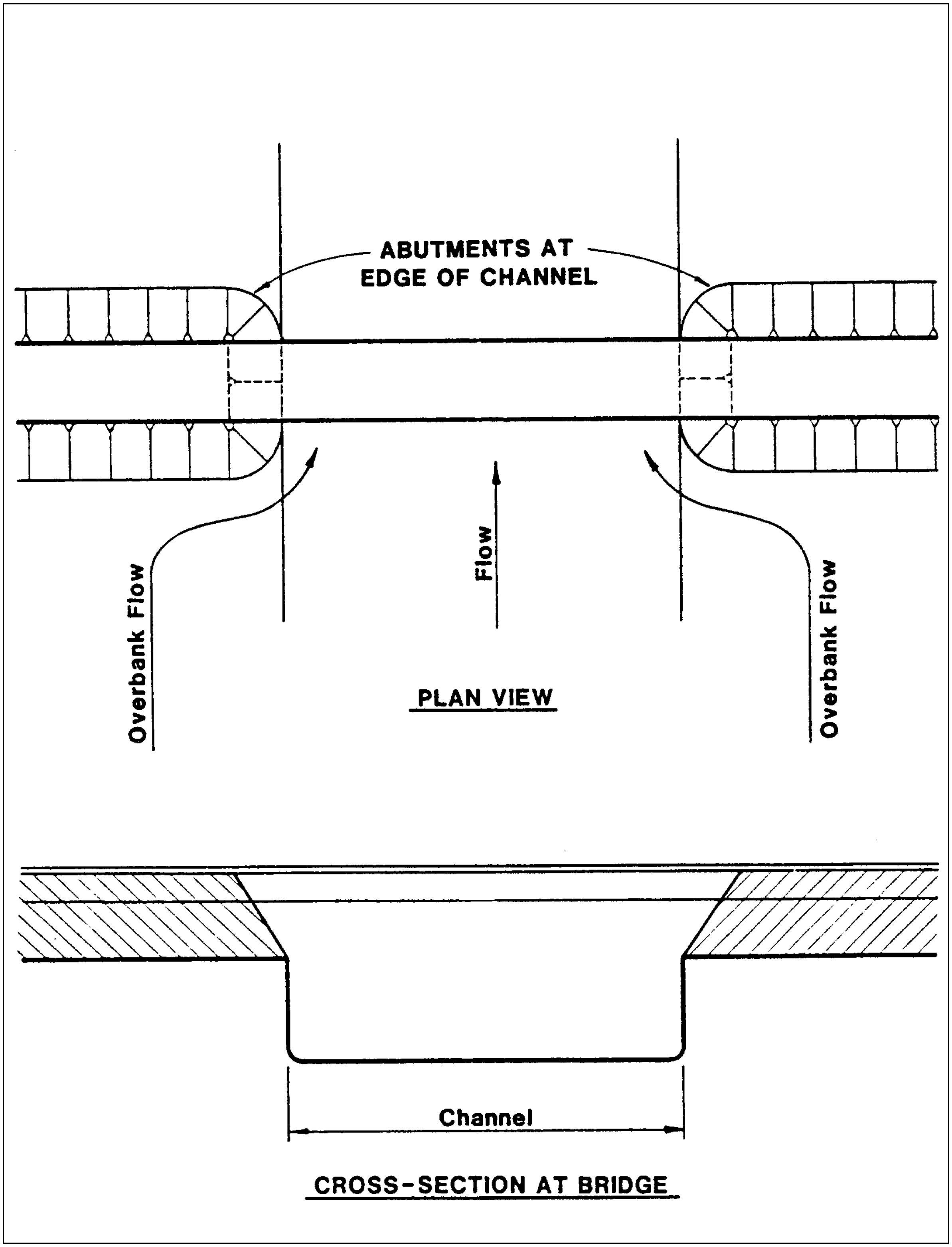


Figure 5.2. Case 1B: Abutments at edge of channel.

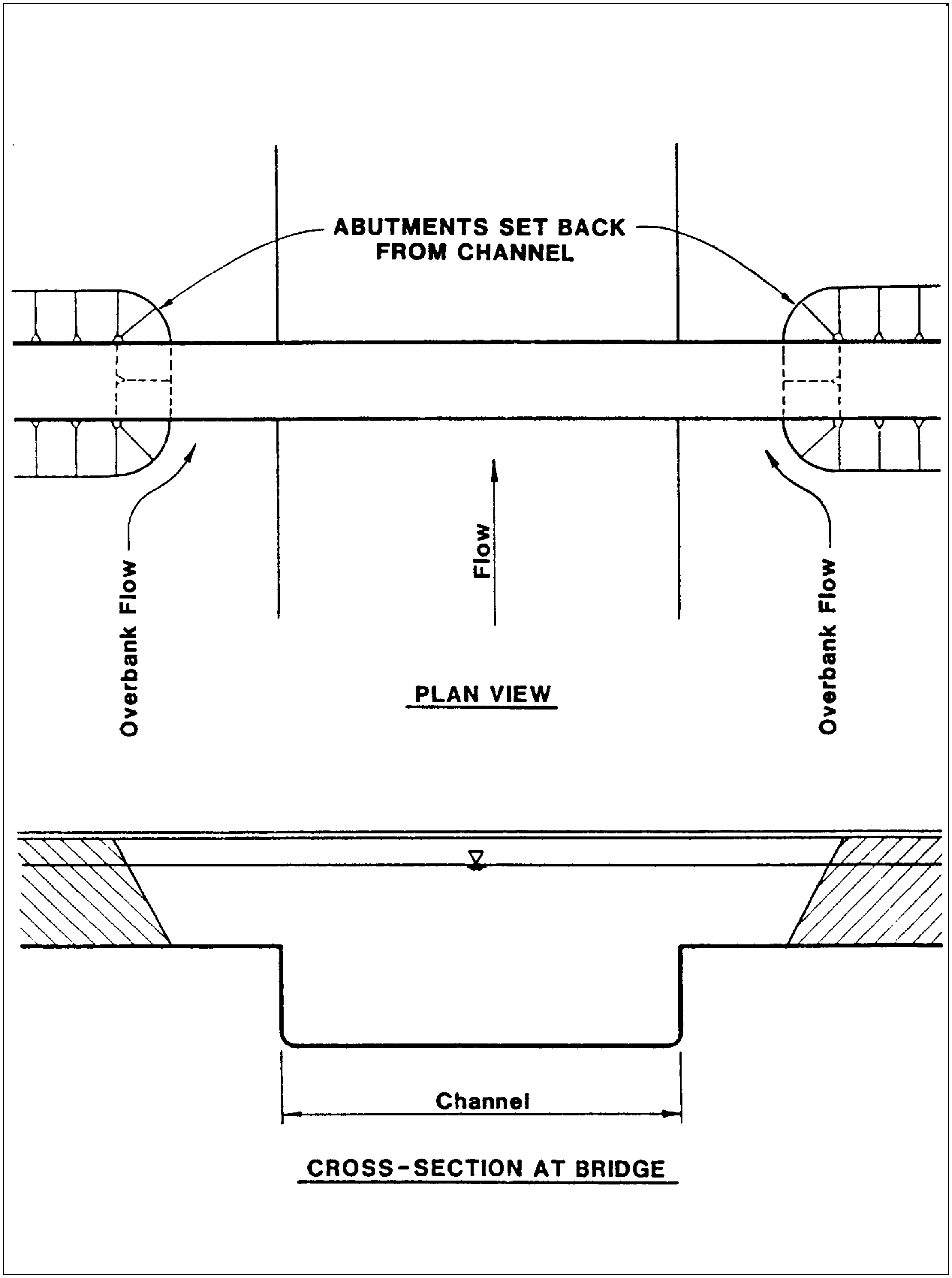


Figure 5.3. Case 1C: Abutments set back from channel.

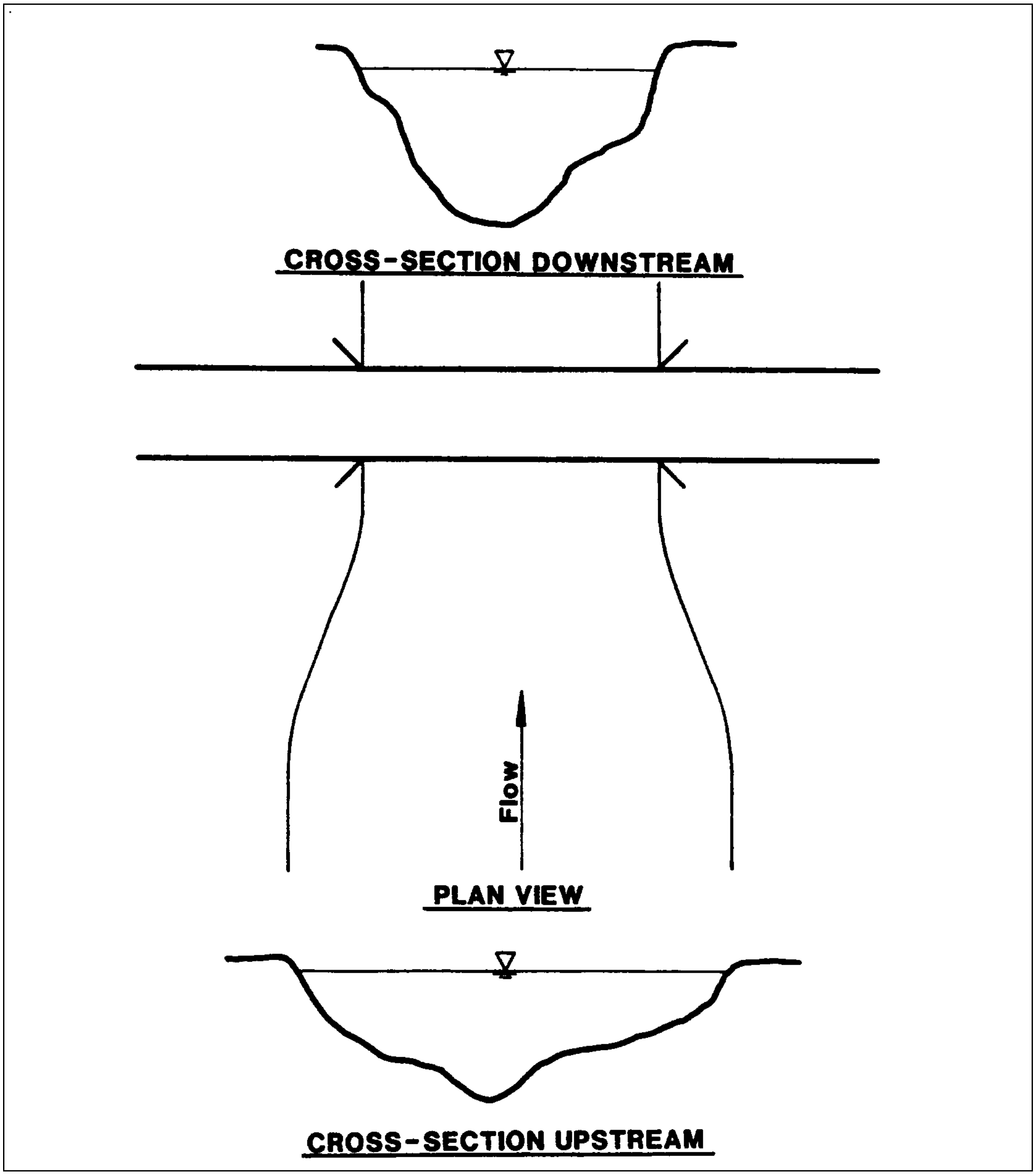


Figure 5.4. Case 2A: River narrows.

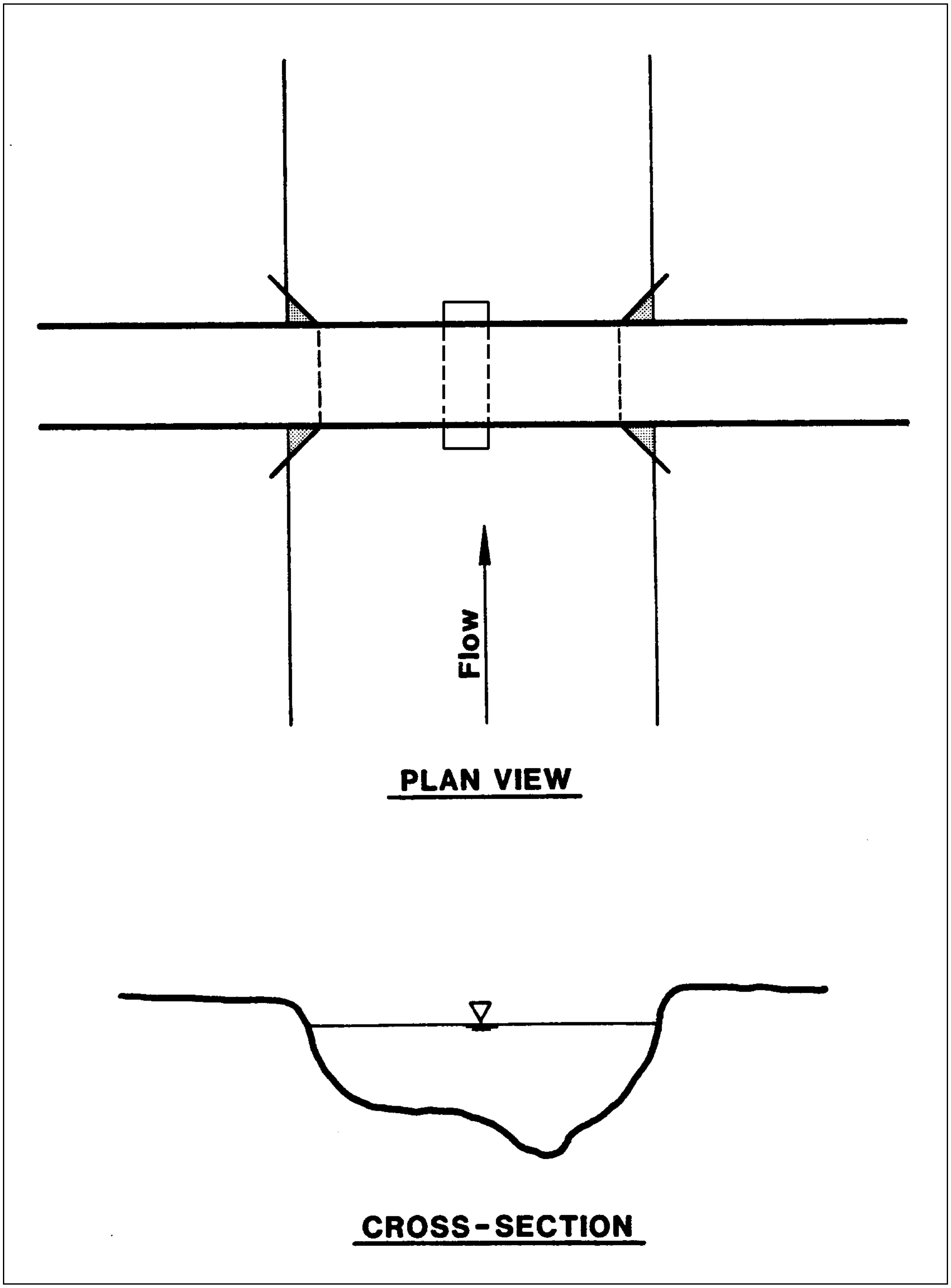


Figure 5.5. Case 2B: Bridge abutments and/or piers constrict flow.

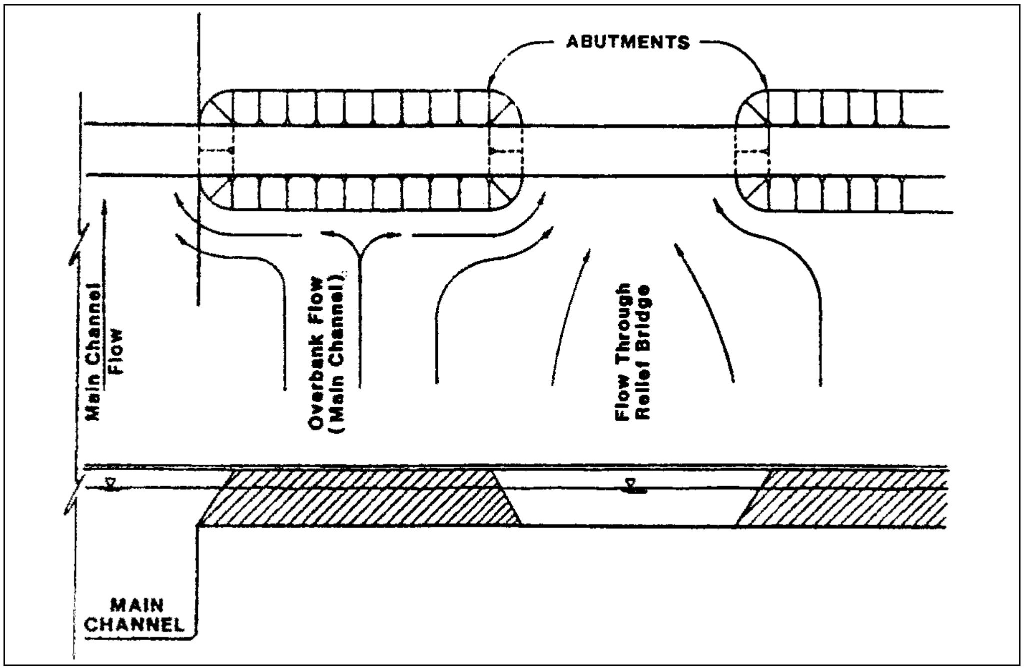


Figure 5.6. Case 3: Relief bridge over floodplain.

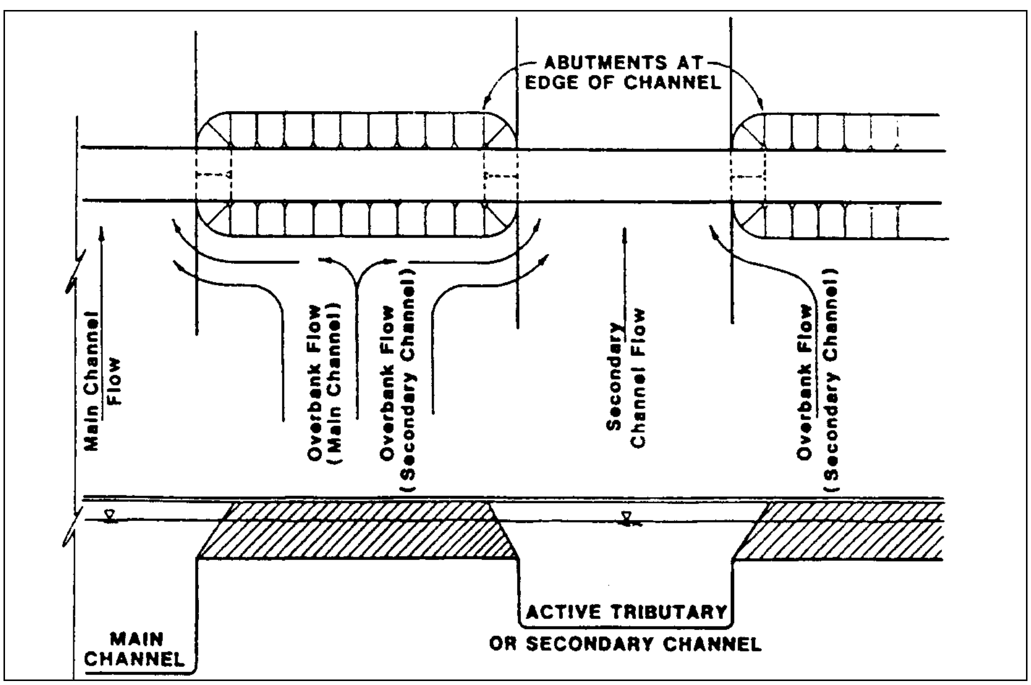


Figure 5.7. Case 4: Relief bridge over secondary stream.

3. **Case 3** may be clear-water scour even though the floodplain bed material is composed of sediments with a critical velocity that is less than the flow velocity in the overbank area. The reasons for this are (1) there may be vegetation growing part of the year, and (2) if the bed material is fine sediments, the bed material discharge may go into suspension (wash load) at the bridge and not influence contraction scour.

4. **Case 4** is similar to Case 3, but there is sediment transport into the relief bridge opening (live-bed scour). This case can occur when a relief bridge is over a secondary channel on the floodplain. Hydraulically this is no different from case 1, but analysis is required to determine the floodplain discharge associated with the relief opening and the flow distribution going to and through the relief bridge. This information could be obtained from WSPRO[15] or HEC-RAS.[16, 17]

5.3 LIVE-BED CONTRACTION SCOUR

A modified version of Laursen's 1960 equation for live-bed scour at a long contraction is recommended to predict the depth of scour in a contracted section.[43] The original equation is given in Appendix C. The modification is to eliminate the ratio of Manning's n (see the following Note #3). The equation assumes that bed material is being transported from the upstream section.

$$\frac{y_2}{y_1} = \left(\frac{Q_2}{Q_1}\right)^{6/7} \left(\frac{W_1}{W_2}\right)^{k_1} \tag{5.2}$$

$$y_s = y_2 - y_o = \text{(average contraction scour depth)} \tag{5.3}$$

where:

y_1 = Average depth in the upstream main channel, m (ft)
y_2 = Average depth in the contracted section, m (ft)
y_o = Existing depth in the contracted section before scour, m (ft) (see Note 7)
Q_1 = Flow in the upstream channel transporting sediment, m^3/s (ft^3/s)
Q_2 = Flow in the contracted channel, m^3/s (ft^3/s)

W_1 = Bottom width of the upstream main channel that is transporting bed material, m (ft)
W_2 = Bottom width of the main channel in the contracted section less pier width(s), m (ft)
k_1 = Exponent determined below

V_*/ω	k_1	Mode of Bed Material Transport
<0.50	0.59	Mostly contact bed material discharge
0.50 to 2.0	0.64	Some suspended bed material discharge
>2.0	0.69	Mostly suspended bed material discharge

V_* = $(\tau_o/\rho)^{1/2} = (g y_1 S_1)^{1/2}$, shear velocity in the upstream section, m/s (ft/s)
ω = Fall velocity of bed material based on the D_{50}, m/s (Figure 5.8)
For fall velocity in English units (ft/s) multiply ω in m/s by 3.28
g = Acceleration of gravity (9.81 m/s^2) (32.2 ft/s^2)
S_1 = Slope of energy grade line of main channel, m/m (ft/ft)

τ_o = Shear stress on the bed, Pa (N/m^2) (lb/ft^2)
ρ = Density of water (1000 kg/m^3) (1.94 slugs/ft^3)

Notes:

1. Q_2 may be the total flow going through the bridge opening as in cases 1a and 1b. **It is not the total flow for Case 1c**. For Case 1c contraction scour must be computed separately for the main channel and the left and/or right overbank areas.

2. Q_1 is the flow in the main channel upstream of the bridge, not including overbank flows.

3. The Manning's n ratio is eliminated in Laursen live-bed equation to obtain Equation 5.2 (Appendix C).This was done for the following reasons. The ratio can be significant for a condition of dune bed in the upstream channel and a corresponding plane bed, washed out dunes or antidunes in the contracted channel. However, Laursen's equation does not correctly account for the increase in transport that will occur as the result of the bed planning out (which decreases resistance to flow, increases the velocity and the transport of bed material at the bridge). That is, Laursen's equation indicates a decrease in scour for this case, whereas in reality, there would be an increase in scour depth. In addition, at flood flows, a plane bedform will usually exist upstream and through the bridge waterway, and the values of Manning's n will be equal. Consequently, the n value ratio is not recommended or presented in Equation 5.2.

4. W_1 and W_2 are not always easily defined. In some cases, it is acceptable to use the topwidth of the main channel to define these widths. Whether topwidth or bottom width is used, it is important to be consistent so that W_1 and W_2 refer to either bottom widths or top widths.

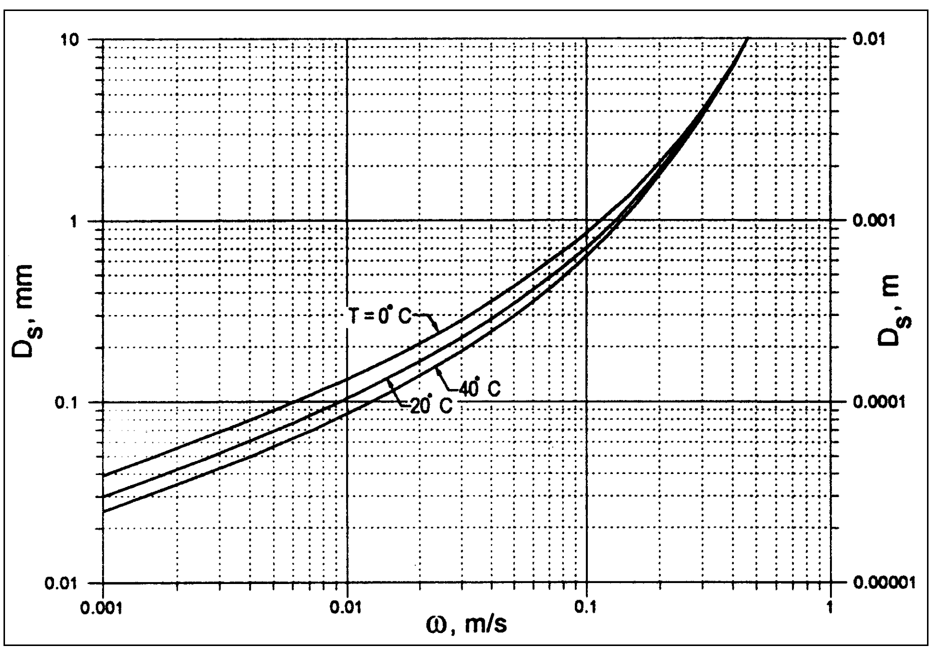


Figure 5.8. Fall velocity of sand-sized particles with specific gravity of 2.65 in metric units.

5. The average width of the bridge opening (W_2) is normally taken as the bottom width, with the width of the piers subtracted.

6. Laursen's equation will overestimate the depth of scour at the bridge if the bridge is located at the upstream end of a natural contraction or if the contraction is the result of the bridge abutments and piers. At this time, however, it is the best equation available.

7. In sand channel streams where the contraction scour hole is filled in on the falling stage, the y_0 depth may be approximated by y_1. Sketches or surveys through the bridge can help in determining the existing bed elevation.

8. **Scour depths with live-bed contraction scour may be limited by coarse sediments in the bed material armoring the bed. Where coarse sediments are present, it is recommended that scour depths be calculated for live-bed scour conditions using the clear-water scour equation (given in the next section) in addition to the live-bed equation, and that the smaller calculated scour depth be used.**

5.4 CLEAR-WATER CONTRACTION SCOUR

The recommended clear-water contraction scour equation is based on a development suggested by Laursen[44] (presented in the Appendix C). The equation is:

$$y_2 = \left[\frac{K_u Q^2}{D_m^{2/3} W^2} \right]^{3/7} \tag{5.4}$$

$$y_s = y_2 - y_o = \text{(average contraction scour depth)} \tag{5.5}$$

where:

y_2	=	Average equilibrium depth in the contracted section after contraction scour, m (ft)
Q	=	Discharge through the bridge or on the set-back overbank area at the bridge associated with the width W, m^3/s (ft^3/s)
D_m	=	Diameter of the smallest nontransportable particle in the bed material (1.25 D_{50}) in the contracted section, m (ft)
D_{50}	=	Median diameter of bed material, m (ft)
W	=	Bottom width of the contracted section less pier widths, m (ft)
y_o	=	Average existing depth in the contracted section, m (ft)
K_u	=	0.025 SI units
K_u	=	0.0077 English units

Equation 5.4 is a rearranged version of 5.1.

Because D_{50} is not the largest particle in the bed material, the scoured section can be slightly armored. Therefore, the D_m is assumed to be 1.25 D_{50}. For stratified bed material the depth of scour can be determined by using the clear-water scour equation sequentially with successive D_m of the bed material layers.

5.5 CONTRACTION SCOUR WITH BACKWATER

The **live-bed** contraction scour equation is derived assuming a uniform reach upstream and a long contraction into a uniform reach downstream of the bridge. With live-bed scour the equation computes a depth after the long contraction where the sediment transport into the downstream reach is equal to the sediment transport out. The **clear-water** contraction scour equations are derived assuming that the depth at the bridge increases until the shear-stress and velocity are decreased so that there is no longer any sediment transport. With the clear-water equations it is assumed that flow goes from one uniform flow condition to another. Both equations calculate contraction scour depth assuming a level water surface ($y_s = y_2 - y_o$). A more consistent computation would be to write an energy balance before and after the scour. For live-bed the energy balance would be between the approach section (1) and the contracted section (2). Whereas, for clear-water scour it would be the energy at the same section before (1) and after (2) the contraction scour.

Backwater, in extreme cases, can decrease the velocity, shear stress and the sediment transport in the upstream section. This will increase the scour at the contracted section. The backwater can, by storing sediment in the upstream section, change live-bed scour to clear-water scour.

5.6 CONTRACTION SCOUR EXAMPLE PROBLEMS (SI)

5.6.1 Example Problem 1 - Live-Bed Contraction Scour (SI)

Given:

The upstream channel width = 98.2 m; depth = 2.62 m

The discharge is 773 m^3/s and is all contained within the channel. Channel slope = 0.004 m/m

The bridge abutments consist of vertical walls with wing walls. Bridge width = 37.2 m; with 3 sets of piers consisting of 3 columns, 0.38 m in diameter.

The bed material size: from 0 to 0.9 m, the D_{50} is 0.31 mm and below 0.9 m the D_{50} is 0.70 mm with a fall velocity of 0.10 m/s

Original depth at bridge is estimated as 2.16 m

Determine:

The magnitude of the contraction scour depth.

Solution:

1. Determine if it is live-bed or clear-water scour.

Average velocity in the upstream reach

V = 773/ (2.62 X 98.2) = 3.0 m/s

For velocities this large and bed material this fine **live-bed** scour will occur. Check by calculating V_c for 0.7 mm bed material size. If live-bed scour occurs for 0.7mm it would also be live-bed for D_{50} = 0.3 mm.

$V_c = 6.19\ (2.62)^{1/6}\ (0.0007)^{1/3} = 0.65$ m/s

Live-bed contraction scour is verified

2. Calculate contraction scour

a. Determine k_1 for mode of bed material transport

$V_* = (9.81 \times 2.62 \times 0.004)^{0.5} = 0.32$ m/s

$\omega = 0.10$; $V_*/\omega = 3.2$; $k_1 = 0.69$

b. Live-bed contraction scour. Equation 5.2

$$\frac{y_2}{2.62} = \left[\frac{98.2}{36.06}\right]^{0.69} = 2.00$$

$Q_1 = Q_2$

$y_2 = 2.62 \times 2.00 = 5.24$ m from water surface.
$y_s = 5.24 - 2.16 = 3.08$ m from original bed surface

5.6.2 Example Problem 2 - Alternate Method (SI)

An alternative approach to calculating y_s in Problem 1 is to calculate the scour depth using both the clear-water and the live-bed equation and take the smaller scour depth.

a. Live bed-bed scour depth is 3.08 m from Problem 1.

b. Clear-water scour depth (Equation 5.4)

$$D_m = 1.25\ D_{50} = 1.25\ (0.0007) = 0.0009\ \text{m}$$

$$y_2 = \left[\frac{0.025\,(773)^2}{0.0009^{2/3}\,(36.06)^2}\right]^{3/7} = 21.12\,\text{m}$$

$y_s = 21.12 - 2.16 = 18.96$ m from original bed surface

c. Live-bed scour (3.08 m < 18.96 m). The sediment transport limits the contraction scour depth rather than the size of the bed material.

5.6.3 Example Problem 3 - Relief Bridge Contraction Scour (SI)

The 1952 flood on the Missouri River destroyed several relief bridges on Highway 2 in Iowa near Nebraska City, Nebraska. The USGS made continuous measurements during the period April 2 through April 29, 1952. This data set is from the April 21, 1952 measurement (measurement # 1013). The discharge in the relief bridge was 368 m^3/s. The measurement was made on the upstream side of Cooper Creek ditch using a boat and tag line.

$Q = 368\ m^3/s$; Bridge width (minus piers) = 91.4 m; Area = 706.43 m^2
$V_{average} = 0.52$ m/s; $y_0 = 1.28$ to 1.62 m

D_{50} = 0.24 mm (D_m = 1.25 x 0.24 = 0.3 mm)
Clear- water scour because of low velocity flow on the floodplain (Equation 5.4)
Calculate y_2:

$$y_2 = \left[\frac{0.025\,(368)^2}{(0.0003)^{2/3}\,(914)^2} \right]^{3/7} = 6.89\,\text{m}$$

y_2 = 6.89 m from the water surface, this compares to 7.71 m measured at the site.

5.7 CONTRACTION SCOUR EXAMPLE PROBLEMS (ENGLISH)

5.7.1 Example Problem 1 - Live-Bed Contraction Scour (English)

Given:

The upstream channel width = 322 ft; depth = 8.6 ft
The discharge is 27,300 cfs and is all contained within the channel. Channel slope = 0.004 (ft/ft)
The bridge abutments consist of vertical walls with wing walls, width = 122 ft; with 3 sets of piers consisting of 3 columns 15 inches in diameter.
The bed material size: from 0 to 3 ft the D_{50} is 0.31 mm (0.0010 ft) and below 3 ft the D_{50} is 0.70 mm (0.0023 ft) with a fall velocity of 0.33 ft/sec
Original depth at bridge is estimated as 7.1 ft

Determine:

The magnitude of the contraction scour depth.

Solution:

1. Determine if it is live-bed or clear-water scour.

Average velocity in the upstream reach

V = 27,300/(8.6 x 322) = 9.86 ft/s

For velocities this large and bed material this fine **live-bed** scour will occur. Check by calculating V_c for 0.7 mm bed material size. If live-bed scour occurs for 0.7mm it would also be live-bed for 0.3mm.

$V_c = 11.17\,(8.6)^{1/6}\,(0.0023)^{1/3} = 2.11$ ft/s

Live-bed contraction scour is verified

2. Calculate contraction scour

a. Determine K_1 for mode of bed material transport

$V_* = (32.2 \times 8.6 \times 0.004)^{0.5} = 1.05$ ft/s

ω = 0.33; V_*/ω = 3.2; K_1 =0.69

b. Live-bed contraction scour. Equation 5.2

$Q_1 = Q_2$

$$\frac{y_2}{8.6} = \left[\frac{322}{118.25}\right]^{0.69} = 2.00$$

y_2 = 8.6 x 2.00 = 17.2 ft from water surface.
y_s = 17.2 - 7.1 = 10.1 ft from original bed surface

5.7.2 Example Problem 2 - Alternate Method (English)

An alternative approach is demonstrated to calculating y_s in Problem 1 to determine if scour is clear-water or live-bed. In this method calculate the scour depth using both the clear-water and the live-bed equation and take the smaller scour depth.

a. Live-bed scour depth is 10.1 ft from Problem 1.

b. Clear-water scour depth (Equation 5.4)

$$D_m = 1.25\ D_{50} = 1.25\ (0.0023) = 0.0030\ \text{ft}$$

$$y_2 = \left[\frac{0.0077\ (27{,}300)^2}{0.0030^{2/3}\ (118.25)^2}\right]^{3/7} = 69.31\text{ft}$$

y_s = 69.31 - 7.1 = 62.2 ft from original bed surface

c. Live-bed scour (10.1 ft < 62.2 ft). The sediment transport limits the contraction scour depth rather than the size of the bed material.

5.7.3 Example Problem 3 - Relief Bridge Contraction Scour (English)

The 1952 flood on the Missouri River destroyed several relief bridges on Highway 2 in Iowa near Nebraska City, Nebraska. The USGS made continuous measurements during the period April 2 through April 29, 1952. This data set is from the April 21, 1952 measurement (measurement #1013). The discharge in the relief bridge was 13,012 cfs. The measurement was made on the upstream side of Cooper Creek ditch using a boat and tag line.

Q = 13,012 cfs; Bridge width (minus piers) = 300 ft; Area = 7,604 ft^2
$V_{average}$ = 1.71 ft/s; y_0 = 4.2 to 5.3 ft
D_{50} = 0.24 mm (D_m = 1.25 x 0.24 = 0.3 mm)
Clear- water scour because of low velocity flow on the floodplain (Equation 5.4)

$$y_2 = \left[\frac{0.0077\ (13{,}012)^2}{(0.0010)^{2/3}\ (300)^2}\right]^{3/7} = 22.6\ \text{ft}$$

y_2 = 22.6 ft from the water surface, this compares to 25.3 ft measured at the site.

5.8 OTHER GENERAL SCOUR CONDITIONS

5.8.1 Discussion

In a natural channel, the depth of flow is usually greater on the outside of a bend. In fact, there may well be deposition on the inner portion of the bend at a point bar. If a bridge is located on or close to a bend, the general scour will be concentrated on the outer portion of the bend. Also, in bends, the thalweg (the part of the stream where the flow is deepest and, typically, the velocity is the greatest) may shift toward the inside of the bend as the flow increases. This can increase scour and nonuniform distribution of scour in the bridge opening. In some cases during high flow the point bar may have a channel (chute channel) eroded across it. This can further skew the distribution of scour in the bridge reach. Consequently, other general scour conditions such as these are differentiated from contraction scour which involves removal of material from the bed across all or most of the channel width.

The relatively shallow straight reaches between bendway pools are called crossings. With changes in discharge and stage the patterns of scour and fill can also change in the crossing and pool sequence. These geomorphic processes are discussed in more detail in HEC-20 and HDS 6.[6,22] These processes are considered part of general scour. They are cyclic and may be in equilibrium around some general bed elevation. There are no equations for predicting these changes in elevation. Generally, a study of the stream using aerial photographs and/or successive cross section surveys can determine trends. In this case, the long-term safety of the bridge depends, primarily, on inspection.

Some general scour conditions are associated with a particular channel morphology. Braided channels will have deep scour holes when two channels come together downstream from a bar or island (confluence scour). At other times a bar or island will move into the bridge opening concentrating the flow onto a pier or abutment or changing the angle of attack. In anabranching flow, where flow is in two or more channels around semi-permanent islands, there is a problem of determining the distribution of flow between the channels, and over time the distribution may change. The bridge could be designed for the anticipated worst case flow distribution or designed using the present distribution. In either case, inspection and maintenance personnel should be informed of the potential for the flow distribution and scour conditions to change.

Other general scour can be caused by short-term (daily, weekly, yearly, or seasonal) changes in the downstream water surface elevation that control backwater and hence, the velocity through the bridge opening. Similarly, a bridge located upstream or downstream of a confluence can experience general scour caused by variable flow conditions on the main river and tributary. Because this scour is reversible, it is considered other general scour rather than long-term aggradation or degradation. These channel changes and other general scour conditions are also discussed in HEC-20 and HDS 6.[6,22]

5.8.2 Determining Other General Scour

Scour at a bridge cross-section resulting from variable water surface elevation downstream of the bridge (e.g., tributary or downstream control) is analyzed by determining the lowest potential water-surface elevation downstream of the bridge insofar as scour processes are concerned. Then determine contraction and local scour depths using these worst-case conditions.

General scour in a channel bendway resulting from the flow through the bridge being concentrated toward the outside of the bend is analyzed by determining the superelevation of the water surface on the outside of the bend and estimating the resulting velocities and depths through the bridge. The maximum velocity in the outer part of the bend can be 1.5 to 2 times the mean velocity. A physical model study can also be used to determine the velocity and scour depth distribution through the bridge for this case.

Estimating general scour across the bridge cross-section for unusual situations involves particular skills in the application of principles of river mechanics to the site-specific conditions. To determine the scour across the bridge opening in many bridge crossings will require 2-dimensional (2-D) computer programs (for example, FESWMS(45) - see discussion Chapter 9, Section 9.5) or a physical model (HEC-23).(7) Such studies should be undertaken by engineers experienced in the fields of hydraulics and river mechanics.

CHAPTER 6

DETERMINATION OF LOCAL SCOUR AT PIERS

6.1 GENERAL

Local scour at piers is a function of bed material characteristics, bed configuration, flow characteristics, fluid properties, and the geometry of the pier and footing. The bed material characteristics are granular or non granular, cohesive or noncohesive, erodible or non erodible rock. Granular bed material ranges in size from silt to large boulders and is characterized by the D_{50} and a coarse size such as the D_{84} or D_{90} size. Cohesive bed material is composed of silt and clay, possibly with some sand which is bonded chemically (see discussion in Chapter 3). Rock may be solid, massive, or fractured. It may be sedimentary or igneous and erodible or non erodible.

Flow characteristics of interest for local pier scour are the velocity and depth just upstream of the pier, the angle the velocity vector makes to the pier (angle of attack), and free surface or pressure flow. Fluid properties are viscosity, and surface tension which for the field case can be ignored.

Pier geometry characteristics are its type, dimensions, and shape. Types of piers include single column, multiple columns, or rectangular; with or without friction or tip bearing piles; with or without a footing or pile cap; footing or pile cap in the bed, on the surface of the bed, in the flow or under the deck out of the flow. Important dimensions are the diameter for circular piers or columns, spacing for multiple columns, and width and length for solid piers. Shapes include round, square or sharp nose, circular cylinder, group of cylinders, or rectangular. In addition, piers may be simple or complex. A simple pier is a single shaft, column or multiple columns exposed to the flow. Whereas, a complex pier may have the pier, footing or pile cap, and piles exposed to the flow.

Local scour at piers has been studied extensively in the laboratory; however, there is limited field data. The laboratory studies have been mostly of simple piers, but there have been some laboratory studies of complex piers. Often the studies of complex piers are model studies of actual or proposed pier configurations. As a result of the many laboratory studies, there are numerous pier scour equations. In general, the equations are for live-bed scour in cohesionless sand-bed streams.

A graphical comparison by Jones of the more common equations is given in Figure 6.1.[(46)] An equation given by Melville and Sutherland to calculate scour depths for live-bed scour in sand-bed streams has been added to the original figure.[(28)] Some of the equations have velocity as a variable, normally in the form of a Froude Number. However, some equations, such as Laursen's do not include velocity.[(43)] A Froude Number of 0.3 was used in Figure 6.1 for purposes of comparing commonly used scour equations. Jones also compared the equations with the available field data. His study showed that the CSU equation enveloped all the data, but gave lower values of scour than the Jain and Fischer, Laursen, Melville and Sutherland, and Neill equations.[(22,47,48,28,46)] The CSU equation includes the velocity of the flow just upstream of the pier by including the Froude Number in the equation. On the basis of Jones' studies[(46)] the Colorado State University (CSU) equation was recommended in the Interim Procedures that accompanied FHWA's Technical Advisory T5140.20.[(11,9)] With modifications, the CSU equation was recommended in previous editions of HEC-18. The modifications were the addition of coefficients for the effect of bed form and size of bed material.

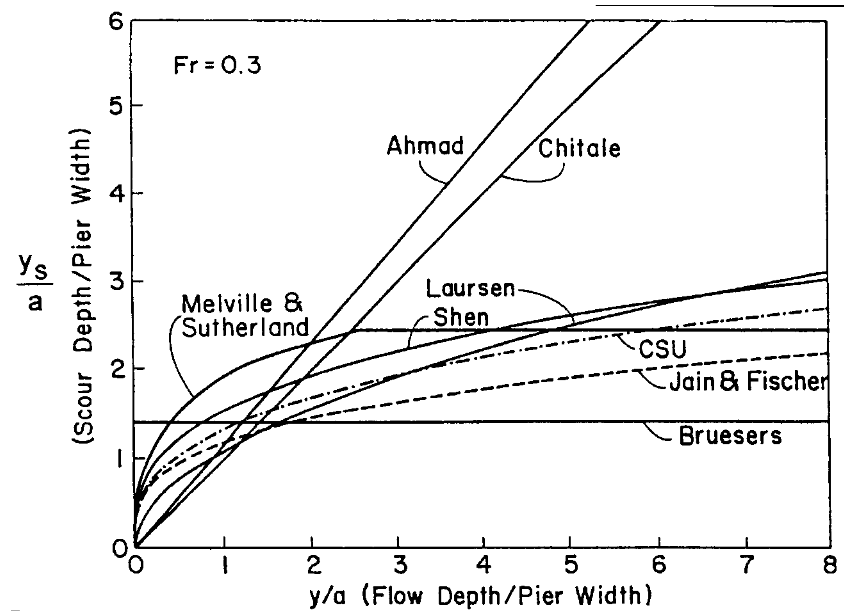


Figure 6.1. Comparison of scour equations for variable depth ratios (y/a) (after Jones).[46]

Mueller[49] compared 22 scour equations using field data collected by the USGS[50]. He concluded that the HEC-18 equation was good for design because it rarely under predicted measured scour depth. However, it frequently over-predicted the observed scour. The data contained 384 field measurements of scour at 56 bridges (Figure 6.2).

From laboratory data, Melville and Sutherland reported 2.4 as an upper limit for the depth of scour to pier width ratio (y_s/a) for cylindrical piers.[28] In these studies, the Froude Number was less than 1.0. Chang[51] also, noted that in all the data he studied, there were no values of the ratio of scour depth to pier width (y_s/a) larger than 2.3. However, values of y_s/a around 3.0 were obtained by Jain and Fischer for chute-and-pool flows with Froude Numbers as high as 1.5.[47] The largest value of y_s/a for antidune flow was 2.5 with a Froude Number of 1.2. These upper limits were derived for circular piers and were uncorrected for pier shape or for skew. Also, pressure flow, ice or debris can increase the ratio.

From the above discussion, the ratio of y_s/a can be as large as 3 at large Froude Numbers. Therefore, it is recommended that the maximum value of the ratio be taken as 2.4 for Froude Numbers less than or equal to 0.8 and 3.0 for larger Froude Numbers. These limiting ratio values apply only to round nose piers which are aligned with the flow.

6.2 LOCAL PIER SCOUR EQUATION

To determine pier scour, an equation based on the CSU equation is recommended for both live-bed and clear-water pier scour.[22] The equation predicts maximum pier scour depths. The equation is:

$$\frac{y_s}{y_1} = 2.0\, K_1\, K_2\, K_3\, K_4 \left(\frac{a}{y_1}\right)^{0.65} Fr_1^{0.43} \qquad (6.1)$$

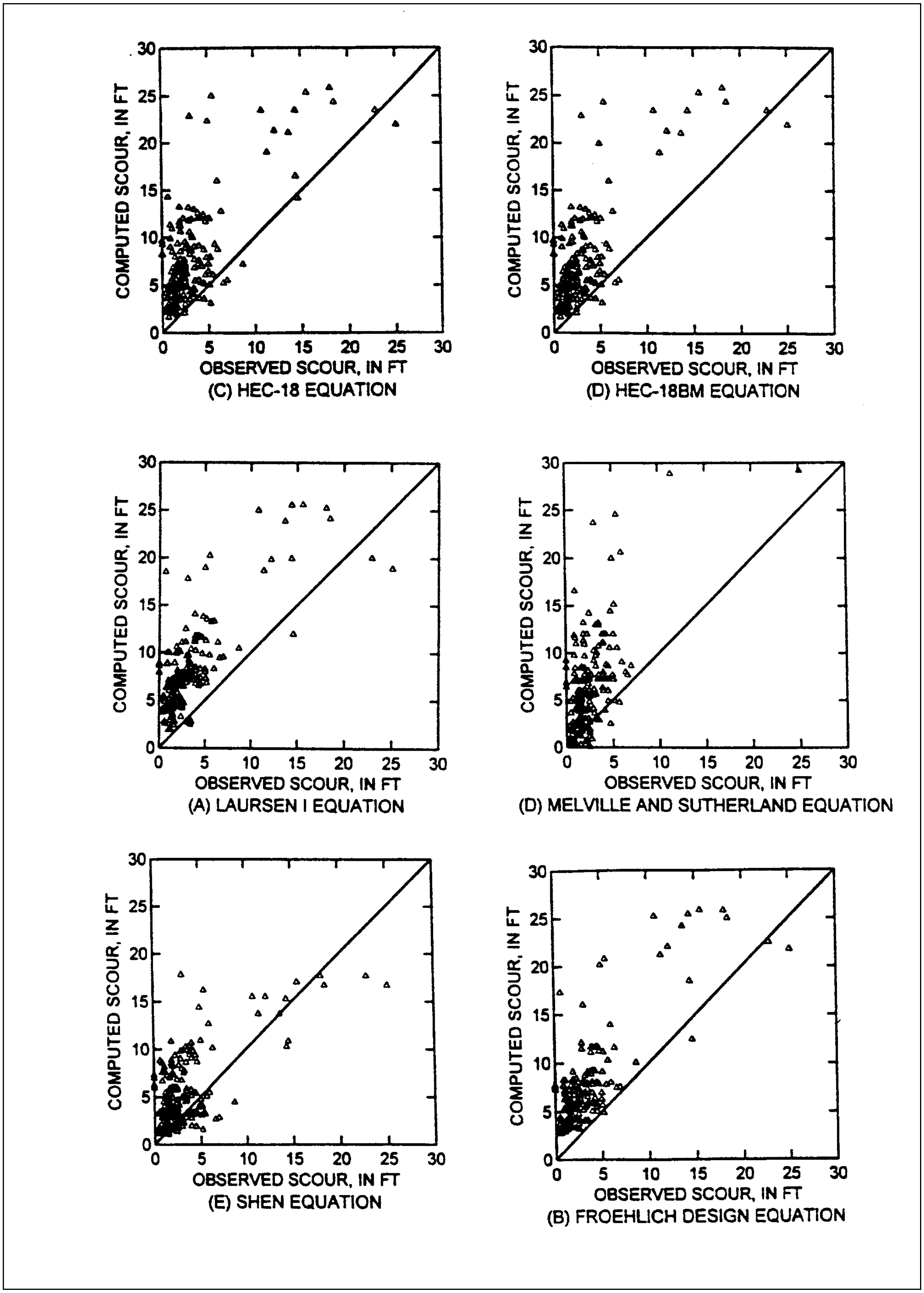


Figure 6.2. Comparison of scour equations with field scour measurements (after Mueller).[49]

As a Rule of Thumb, the maximum scour depth for round nose piers aligned with the flow is:

$y_s \leq$ 2.4 times the pier width (a) for $Fr \leq 0.8$ (6.2)
$y_s \leq$ 3.0 times the pier width (a) for $Fr > 0.8$

In terms of y_s/a, Equation 6.1 is:

$$\frac{y_s}{a} = 2.0\, K_1\, K_2\, K_3\, K_4 \left(\frac{y_1}{a}\right)^{0.35} Fr_1^{0.43} \quad (6.3)$$

where:

y_s	=	Scour depth, m (ft)
y_1	=	Flow depth directly upstream of the pier, m (ft)
K_1	=	Correction factor for pier nose shape from Figure 6.3 and Table 6.1
K_2	=	Correction factor for angle of attack of flow from Table 6.2 or Equation 6.4
K_3	=	Correction factor for bed condition from Table 6.3
K_4	=	Correction factor for armoring by bed material size from Equation 6.5
a	=	Pier width, m (ft)
L	=	Length of pier, m (ft)
Fr_1	=	Froude Number directly upstream of the pier = $V_1/(gy_1)^{1/2}$
V_1	=	Mean velocity of flow directly upstream of the pier, m/s (ft/s)
g	=	Acceleration of gravity (9.81 m/s^2) (32.2 ft/s^2)

The correction factor, K_2, for angle of attack of the flow, θ, is calculated using the following equation:

$$K_2 = (\text{Cos}\ \theta + L / a\ \text{Sin}\ \theta)^{0.65} \quad (6.4)$$

If L/a is larger than 12, use L/a = 12 as a maximum in Equation 6.4 and Table 6.2. Table 6.2 illustrates the magnitude of the effect of the angle of attack on local pier scour.

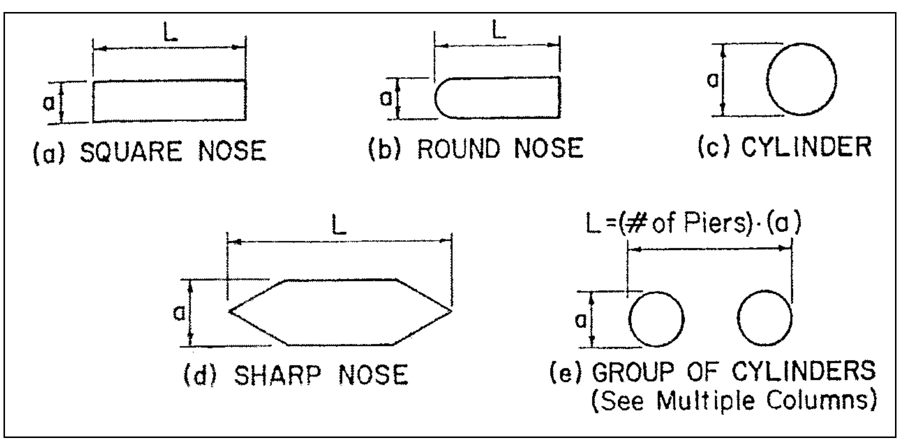


Figure 6.3. Common pier shapes.

Table 6.1. Correction Factor, K_1, for Pier Nose Shape.	
Shape of Pier Nose	K_1
(a) Square nose	1.1
(b) Round nose	1.0
(c) Circular cylinder	1.0
(d) Group of cylinders	1.0
(e) Sharp nose	0.9

Table 6.2. Correction Factor, K_2, for Angle of Attack, θ, of the Flow.			
Angle	L/a=4	L/a=8	L/a=12
0	1.0	1.0	1.0
15	1.5	2.0	2.5
30	2.0	2.75	3.5
45	2.3	3.3	4.3
90	2.5	3.9	5.0
Angle = skew angle of flow L = length of pier, m			

Table 6.3. Increase in Equilibrium Pier Scour Depths, K_3, for Bed Condition.		
Bed Condition	Dune Height m	K_3
Clear-Water Scour	N/A	1.1
Plane bed and Antidune flow	N/A	1.1
Small Dunes	$3 > H \geq 0.6$	1.1
Medium Dunes	$9 > H \geq 3$	1.2 to 1.1
Large Dunes	$H \geq 9$	1.3

Notes:

1. The correction factor K_1 for pier nose shape should be determined using Table 6.1 for angles of attack up to 5 degrees. **For greater angles, K_2 dominates and K_1 should be considered as 1.0**. If L/a is larger than 12, use the values for L/a = 12 as a maximum in Table 6.2 and Equation 6.4.

2. The values of the correction factor K_2 should be applied only when the field conditions are such that the entire length of the pier is subjected to the angle of attack of the flow. Use of this factor will result in a significant over-prediction of scour if (1) a portion of the pier is shielded from the direct impingement of the flow by an abutment or another pier; or (2) an abutment or another pier redirects the flow in a direction parallel to the pier. For such cases, judgment must be exercised to reduce the value of the K_2 factor by selecting the effective length of the pier actually subjected to the angle of attack of the flow. **Equation 6.4 should be used for evaluation and design**. Table 6.2 is intended to illustrate the importance of angle of attack in pier scour computations and to establish a cutoff point for K_2 (i.e., a maximum value of 5.0).

3. The correction factor K_3 results from the fact that for plane-bed conditions, which is typical of most bridge sites for the flood frequencies employed in scour design, the maximum scour may be 10 percent greater than computed with Equation 6.1. In the **unusual** situation where a dune bed configuration **with large dunes** exists at a site during flood flow, the maximum pier scour may be 30 percent greater than the predicted equation value. This may occur on very large rivers, such as the Mississippi. For smaller streams that have a dune bed configuration at flood flow, the dunes will be smaller and the maximum scour may be only 10 to 20 percent larger than equilibrium scour. For antidune bed configuration the maximum scour depth may be 10 percent greater than the computed equilibrium pier scour depth.

4. Piers set close to abutments (for example at the toe of a spill through abutment) must be carefully evaluated for the angle of attack and velocity of the flow coming around the abutment.

The correction factor K_4 decreases scour depths for armoring of the scour hole for bed materials that have a D_{50} equal to or larger than 2.0 mm and D_{95} equal to or larger than 20 mm. The correction factor results from recent research by Molinas and Mueller. Molinas's research for FHWA showed that when the approach velocity (V_1) is less than the critical velocity (V_{c90}) of the D_{90} size of the bed material and there is a gradation in sizes in the bed material, the D_{90} will limit the scour depth.[30, 52] Mueller and Jones[53] developed a K_4 correction coefficient from a study of 384 field measurements of scour at 56 bridges. The equation developed by Jones[54] given in HEC-18 Third Edition should be replaced with the following:

- If D_{50} < 2 mm or D_{95} < 20 mm, then $K_4 = 1$
- If $D_{50} \geq$ 2 mm and $D_{95} \geq$ 20 mm

then:

$$K_4 = 0.4\,(V_R)^{0.15} \tag{6.5}$$

where:

$$V_R = \frac{V_1 - V_{icD_{50}}}{V_{cD_{50}} - V_{icD_{95}}} > 0 \tag{6.6}$$

and:

V_{icDx} = approach velocity (m/s or ft/sec) required to initiate scour at the pier for the grain size D_x (m or ft)

$$V_{icD_x} = 0.645\left(\frac{D_x}{a}\right)^{0.053} V_{cD_x} \tag{6.7}$$

V_{cDx} = critical velocity (m/s or ft/s) for incipient motion for the grain size D_x (m or ft)

$$V_{cD_x} = K_u\, y_1^{1/6}\, D_x^{1/3} \tag{6.8}$$

where:

y_1	=	Depth of flow just upstream of the pier, excluding local scour, m (ft)	
V_1	=	Velocity of the approach flow just upstream of the pier, m/s (ft/s)	
D_x	=	Grain size for which x percent of the bed material is finer, m (ft)	
K_u	=	6.19	SI Units
K_u	=	11.17	English Units

While K_4 provides a good fit with the field data the velocity ratio terms are so formed that if D_{50} is held constant and D_{95} increases, the value of K_4 increases rather than decreases.[53] For field data an increase in D_{95} was always accompanied with an increase in D_{50}. **The minimum value of K_4 is 0.4**.

6.3 PIER SCOUR CORRECTION FACTOR FOR VERY WIDE PIERS

Flume studies on scour depths at wide piers in shallow flows and field observations of scour depths at bascule piers in shallow flows indicate that existing equations, including the CSU equation, overestimate scour depths. Johnson and Torrico[55] suggest the following equations for a K_w factor to be used to correct Equation 6.1 or 6.3 for wide piers in shallow flow. **The correction factor should be applied when the ratio of depth of flow (y) to pier width (a) is less than 0.8 (y/a < 0.8); the ratio of pier width (a) to the median diameter of the bed material (D_{50}) is greater than 50 (a/D_{50} > 50); and the Froude Number of the flow is subcritical**.

$$K_w = 2.58\left(\frac{y}{a}\right)^{0.34} Fr_1^{0.65} \quad \text{for } V/V_c < 1 \tag{6.9}$$

$$K_w = 10\left(\frac{y}{a}\right)^{0.13} Fr_1^{0.25} \quad \text{for } V/V_c \geq 1 \tag{6.10}$$

where:

K_w = Correction factor to Equation 6.1 or 6.3 for wide piers in shallow flow.
The other variables as previously defined.

Engineering judgment should be used in applying K_w because it is based on limited data from flume experiments. Engineering judgment should take into consideration the volume of traffic, the importance of the highway, cost of a failure (potential loss of lives and dollars) and the change in cost that would occur if the K_w factor is used.

6.4 SCOUR FOR COMPLEX PIER FOUNDATIONS

6.4.1 Introduction

As Salim and Jones[56,57,58] point out most pier scour research has focused on **solid piers** with limited attention to the determining scour depths for (1) pile groups, (2) pile groups and pile caps, or (3) pile groups, pile caps and solid piers exposed to the flow. The three types of exposure to the flow may be by design or by scour (long-term degradation, general (contraction) scour, and local scour, in addition to stream migration). In the general case, the flow could be obstructed by three substructural elements, herein referred to as the scour-producing components, which include the pier stem, the pile cap or footing, and the pile group. Nevertheless, ongoing research has determined methods and equations to determine scour depths for complex pier foundations. The results of this research are recommended for use and are given in the following sections. Physical Model studies are still recommended for complex piers with unusual features such as staggered or unevenly spaced piles or for major bridges where conservative scour estimates are not economically acceptable. However, the methods presented in this section provide a good estimate of scour for a variety of complex pier situations.

The steps listed below are recommended for determining the depth of scour for any combination of the three substructural elements exposed to the flow,[59] but engineering judgment is an essential element in applying the design graphs and equations presented in this section as well as in deciding when a more rigorous level of evaluation is warranted. Engineering judgment should take into consideration the volume of traffic, type of traffic (school bus, ambulance, fire trucks, local road, interstate, etc.), the importance of the highway, cost of a failure (potential loss of life and dollars) and the increase in cost that would occur if the most conservative scour depth is used. The stability of the foundation should be checked for:

- The scour depths should be determined for the 100-year flood or smaller discharge if it causes deeper scour and the superflood, i.e., the 500-year flood, as recommended in this manual.

- If needed use computer programs (HEC-RAS,[16, 17] WSPRO,[15] FESWMS,[45] etc.) to compute the hydraulic variables.

- Total scour depth is determined by separating the scour producing components, determining the scour depth for each component and adding the results. The method is called **"Superposition of the Scour Components."**

- Analyze the complex pile configuration to determine the components of the pier that are exposed to the flow or will be exposed to the flow which will cause scour.

- Determine the scour depths for each component exposed to the flow using the equations and methods presented in the following sections.

- Add the components to determine the total scour depths.

- Plot the scour depths and analyze the results using an interdisciplinary team to determine their reliability and adequacy for the bridge, flow and site conditions, safety and costs.

- Conduct a physical model study (Section 6.9) if engineering judgment determines it will reduce uncertainly, increase the safety of the design and/or reduce cost.

6.4.2 Superposition of Scour Components Method of Analysis

The components of a complex pier are illustrated in Figure 6.4.[59] This is followed by a definition of the variables. Note that the pile cap can be above the water surface, at the water surface, in the water or on the bed. The location of the pile cap may result from design or from long-term degradation and/or contraction scour. The pile group, as illustrated, is in uniform (lined up) rows and columns. This may not always be the case. The support for the bridge in many flow fields and designs may require a more complex arrangement of the pile group. In more complex pile group arrangements, the methods of analysis given in this manual may give smaller or larger scour depths.

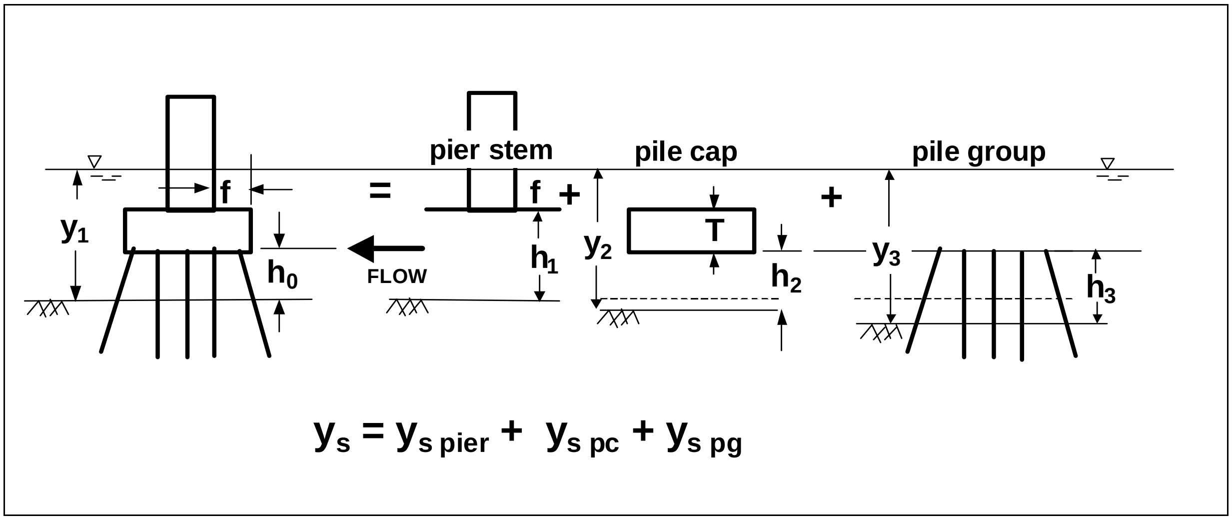


Figure 6.4. Definition sketch for scour components for a complex pier.[59]

The variables illustrated in Figure 6.4 and others used in computations are as follows:

f	=	Distance between front edge of pile cap or footing and pier, m (ft)
h_o	=	Height of the pile cap above bed at beginning of computation, m (ft)
h_1	=	$h_o + T$ = height of the pier stem above the bed before scour, m (ft)
h_2	=	$h_o + y_{s\ pier}/2$ = height of pile cap after pier stem scour component has been computed, m (ft)
h_3	=	$h_o + y_{s\ pier}/2 + y_{s\ pc}/2$ = height of pile group after the pier stem and pile cap scour components have been computed, m (ft)
S	=	Spacing between columns of piles, pile center to pile center, m (ft)
T	=	Thickness of pile cap or footing, m (ft)
y_1	=	Approach flow depth at the beginning of computations, m (ft)
y_2	=	$y_1 + y_{s\ pier}/2$ = adjusted flow depth for pile cap computations m (ft)
y_3	=	$y_1 + y_{s\ pier}/2 + y_{s\ pc}/2$ = adjusted flow depth for pile group computations, m (ft)
V_1	=	Approach velocity used at the beginning of computations, m/sec (ft/sec)
V_2	=	$V_1(y_1/y_2)$ =adjusted velocity for pile cap computations, m/sec (ft/sec)
V_3	=	$V_1(y_1/y_3)$ =adjusted velocity for pile group computations, m/sec (ft/sec)

Total scour from superposition of components is given by:

$$y_s = y_{s\ pier} + y_{s\ pc} + y_{s\ pg} \tag{6.11}$$

where:

y_s	=	Total scour depth, m (ft)
$y_{s\ pier}$	=	Scour component for the pier stem in the flow, m (ft)
$y_{s\ pc}$	=	Scour component for the pier cap or footing in the flow, m (ft)
$y_{s\ pg}$	=	Scour component for the piles exposed to the flow, m (ft)

Each of the scour components is computed from the basic pier scour Equation 6.1 using an equivalent sized pier to represent the irregular pier components, adjusted flow depths and velocities as described in the list of variables for Figure 6.4, and height adjustments for the pier stem and pile group. The height adjustment is included in the equivalent pier size for the pile cap. In the following sections guidance for calculating each of the components is given.

6.4.3 Determination of the Pier Stem Scour Depth Component

The need to compute the pier stem scour depth component occurs when the pier cap or the footing is in the flow and the pier stem is subjected to sufficient flow depth and velocity as to cause scour. The first computation is the scour estimate, $y_{s\ pier}$, for a full depth pier that has the width and length of the pier stem using the basic pier equation (Equation 6.1). In Equation 6.1, a_{pier} is the pier width and other variables in the equation are as defined previously. This base scour estimate is multiplied by $K_{h\ pier}$, given in Figure 6.5 as a function of h_1/a_{pier} and f/a_{pier}, to yield the pier stem scour component as follows:

$$\frac{y_{s\,pier}}{y_1} = K_{h\,pier}\left[2.0K_1K_2K_3K_4\left(\frac{a_{pier}}{y_1}\right)^{0.65}\left(\frac{V_1}{\sqrt{g y_1}}\right)^{0.43}\right] \qquad (6.12)$$

where:

$K_{h\ pier}$ = Coefficient to account for the height of the pier stem above the bed and the shielding effect by the pile cap overhang distance "f" in front of the pier stem (from Figure 6.5)

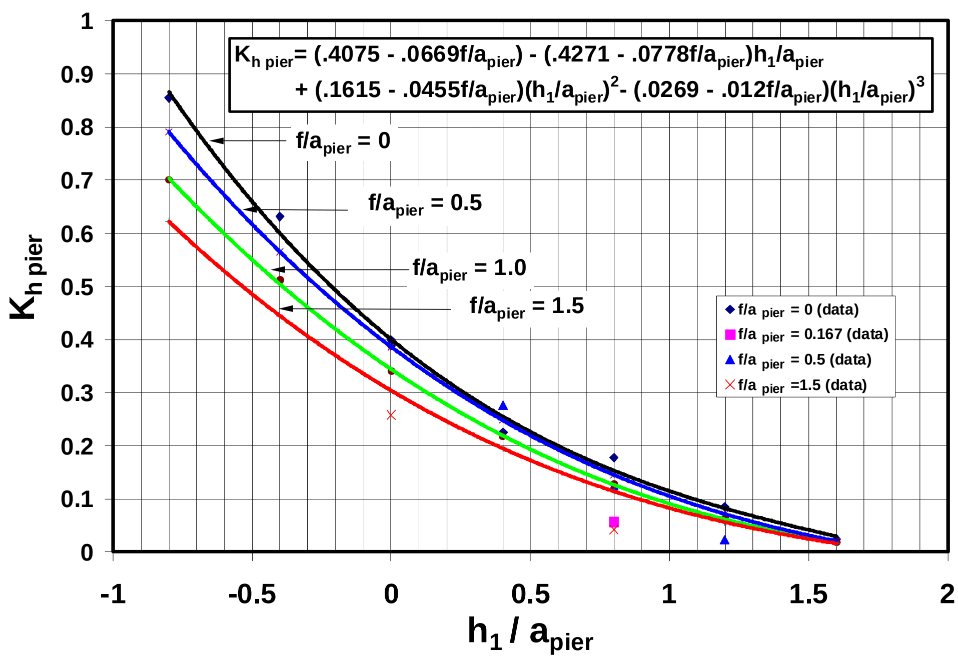


Figure 6.5. Suspended pier scour ratio.[59]

The quantity in the square brackets in Equation 6.12 is the basic pier scour ratio as if the pier stem were full depth and extended below the scour.

6.4.4 Determination of the Pile Cap (Footing) Scour Depth Component

The need to compute the pile cap or footing scour depth component occurs when the pile cap is in the flow by design, or as the result of long-term degradation, contraction scour, and/or by local scour attributed to the pier stem above it. As described below, there are two cases to consider in estimating the scour caused by the pile cap (or footing). Equation 6.1 is used to estimate the scour component in both cases, but the conceptual strategy for determining the variables to be used in the equation is different (partly due to limitations in the research that has been done to date). In both cases the wide pier factor, K_w, in Section 6.3 may be applicable for this computation.

Case 1: The bottom of the pile cap is above the bed and in the flow either by design or after the bed has been lowered by scour caused by the pier stem component. The strategy is to reduce the pile cap width, a_{pc}, to an equivalent full depth solid pier width, a^*_{pc}, using Figure 6.6. The equivalent pier width, an adjusted flow depth, y_2, and an adjusted flow velocity, V_2, are then used in Equation 6.1 to estimate the scour component.

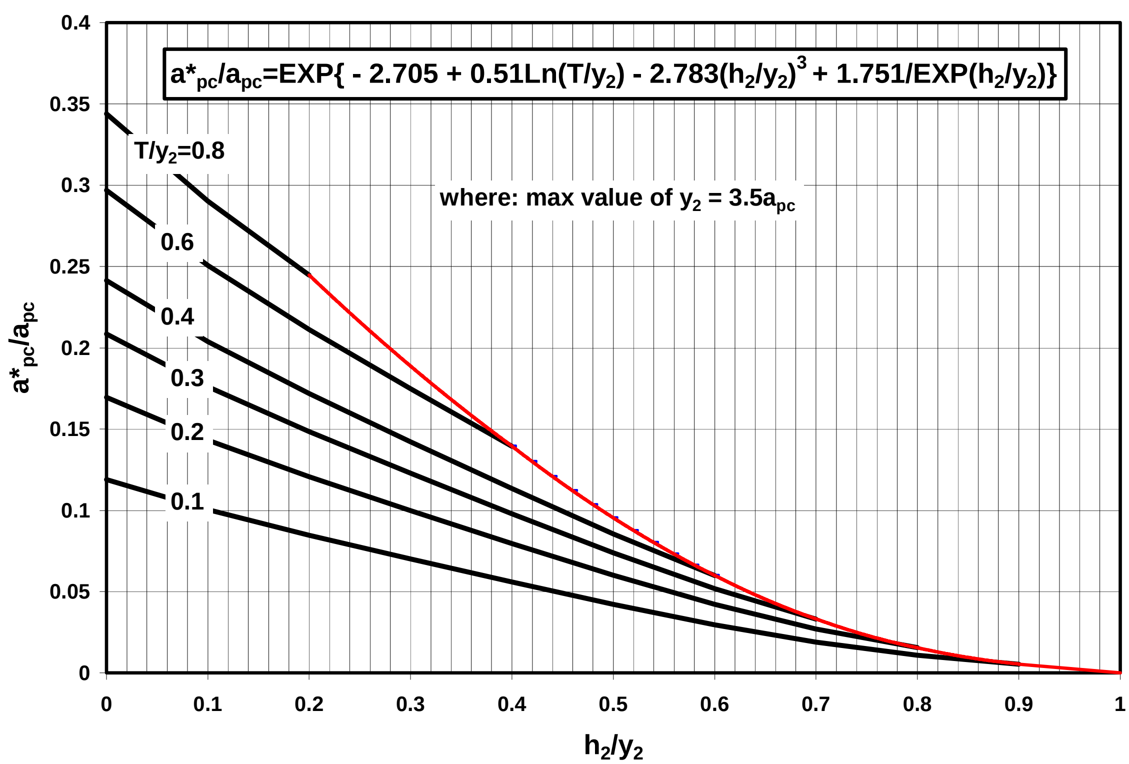


Figure 6.6. Pile cap (footing) equivalent width.[59]

Case 2: The bottom of the pile cap or footing is on or below the bed. The strategy is to treat the pile cap or exposed footing like a short pier in a shallow stream of depth equal to the height to the top of the footing above bed. The portion of the flow that goes over the top of the pile cap or footing is ignored. Then, the full pile cap width, a_{pc}, is used in the computations, but the exposed footing height, y_f, (in lieu of the flow depth), and the average velocity, V_f, in the portion of the profile approaching the footing are used in Equation 6.1 to estimate the scour component.

An inherent assumption in this second case is that the footing is deeper than the scour depth so it is not necessary to add the pile group scour as a third component in this case. If the bottom of the pile cap happens to be right on the bed, either the case 1 or case 2 method could be applied, but they won't necessarily give the same answers. If both methods are tried, then engineering judgment should dictate which one to accept.

Details for determining the pile cap or footing scour component for these two cases are described in the following paragraphs.

Case 1. Bottom of the Pile Cap (Footing) in the Flow above the Bed

T = Thickness of the pile cap exposed to the flow, m (ft)
h_2 = $h_o + y_{s\ pier}/2$, m (ft)
y_2 = $y_1 + y_{s\ pier}/2$, = adjusted flow depth, m (ft)
V_2 = $V_1(y_1/y_2)$ = adjusted flow velocity, m/s (ft/s)

where:

h_o = Original height of the pile cap above the bed, m (ft)
y_1 = Original flow depth at the beginning of the computations before scour, m (ft)
$y_{s\ pier}$ = Pier stem scour depth component, m (ft)
V_1 = Original approach velocity at the beginning of the computations, m/s (ft/s)

Determine a^*_{pc}/a_{pc} from Figure 6.6 as a function of h_2/y_2 and T/y_2 (note that the maximum value of $y_2 = 3.5\ a_{pc}$).

Compute $a^*_{pc} = (a^*_{pc}/a_{pc})\ a_{pc}$; where a^*_{pc} is the width of the equivalent pier to be used in Equation 6.1 and a_{pc} is the width of the original pile cap. Compute the pile cap scour component, $y_{s\ pc}$ from Equation 6.1 using a^*_{pc}, y_2, and V_2 as the pier width, flow depth, and velocity parameters, respectively. The rationale for using the adjusted velocity for this computation is that the near bottom velocities are the primary currents that produce scour and they tend to be reduced in the local scour hole from the overlying component. **For skewed flow use the L/a for the original pile cap as the L/a for the equivalent pier to determine K_2.** Apply the wide pier correction factor, K_w, if (1) the total depth, $y_2 < 0.8\ a^*_{pc}$, (2) the Froude Number $V_2/(g\ y_2)^{1/2} < 1$, and (3) $a^*_{pc} > 50\ D_{50}$. The scour component equation for the case 1 pile cap can then be written:

$$\frac{y_{s\,pc}}{y_2} = 2.0 K_1 K_2 K_3 K_4 K_w \left(\frac{a^*_{pc}}{y_2}\right)^{0.65} \left(\frac{V_2}{\sqrt{g y_2}}\right)^{0.43} \qquad (6.13)$$

Next, the pile group scour component should be computed. This is discussed in Section 6.4.5.

Case 2. Bottom of the Pile Cap (Footing) Located On or Below the Bed.

One limitation of the procedure described above is that the design chart in Figure 6.6 has not been developed for the case of the bottom of the pile cap or footing being below the bed (i.e., negative values of h_2). In this case, use a modification of the exposed footing procedure that has been described in previous editions of HEC-18. The previous procedure was developed from experiments in which the footing was never undermined by scour and tended to be an over predictor if the footing is undermined.

As for case 1:

$$y_2 = y_1 + y_{s\,pier}/2, \text{ m (ft)}$$
$$V_2 = V_1(y_1/y_2), \text{ m/s (ft/s)}$$

The average velocity of flow at the exposed footing (V_f) is determined using the following equation:

$$\frac{V_f}{V_2} = \frac{\ln\left(10.93\frac{y_f}{k_s}+1\right)}{\ln\left(10.93\frac{y_2}{k_s}+1\right)} \tag{6.14}$$

where:

- V_f = Average velocity in the flow zone below the top of the footing, m/s (ft/s)
- V_2 = Average adjusted velocity in the vertical of flow approaching the pier, m/s (ft/s)
- ln = Natural log to the base e
- y_f = $h_1 + y_{s\,pier}/2$ = distance from the bed (after degradation, contraction scour, and pier stem scour) to the top of the footing, m (ft)
- k_s = Grain roughness of the bed (normally taken as the D_{84} for sand size bed material and 3.5 D_{84} for gravel and coarser bed material), m (ft)
- y_2 = Adjusted depth of flow upstream of the pier, including degradation, contraction scour and half the pier stem scour, m (ft)

See Figure 6.7 for an illustration of variables.

Compute the pile cap scour depth component, $y_{s\,pc}$ from Equation 6.1 using the full pile cap width, a_{pc}, y_f, V_f as the width, flow depth, and velocity parameters, respectively. The wide pier factor K_w in Section 6.3 should be used in this computation if (1) the total depth $y_2 < 0.8\, a_{pc}$, (2) the Froude Number $V_2/(gy_2)^{1/2} < 1$, and (3) $a_{pc} > 50\, D_{50}$. Use y_2/a_{pc} to compute the K_w factor if it is applicable. The scour component equation for the case 2 pile cap or footing can then be written:

$$\frac{y_{s\,pc}}{y_f} = 2.0 K_1 K_2 K_3 K_4 K_w \left(\frac{a_{pc}}{y_f}\right)^{0.65}\left(\frac{V_f}{\sqrt{g\,y_f}}\right)^{0.43} \tag{6.15}$$

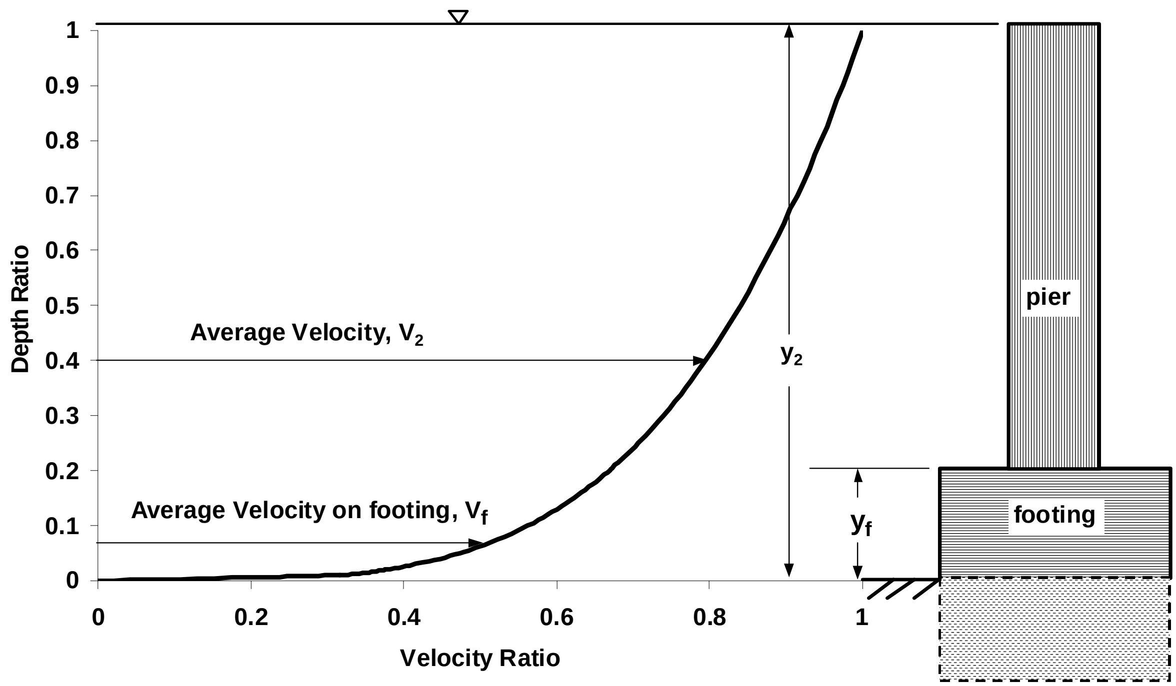


Figure 6.7. Definition sketch for velocity and depth on exposed footing.

In this case assume the pile cap scour component includes the pile group scour and compute the total scour depth as:

$$y_s = y_{s\,pier} + y_{s\ pc} \qquad \text{(For case 2 only)} \tag{6.16}$$

In earlier editions of HEC-18, the recommendation was to use the larger of the exposed footing scour estimate or the pier stem scour estimate, treating the pier stem portion as a full depth pier that extended below the scour depth. **Now the recommendation is to add the components using a more realistic estimate of the pier stem component and using an adjusted approach velocity, V_2, to calculate V_f and the wide pier correction in the computations for the exposed footing component.**

6.4.5 Determination of the Pile Group Scour Depth Component

Research by Salim and Jones[56,57,58,60] and by Smith[61] has provided a basis for determining pile group scour depth by taking into consideration the spacing between piles, the number of pile rows and a height factor to account for the pile length exposed to the flow. Guidelines are given for analyzing the following typical cases:

- Special case of piles aligned with each other and with the flow. No angle of attack.

- General case of the pile group skewed to the flow, with an angle of attack, or pile groups with staggered rows of piles.

The strategy for estimating the pile group scour component is the same for both cases, but the technique for determining the projected width of piles is simpler for the special case of aligned piles. The strategy is as follows:

- Project the width of the piles onto a plane normal to the flow.
- Determine the effective width of an equivalent pier that would produce the same scour if the pile group penetrated the water surface.
- Adjust the flow depth, velocity and exposed height of the pile group to account for the pier stem and pile cap scour components previously calculated.
- Determine the pile group height factor based on the exposed height of the pile group above the bed.
- Compute the pile group scour component using a modified version of Equation 6.1.

<u>Projected width of piles</u>

For the special case of aligned piles, the projected width, a_{proj}, onto a plane normal to the flow is simply the width of the collapsed pile group as illustrated in Figure 6.8.

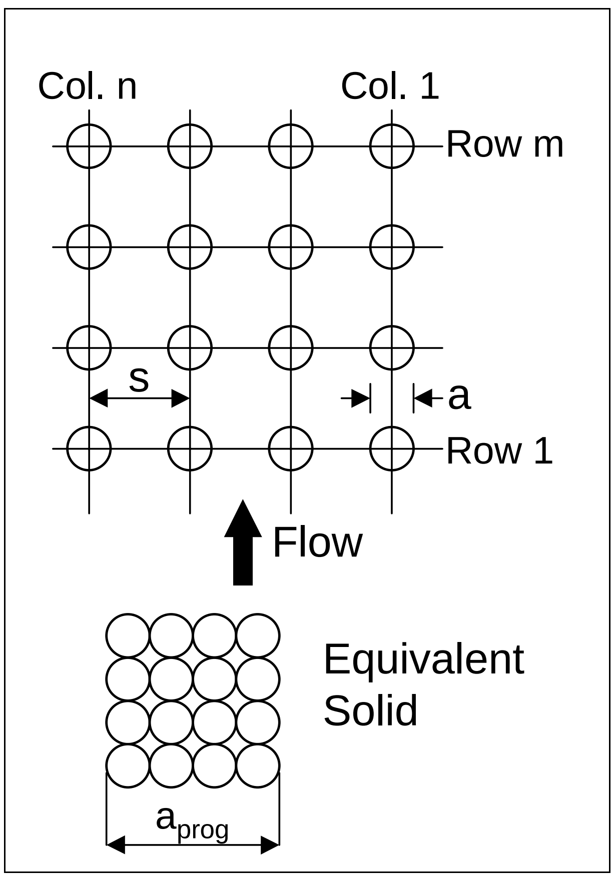


Figure 6.8. Projected width of piles for the special case of aligned flow.

For the general case, Smith[61] determined that a pile group could be represented by an equivalent solid pier that has an effective width, a^*_{pg}, equal to a spacing factor multiplied by the sum of the non-overlapping projected widths of the piles onto a plane normal to the flow direction. The aligned pile group is a special case in which the sum of the non-overlapping projected widths happens to be the same as the width of the collapsed pile group. The procedure for the general case is the same as the procedure for the aligned pile groups except for the determination of the width of the equivalent solid which is a more tedious process for the general case. The sum of the projected widths can be determined by sketching the pile group to scale and projecting the outside edges of each pile onto the projection plane as illustrated in Figure 6.9 or by systematically calculating coordinates of the edges of each pile along the projection plane. The coordinates are sorted in ascending order to facilitate inspection to eliminate double counting of overlapping areas. Additional experiments are being conducted at the FHWA hydraulics laboratory to test simpler techniques for estimating the effective width, but currently Smith's summation technique is a logical choice.

Smith attempted to derive weighting factors to adjust the impact of piles according to their distance from the projection plane, but concluded that there was not enough data and the procedure would become very cumbersome with weighting factors. **A reasonable alternative to using weighting factors is to exclude piles other than the two rows and one column closest to the plane of projection as illustrated by the bold outlines in Figure 6.9**.

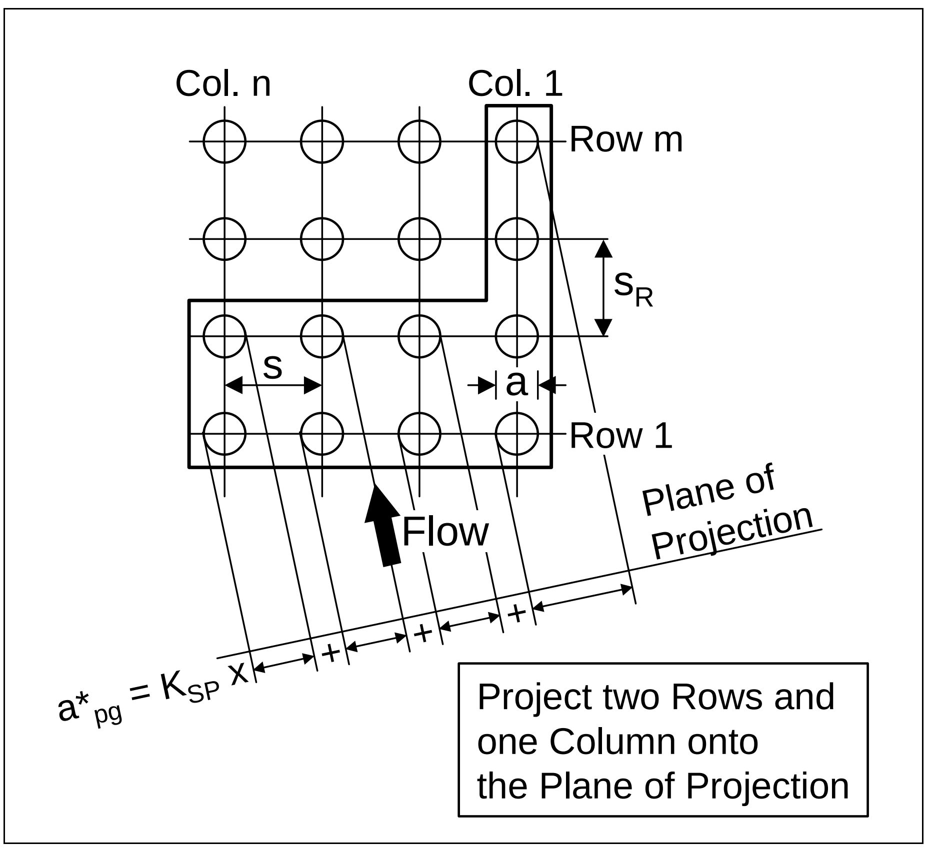


Figure 6.9. Projected width of piles for the general case of skewed flow.

Effective width of an equivalent full depth pier

The effective width of an equivalent full depth pier is the product of the projected width of piles multiplied by a spacing factor and a number of aligned rows factor (used for the special case of aligned piles only).

$$a^*_{pg} = a_{proj} K_{sp} K_m \tag{6.17}$$

where:

a_{proj}	=	Sum of non-overlapping projected widths of piles (see Figures 6.8 and 6.9)
K_{sp}	=	Coefficient for pile spacing (Figure 6.10)
K_m	=	Coefficient for number of aligned rows, m, (Figure 6.11 - note that K_m is constant for all S/a values when there are more than 6 rows of piles)
K_m	=	1.0 for skewed or staggered pile groups

The number of rows factor, K_m, is 1.0 for the general case of skewed or staggered rows of piles because the projection technique for skewed flow accounts for the number of rows and is already conservative for staggered rows.

Adjusted flow depth and velocity

The adjusted flow depth and velocity to be used in the pier scour equation are as follows:

$$y_3 = y_1 + y_{s\ pier}/2 + y_{s\ pc}/2, \text{ m (ft)} \tag{6.18}$$

$$V_3 = V_1 (y_1/y_3), \text{ m/s (ft/s)} \tag{6.19}$$

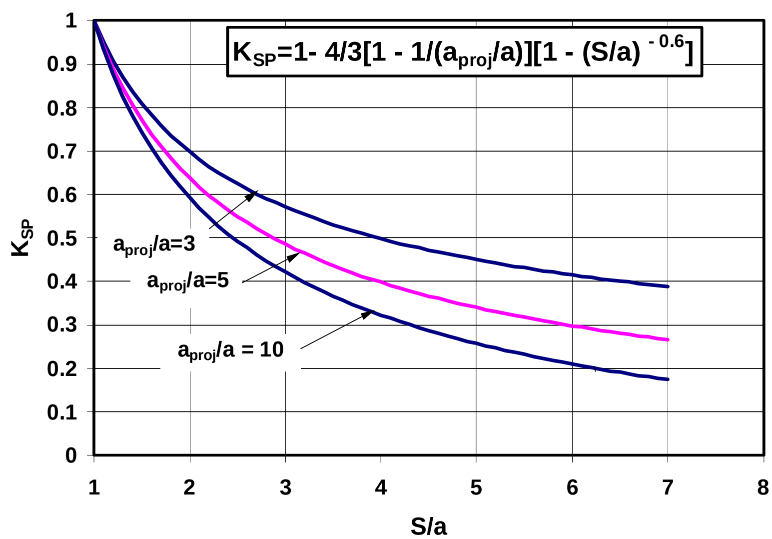


Figure 6.10. Pile spacing factor (refer to Sheppard).[62]

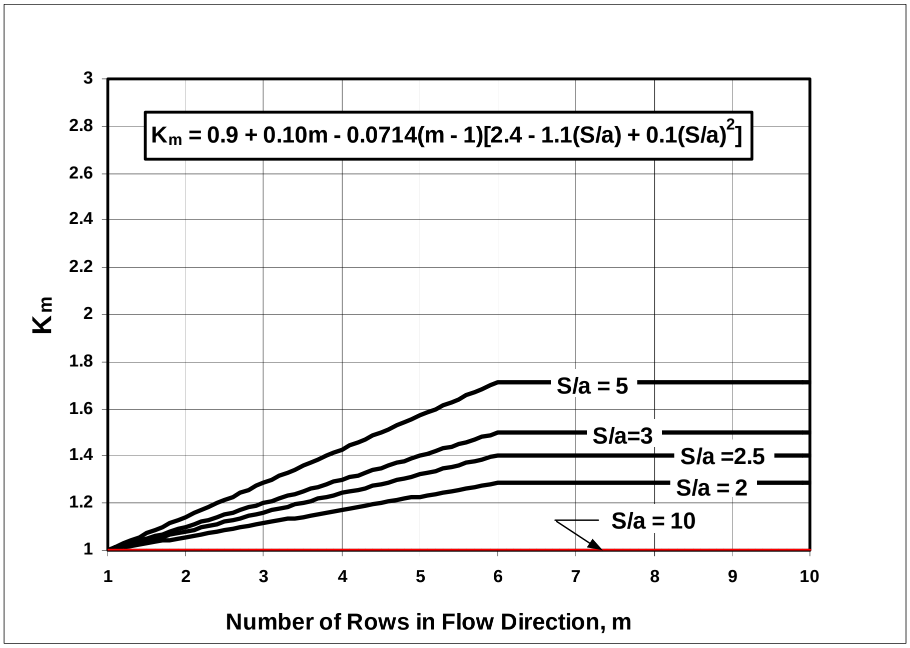


Figure 6.11. Adjustment factor for number of aligned rows of piles (refer to Sheppard).[62]

The scour equation for a pile group can then be written as follows:

$$\frac{Y_{s\,pg}}{y_3} = K_{h\,pg}\left[2.0 K_1 K_3 K_4 \left(\frac{a^*_{pg}}{y_3}\right)^{0.65} \left(\frac{V_3}{\sqrt{g y_3}}\right)^{0.43}\right] \qquad (6.20)$$

where:

$K_{h\,pg}$ = Pile group height factor given in Figure 6.12 as a function of h_3/y_3 (note that the maximum value of $y_3 = 3.5\ a^*_{pg}$)

h_3 = $h_0 + y_{s\,pier}/2 + y_{s\,pc}/2$ = height of pile group above the lowered stream bed after pier and pile cap scour components have been computed, m, (ft)

K_2 from Equation 6.1 has been omitted because pile widths are projected onto a plane that is normal to the flow. The quantity in the square brackets is the scour ratio for a solid pier of width, a^*_{pg}, if it extended to the water surface. This is the scour ratio for a full depth pile group.

6.4.6 Determination of Total Scour Depth for the Complex Pier

The total scour for the complex pier from Equation (6.11) is:

$y_s = y_{s\,pier} + y_{s\,pc} + y_{s\,pg}$

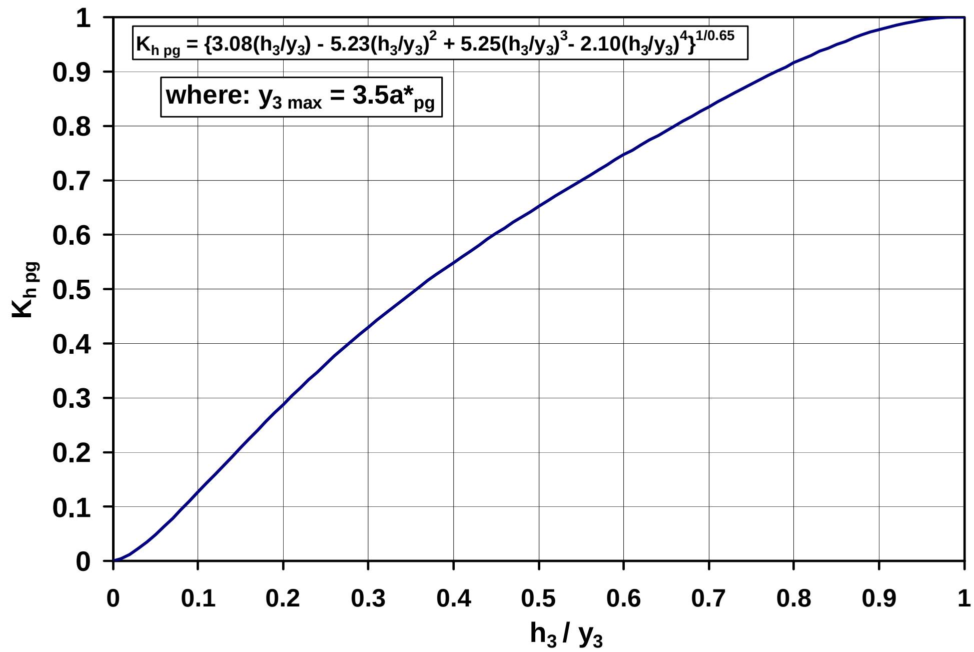


Figure 6.12. Pile group height adjustment factor (refer to Sheppard).[62]

The guidelines described in this section can be used to compute scour for a simple full depth pile group in which case the first two components will be zero and the pile group height factor will be 1.0. Engineering judgment must be used if debris is considered a factor in which case it would be logical to treat the pile group and debris as a vertical extension of the pile cap and to compute scour using the case 2 pile cap procedure described previously.

In cases of complex pile configurations where costs are a major concern, where significant savings are anticipated, and/or for major bridge crossings, physical model studies are still the best guide. Nevertheless, the guidelines described in this section provide a first estimate and a good indication of what can be anticipated from a physical model study.

In many complex piers, the pile groups have a different number of piles in a row or column, the spacing between piles is not uniform, and the widths of the piles may not all be the same. An estimate of the scour depth can be obtained using the methods and equations in this section. However, again it is recommended that a physical model study be conducted to arrive at the final design and to determine the scour depths.

6.5 MULTIPLE COLUMNS SKEWED TO THE FLOW

For multiple columns (illustrated as a group of cylinders in Figure 6.13) skewed to the flow, the scour depth depends on the spacing between the columns. The correction factor for angle of attack would be smaller than for a solid pier. Raudkivi in discussing effects of alignment states "...the use of cylindrical columns would produce a shallower scour; for example, with five-diameter spacing the local scour can be limited to about 1.2 times the local scour at a single cylinder."[26]

In application of Equation 6.1 with multiple columns spaced less than 5 pier diameters apart, the pier width 'a' is the total projected width of all the columns in a single bent, normal to the flow angle of attack (Figure 6.13). For example, three 2.0 m (6.6 ft) cylindrical columns spaced at 10.0 m (33 ft) would have an 'a' value ranging between 2.0 and 6.0 m (6.6 and 33 ft), depending upon the flow angle of attack. **This composite pier width would be used in Equation 6.1 to determine depth of pier scour.** The correction factor K_1 in Equation 6.1 for the multiple column would be 1.0 regardless of column shape. The coefficient K_2 would also be equal to 1.0 since the effect of skew would be accounted for by the projected area of the piers normal to the flow.

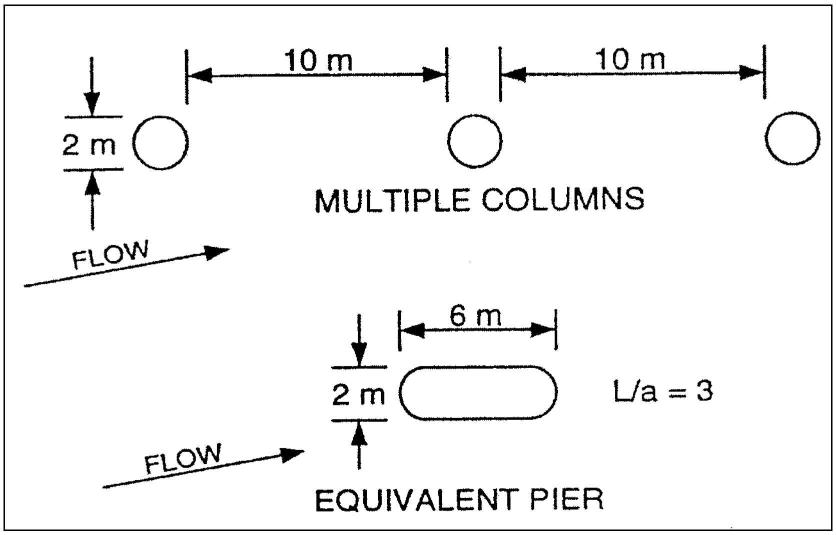


Figure 6.13. Multiple columns skewed to the flow.

The scour depth for multiple columns skewed to the flow can also be determined by determining the K_2 factor using Equation 6.4 and using it in Equation 6.1. The width “a’ in Equation 6.1 would be the width of a single column. An example problem illustrates all three methods of obtaining the scour depth for multiple columns.

If the multiple columns are spaced 5 diameter or greater apart; and debris is not a problem, limit the scour depths to a maximum of 1.2 times the local scour of a single column.

The depth of scour for a multiple column bent will be analyzed in this manner except when addressing the effect of debris lodged between columns. If debris is evaluated, it would be logical to consider the multiple columns and debris as a solid elongated pier. The appropriate L/a value and flow angle of attack would then be used to determine K_2 in Equation 6.4.

Additional laboratory studies are necessary to provide guidance on the limiting flow angles of attack for given distance between multiple columns beyond which multiple columns can be expected to function as solitary members with minimal influence from adjacent columns.

6.6 PRESSURE FLOW SCOUR

Pressure flow, which is also denoted as orifice flow, occurs when the water surface elevation at the upstream face of the bridge is greater than or equal to the low chord of the bridge superstructure (Figure 6.14). Pressure flow under the bridge results from a pile up of water on the upstream bridge face, and a plunging of the flow downward and under the bridge. At higher approach flow depths, the bridge can be entirely submerged with the resulting flow being a complex combination of the plunging flow under the bridge (orifice flow) and flow over the bridge (weir flow).

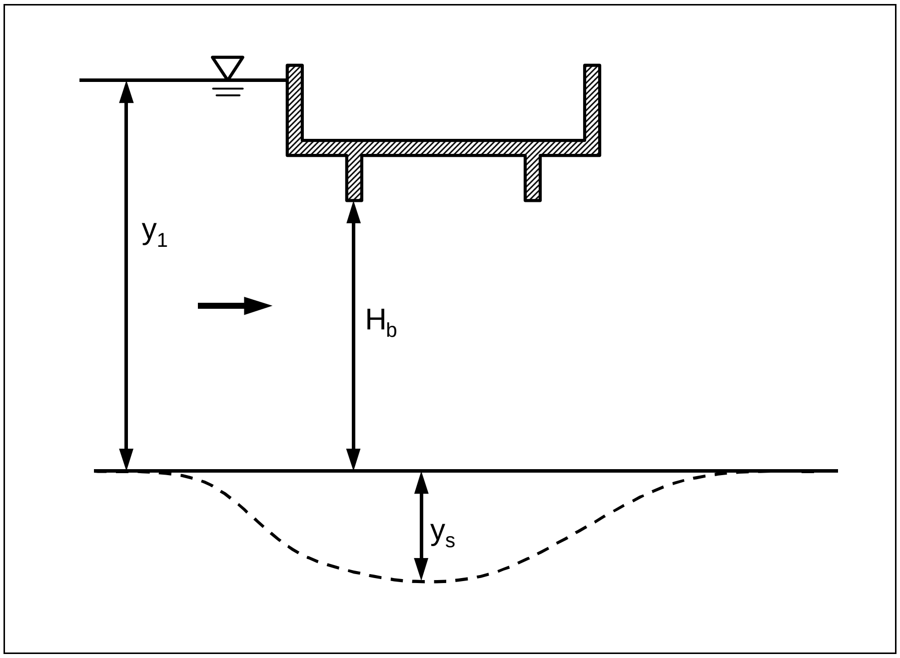


Figure 6.14. Definition sketch of vertical contraction scour resulting from pressure flow.

In many cases, when a bridge is submerged, flow will also overtop adjacent approach embankments. This highway approach overtopping is also weir flow. Hence, for any overtopping situation the total weir flow can be subdivided into weir flow over the bridge and weir flow over the approach. Weir flow over approach embankments serves to reduce the discharge which must pass either under or over the bridge. In some cases, when the approach embankments are lower than the low chord of the bridge, the relief obtained from overtopping of the approach embankments will be sufficient to prevent the bridge from being submerged.

The hydraulic bridge computer models WSPRO or HEC-RAS are suitable for determination of the amount of flow which will flow over the roadway embankment, over the bridge as weir flow, and through the bridge opening as orifice flow, provided that the top of the highway is properly included in the input data.[15, 16, 17] These models can be used to determine average flow depths and velocities over the road and bridge, as well as average velocities under the bridge. **It is recommended that one of these models be used to analyze the scour problem when the bridge is overtopped with or without overtopping of the approach roadway.**

With pressure flow, the local scour depths at a pier or abutment can be much larger than for free surface flow with similar depths and approach velocities. The increase in local scour at a pier subjected to pressure flow results from the flow being directed downward towards the bed by the superstructure (vertical contraction of the flow) and by increasing the intensity of the horseshoe vortex. The vertical contraction of the flow can be a more significant cause of the increased scour depth. However, in many cases, when a bridge becomes submerged, the average velocity under the bridge is reduced due to a combination of additional backwater caused by the bridge superstructure impeding the flow, and a reduction of the discharge which must pass under the bridge due to weir flow over the bridge and/or approach embankments. **As a consequence of this, increases in local scour attributed to pressure flow scour at a particular site, may be offset to a degree by lower velocities through the bridge opening due to increased backwater and a reduction in discharge under the bridge due to overtopping of the bridge and approach embankments.**

Limited studies of pressure flow scour have been made in flumes at Colorado State University and FHWA's Turner Fairbank Highway Research Center which indicate that pier scour can be increased 200 to 300 percent by pressure flow.[63, 64, 65] Both studies were for clear-water scour (no transport of bed material upstream of the bridge). Arneson[66] conducted a more extensive study of pressure flow scour under live bed conditions. FHWA's Turner Fairbank Laboratory and Arneson’s study concluded that (1) pressure flow scour is a combination of vertical contraction scour and local pier scour, (2) the local pier scour component was approximately the same as the free-surface local pier scour measurements for the same approach flow condition, and 3) the two components were additive. Arneson's equation, derived from multiple linear regression of his data, for bed vertical contraction scour is:

$$\frac{y_s}{y_1} = -5.08 + 127\left(\frac{y_1}{H_b}\right) + 4.44\left(\frac{H_b}{y_1}\right) + 0.19\left(\frac{V_a}{V_c}\right) \qquad (6.21)$$

where:

- y_s = Depth of vertical contraction scour relative to mean bed elevation, m (ft)
- y_1 = Depth of flow immediately upstream of the bridge, m (ft)
- H_b = Distance from the low chord of the bridge to the average elevation of the stream bed before scour, m (ft)
- V_a = Average velocity of the flow through the bridge opening before scour occurs, m/s (ft/s)
- V_c = Critical velocity of the D_{50} of the bed material in the bridge opening, m/s (ft/s)

The procedure for calculating pier scour for pressure flow is as follows:

a. Determine the flow variables using a 1-dimensional or 2-dimensional computer model such as WSPRO, HEC-RAS, FESWMS, or RMA-2.

b. Calculate the critical velocity V_c of the D_{50} of the bed material in the bridge opening.

c. Use the flow variables and critical velocity to compute the vertical contraction scour (Equation 6.21).

d. Use the flow variables to compute the local pier scour using Equations 6.1 or 6.3 and the other procedures presented in previous sections.

e. Add the scour components obtained in c and d to obtain the local pier scour for pressure flow.

f. Use engineering judgment to evaluate the local pressure flow pier scour .

6.7 SCOUR FROM DEBRIS ON PIERS

Debris lodged on a pier can increase local scour at a pier. The debris may increase pier width and deflect a component of flow downward. This increases the transport of sediment out of the scour hole. When floating debris is lodged on the pier, the scour depth can be estimated by assuming that the pier width is larger than the actual width. The problem is in determining the increase in pier width to use in the pier scour equation. Furthermore, at large depths, the effect of the debris on scour depth should diminish (for additional discussion, see HEC-20[(6)]).

As with estimating local scour depths with pressure flow, only limited research has been done on local scour with debris. Melville and Dongol have conducted a limited quantitative study of the effect of debris on local pier scour and have made some recommendations which support the approach suggested above.[(67)] However, additional laboratory studies will be necessary to better define the influence of debris on local scour.

An interim procedure for estimating the effect of debris on local scour at piers is presented in Appendix D.

6.8 TOPWIDTH OF SCOUR HOLES

The topwidth of a scour hole in cohesionless bed material from one side of a pier or footing can be estimated from the following equation:(68)

$$W = y_s \ (K + \mathrm{Cot}\ \theta) \tag{6.22}$$

where:

W	=	Topwidth of the scour hole from each side of the pier or footing, m
y_s	=	Scour depth, m (ft)
K	=	Bottom width of the scour hole related to the of scour depth
θ	=	Angle of repose of the bed material ranging from about 30° to 44°

The angle of repose of cohesionless material in air ranges from about 30° to 44°. Therefore, if the bottom width of the scour hole is equal to the depth of scour y_s (K = 1), the topwidth in cohesionless sand would vary from 2.07 to 2.80 y_s. At the other extreme, if K = 0, the topwidth would vary from 1.07 to 1.8 y_s. Thus, the topwidth could range from 1.0 to 2.8 y_s and depends on the bottom width of the scour hole and composition of the bed material. In general, the deeper the scour hole, the smaller the bottom width. In water, the angle of repose of cohesionless material is less than the values given for air; therefore, a topwidth of 2.0 y_s is suggested for practical applications (Figure 6.15).

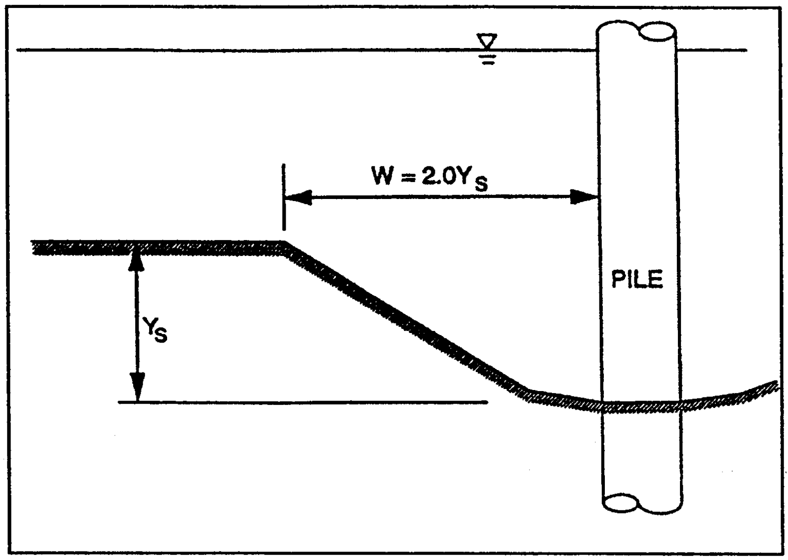


Figure 6.15. Topwidth of scour hole.

6.9 PHYSICAL MODEL STUDIES

For unusual or complex pier foundation configurations a physical model study should be made. The scale between model and prototype is based on the Froude criteria, that is, the Froude number for the model should be the same as for the prototype. In general it is not possible to scale the bed material size. Also, at flood flows in sand bed streams the sediment transport conditions will be live-bed and the bed configuration will be plane bed. Whereas, in the model live-bed transport conditions will be ripples or dunes. These are incomparable pier scour conditions. Therefore, it is recommended that a bed material be used that has a critical velocity just below the model velocity (i.e., clear-water scour conditions). This will usually give the maximum scour depth; but a careful study of the results needs to be made by persons with field and model scour experience. For additional discussion of the use of physical modeling in hydraulic design, see HEC-23.[7]

6.10 PIER SCOUR EXAMPLE PROBLEMS (SI)

6.10.1 Example Problem 1 - Scour at a Simple Solid Pier (SI)

Given:

Pier geometry: a = 1.22 m , L = 18 m, round nose
Flow variables: y_1 = 3.12 m, V_1 =3.36 m/s
Angle of attack = 0 degrees, g = 9.81 m/s^2
Froude No. = 3.36/(9.81 x 3.12)$^{0.5}$ = 0.61
Bed material: D_{50} = 0.32 mm, D_{95} = 7.3 mm
Bed Configuration: Plane bed.

Determine:

The magnitude of pier scour depth.

Solution:

Use Equation 6.1.

$$\frac{y_s}{y_1} = 2.0\ K_1\ K_2\ K_3\ K_4 \left(\frac{a}{y_1}\right)^{0.65} Fr_1^{0.43}$$

$$y_S\ /\ 3.12 = 2.0 \times 1.0 \times 1.0 \times 1.1 \times 1.0 \times (1.22\ /\ 3.12)^{0.65} \times 0.61^{0.43} = 0.97$$

$$y_s = 0.97 \times 3.12 = 3.03\,\text{m}$$

6.10.2 Example Problem 2 - Angle of Attack (SI)

Given:

Same as Problem 1 but angle of attack is 20 degrees

Solution:

Use Equation 6.4 to compute K_2

$$K_2 = (\text{Cos}\theta + L\ /\ a\ \text{Sin}\theta)^{0.65}$$

If L/a is larger than 12, use L/a = 12 as a maximum in Equation 6.4 (see Table 6.2).

L/a = 18 /1.22 = 14.8 > 12 use 12

K_2 = (Cos 20 + 12 Sin 20)$^{0.65}$ = 2.86

y_s = 3.03 X 2.86 = 8.7 m

6.10.3 Example Problem 3 - Coarse Bed Material (SI)

Given:

Same as Problem 1 but the bed material is coarser
Bed material: D_{50} = 17.8 mm, D_{95} = 96.3 mm
Bed configuration: Plane Bed

Determine:

If the coarse bed material would decrease local scour depth. Determine K_4 and y_s.

Solution:

Use Equations 6.5, 6.6, 6.7, and 6.8

K_4 = 1 if D_{50} <2 mm or D_{95} < 20 mm

If $D_{50} \geq 2$ mm and $D_{95} \geq 20$ mm

then:

$$K_4 = 0.4\,(V_R)^{0.15}$$

$$V_R = \frac{V_1 - V_{icD_{50}}}{V_{cD_{50}} - V_{icD_{95}}} > 0$$

where:

V_{icDx} = Approach velocity required to initiate scour at the pier for the grain size D_x, m/s

V_{icDx} = $0.645\left(\frac{D_x}{a}\right)^{0.053} V_{cD_x}$

V_{cDX} = Critical velocity for incipient motion for the grain size D_x ,m/s

V_{cDX} = $6.19\ y_1^{1/6}\ D_x^{1/3}$

V_{cD50} = $6.19\ (3.12)^{1/6}\ (0.0178)^{1/3}$ = 1.95 m/s

V_{cD95} = $6.19\ (3.12)^{1/6}\ (0.0963)^{1/3}$ = 3.43 m/s

V_{icD50} = $0.645\ (0.0178\ /\ 1.22)^{0.053}$ (1.95) = 1.01 m/s

V_{icD95} = $0.645\ (0.0963\ /\ 1.22)^{0.053}$ (3.43) = 1.93 m/s

V_R = $\frac{(3.36 - 1.01)}{(1.95 - 1.93)} = 117.5$

K_4 = $0.4\ (117.5)^{0.15}$ = 0.82

y_s = 0.82 X 3.03 = 2.48 m

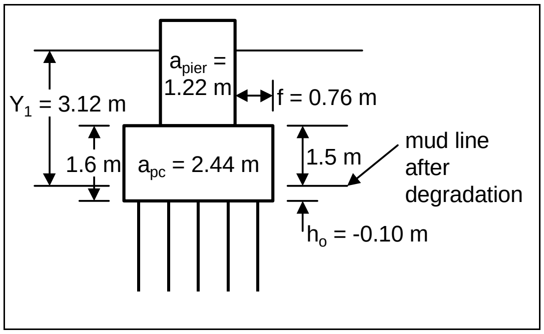
6.10.4 Example Problem 4 - Scour at Complex Piers (Solid Pier on an Exposed Footing)(SI)

Given:

The pier in Problem 1 (Section 6.10.1) is on a 2.44 m wide by 1.60 m high by 19.81 m long rectangular footing. Footing extends 0.76 m upstream from the pier stem. The footing is on an unspecified pile foundation. The footing is exposed 1.50 m by long-term degradation. Determine the local scour.

Pier geometry: a_{pier} = 1.22 m, L= 18 m, round nose
Pile cap or footing geometry: a_{pc} (or a_f) = 2.44 m, L = 19.81 m, T = 1.60 m, f = 0.76 m
Approach flow: y_1 = 3.12 m, V_1 = 3.36 m/s
Angle of attack: 0 degrees

Froude No. = $3.36/(9.81x3.12)^{0.5} = 0.61$
Bed material: D_{50} = 0.32 mm, D_{84} = 7.3 mm, plane bed
See sketch below:



Local Scour from Pier Stem

f = 0.76 m
$h_1 = h_0 + T = -0.10 + 1.60 = 1.50$ m
$K_{h\,pier}$ = function (h_1/a_{pier}, f/a_{pier}) (from Figure 6.5)
$h_1/a_{pier} = 1.5/1.22 = 1.23$
$f/a_{pier} = 0.76/1.22 = 0.62$
$K_{h\,pier} = 0.06$

$$\frac{y_{s\,pier}}{y_1} = K_{h\,pier}\left[2.0K_1K_2K_3K_4\left(\frac{a_{pier}}{y_1}\right)^{0.65}\left(\frac{V_1}{\sqrt{g\,y_1}}\right)^{0.43}\right]$$

$$\frac{y_{s\,pier}}{y_1} = 0.06\left[2.0(1.0)(1.0)(1.1)(1.0)\left(\frac{1.22}{3.12}\right)^{0.65}\left(\frac{3.36}{\sqrt{9.81x3.12}}\right)^{0.43}\right]$$

$y_{s\,pier}$ =0.06x[0.97]x3.12 = 0.18 m

Note: the quantity in the square brackets is the scour ratio for a full depth pier.

Local Scour from the Pile Cap or Footing

Assume the average bed elevation in the vicinity of the pier lowers by ½ the pier stem scour.

$y_2 = y_1 + y_{s\,pier}/2 = 3.12 + 0.18/2 = 3.21$ m

$V_2 = V_1(y_1/y_2) = 3.36\ (3.12/3.21) = 3.26$ m/s

$h_2 = h_0 + y_{s\,pier}/2 = -0.10 + 0.09 = -0.01$

The bottom of the pile cap is below the adjusted mud line; use Case 2 computations for an exposed footing.

$$y_f = h_1 + y_{s\,pier}/2 = 1.50 + 0.09 = 1.59 \text{ m}$$

The velocity on the footing is:

$$\frac{V_f}{V_2} = \frac{\ln\left(10.93\dfrac{y_f}{k_s}+1\right)}{\ln\left(10.93\dfrac{y_2}{k_s}+1\right)} = \frac{\ln\left(10.93\dfrac{1.59}{0.0073}+1\right)}{\ln\left(10.93\dfrac{3.21}{0.0073}+1\right)} = 0.92$$

Note: Assume $K_s = D_{84} = 7.3$ mm

$$V_f = 0.92 x V_2 = 0.92 \text{ x } 3.26 = 2.99 \text{ m/s}$$

$$\frac{y_{s\,footing}}{y_f} = 2.0 K_1 K_2 K_3 K_4 K_w \left(\frac{a_f}{y_f}\right)^{0.65} \left(\frac{V_f}{\sqrt{g y_f}}\right)^{0.43}$$

$$\frac{y_{s\,footing}}{y_f} = 2.0(1.1)(1.0)(1.1)(1.0)(1.0)\left(\frac{2.44}{1.59}\right)^{0.65}\left(\frac{2.99}{\sqrt{9.81 \text{x} 1.59}}\right)^{0.43} = 2.83$$

Note that $y_2/a_f = 1.31$ (>0.8); use $K_W = 1.0$

$$y_{s\,footing} = 2.83 y_f = 2.83 \text{ x } 1.59 = 4.50 \text{ m}$$

Total Local Pier Scour Depth

$$y_s = y_{s\,pier} + y_{s\,footing} = 0.18 + 4.50 = 4.68 \text{ m}$$

6.10.5 Example Problem 5 - Scour at a Complex Pier with Pile Cap in the Flow (SI)

During the design of the new Woodrow Wilson Bridge over the Potomac River several complex pier configurations were tested in physical model studies. The purpose of this problem is to analyze local scour for the possible condition that the main channel migrated to the pier configured as shown in Figure 6.16. It was determined that the water surface elevations would be +2.23 m and +2.96 m for the Q_{100} and the Q_{500} events respectively and the velocities in the main channel would be 3.41 m/sec and 4.27 m/sec for the Q_{100} and the Q_{500} events respectively. The following computations are for the Q_{100} event:

Initial parameters

y_1 = 15.79 m
V_1 = 3.41 m/sec
a_{pier} = 9.754 m
a_{pc} = 16.23 m

h_0 = 7.77 m
$h_1 = h_0 + T$ = 12.65 m (resolution of the pile cap thickness below)
S = 4.19 m (center to center spacing of piles)
T = 4.88 m (assign half of the tapered portion of the cap to the pile cap and half to the pier)
f = 2.627 m (Figure 6.16)
zero angle of attack

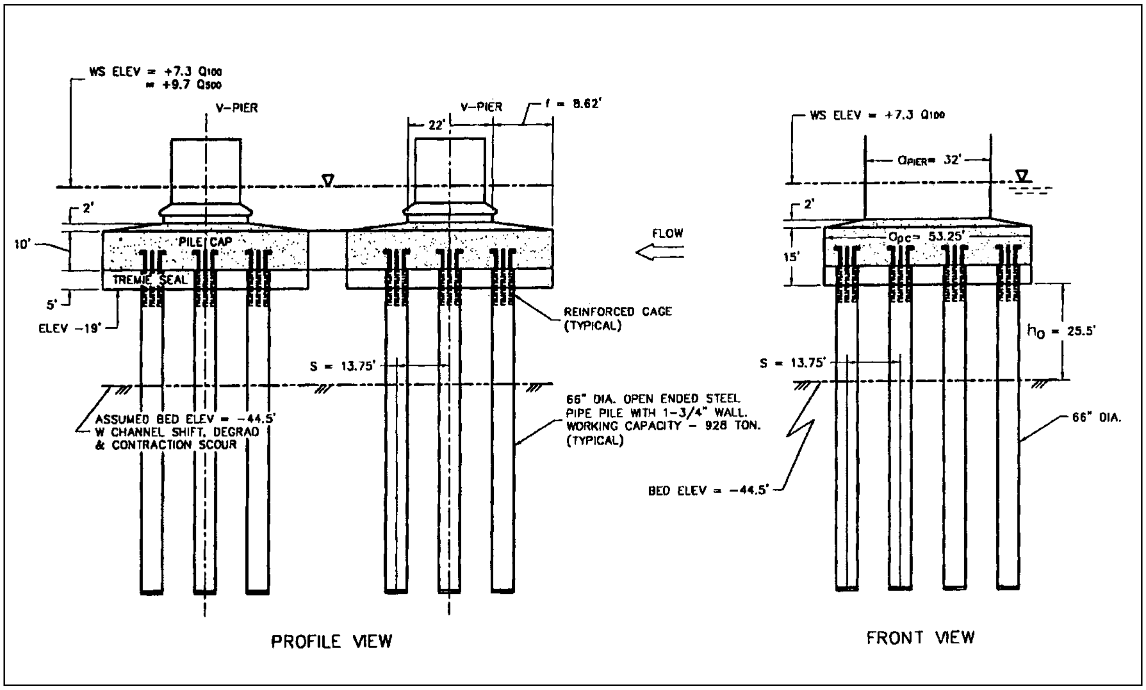


Figure 6.16. Model of complex pier geometry for the Woodrow Wilson Bridge.

Pier Stem Component

f/a_{pier} = 2.627/9.754 = 0.27

h_1/a_{pier} = 12.65/9.754 = 1.30

$K_{h\,pier}$ = 0.062 (from Figure 6.5)

$$\frac{y_{s\,pier}}{y_1} = K_{h\,pier}\left[2.0 K_1 K_2 K_3 K_4 \left(\frac{a_{pier}}{y_1}\right)^{0.65}\left(\frac{V_1}{\sqrt{g y_1}}\right)\right]^{0.43}$$

$$\frac{y_{spier}}{15.79} = 0.062\left[2.0(1.1)(1.0)(1.1)(1.0)\left(\frac{9.754}{15.79}\right)^{0.65}\left(\frac{3.41}{\sqrt{(9.81)15.79}}\right)^{0.43}\right] = 0.0627$$

The quantity in the brackets is the scour ratio for a full depth pier that extends below the scour hole.

$y_{s\ pier} = 0.0627 \times 15.79 \text{ m} = 0.99 \text{ m}$

Pile Cap Component

$h_2 = h_0 + y_{s\ pier}/2 = 7.77 + 0.495 = 8.27 \text{ m}$

$y_2 = y_1 + y_{s\ pier}/2 = 15.79 + 0.495 = 16.28 \text{ m}$

$V_2 = V_1 \times (y_1/y_2) = 3.41 \times (15.79/16.28) = 3.31 \text{ m/s}$

Note: For Figure 6.6, $y_2 = 3.5a_{pc} = 56.81 > 16.28$; use $y_2 = 16.28$ m

$h_2/y_2 = 0.51$

$T/y_2 = 4.88/16.28 = 0.30$

$\frac{a^*_{pc}}{a_{pc}} = 0.07$ (from Figure 6.6)

$a^*_{pc} = 0.07 \times 16.23 = 1.10 \text{ m}$

This is the width of a full depth pier that would produce the same scour depth as the isolated pile cap will produce.

$$\frac{y_{s\,pc}}{y_2} = 2.0 K_1 K_2 K_3 K_4 K_w \left(\frac{a^*_{pc}}{y_2}\right)^{0.65} \left(\frac{V_2}{\sqrt{gy_2}}\right)^{0.43}$$

$$\frac{y_{s\,pc}}{16.28} = 2.0(1.1)(1.0)(1.1)(1.0)(1.0)\left(\frac{1.10}{16.28}\right)^{0.65}\left(\frac{3.31}{\sqrt{(9.81)(16.28)}}\right)^{0.43} = 0.236$$

Note that $y_2/a^*_{pc} = 14.8$ (>0.8); use $K_w = 1.0$

$y_{s\ pc} = 0.236 \times 16.28 = 3.84 \text{ m}$

Pile Group Component

$h_3 = h_0 + (y_{s\ pier} + y_{s\ pc})/2 = 7.77 + (0.99 + 3.84)/2 = 10.19 \text{ m}$

$y_3 = y_1 + (y_{s\ pier} + y_{s\ pc})/2 = 15.79 + (0.99 + 3.84)/2 = 18.20 \text{ m}$

$V_3 = V_1 \times (y_1/y_3) = 3.41 \times (15.79/18.20) = 2.95 \text{ m/sec}$

$a_{proj} = 4 \times 1.676 = 6.71$ m (from Figure 6.8)

$a_{proj}/a = 6.71 / 1.676 = 4.0$

$S/a = 4.19/1.676 = 2.5$ (relative center to center spacing of piles)

$K_{sp} = 0.58$ (from Figure 6.10)

$K_m = 1.16$ (From Figure 6.11 for three rows per foundation; foundations separated)

$a^*_{pg} = K_{sp} \times K_m \times a_{proj} = 0.58 \times 1.16 \times 6.71 = 4.51$ m

Note: for Figure 6.12, $y_{3\,max} = 3.5 \times a^*_{pg} = 15.79 < 18.20$; use $y_3 = 15.79$ m

$h_3/y_3 = 10.19 / 15.79 = 0.65$

$K_{h\ pg} = 0.79$ (from Figure 6.12)

$$\frac{y_{s\,pg}}{y_3} = K_{h\,pg}\left[2.0 K_1 K_2 K_3 K_4 \left(\frac{a^*_{pg}}{y_3}\right)^{0.65}\left(\frac{V_3}{\sqrt{g y_3}}\right)^{0.43}\right]$$

$$\frac{y_{s\,pg}}{15.79} = 0.79\left[2.0(1.0)(1.0)(1.1)(1.0)\left(\frac{4.51}{15.79}\right)^{0.65}\left(\frac{2.95}{\sqrt{(9.81)(15.79)}}\right)^{0.43}\right] = 0.41$$

$y_{s\ pg} = 0.41 \times 15.79 = 6.47$ m

Total Estimated Scour

$y_s = y_{s\ pier} + y_{s\ pc} + y_{s\ pg} = 0.99 + 3.84 + 6.47 = 11.3$ m

6.10.6 Example Problem 6 - Scour at Multiple Columns (SI)

Calculate the scour depth for a pier that consists of six 0.406 m columns spaced at 2.29 m with a flow angle of attack of 26 degrees. Debris is not a problem and there is no armoring at this site.

Data:

Columns: 6 columns 0.406 m, spaced 2.29 m
Velocity: $V_1 = 3.4$ m/s; Depth: $y_1 = 6.1$ m
Angle of attack: 26 degrees
Spacing coefficient = $S/a = 2.29/0.406 = 5.6$; $S/a > 5.0$
Assume $K_3 = 1.1$ for plane bed condition

Determine the depth of local scour:

Three methods of calculating the scour depth will be illustrated:

a. Scour depth according to Raudkivi[26] is 1.2 times the local scour of a single column.

$$\frac{y_s}{6.1} = 2.0 \times 1.0 \times 1.0 \times 1.1 \times 1.0 \left(\frac{0.406}{6.1}\right)^{0.65} \left(\frac{3.4}{(9.81 \times 6.1)^{0.5}}\right)^{0.43} = 0.266$$

$y_s = 6.1 \text{ X } 0.266 \text{ X } 1.2 = 1.95$ m

b. Compare this value with that computed by collapsing the columns.

Collapsed pier width = 6 X 0.406 = 2.44 m

Projected pier width = L Sin 26^0 + a Cos 26^0 = 2.44 Sin 26^0 + .406 Cos 26^0 = 1.44 m

$$\frac{y_s}{6.1} = 2.0 (1.0)(1.0)(1.1) 1.0 \left(\frac{1.44}{6.1}\right)^{0.65} \left(\frac{3.4}{(9.81 \times 6.1)^{0.5}}\right)^{0.43} = 0.604$$

$y_s = 3.68$ m

c. The scour depth can be calculated for multiple columns by calculating the depth for a single column and multiplying it by the K_2 factor given in Equation 6.4. For example:

K_2 = (Cos 26^0 + 2.44/0.406 Sin 26^0 $)^{0.65}$ = 2.27

$$\frac{y_s}{6.1} = 2.0 (1.0)(2.27)(1.1)(1.0) \left(\frac{0.406}{6.1}\right)^{0.65} \left(\frac{3.4}{(9.81 \times 6.1)^{0.5}}\right)^{0.43} = 0.603$$

$y_s = 6.1 \text{ X } 0.603 = 3.68$ m

Spacing between columns for this pier is greater than 5 times column diameter so method (a) applies. Also, a model study of the pier gave a scour depth of 1.95 m. Therefore:

$y_s = 6.1 \text{ X } 0.266 \text{ X } 1.2 = 1.95$ m

6.10.7 Example Problem 7 - Pier Scour with Pressure Flow (SI)

An existing bridge is subjected to pressure flow to the top of a solid guard rail at the 100-year return period flow. There is only a small increase in flow depth at the bridge for the 500-year return period flow due to the large overbank area. A HEC-RAS model of the flow gives the following data:

Data:

y_1 = 9.75 m, V_1 = 2.93 m/s, q_1 = 28.56 cms/m
Pier width a = 0.914 m, is round nose, solid, aligned with the flow
Sand bed with D_{50} = 0.4 mm and D_{84} = 0.9 mm
Distance from stream bed to lower chord (H_b) is 7.93 m before scour

Calculate the local pier scour:

Vertical Contraction Scour Depth

$y_s/y_1 = -5.08 + 1.27\ y_1/H_b + 4.44\ H_b/y_1 + 0.19\ V_a/V_c$

$V_c = 6.19\ (y_1)^{1/6}\ (D_{50})^{1/3} = 6.19\ (9.75)^{1/6}\ (0.0004)^{1/3} = 0.669$ m/s

$V_a = q_1/H_b = 28.56/7.93 = 3.60$ m/s

$y_s/9.75 = -5.08 + 1.27\ (9.75/7.93) + 4.44\ (7.93/9.75) + 0.19\ (3.60/0.669)$

$y_s/9.75 = 1.12$ and $y_s = 10.9$ m

Local Pier Scour

$y_2 = H_b + y_s = 7.93 + 10.92 = 18.85$ m

$V_2 = V_a\ (H_b/y_2) = 3.60\ (7.93/18.85) = 1.51$ m/s

$y_s/y_1 = 2.0\ K_1\ K_2\ K_3\ K_4\ (a/y_1)^{0.65}\ (Fr)^{0.43}$

$K_1 = K_2 = K_4 = 1.0$; $K_3 = 1.1$; $Fr = 1.52 / (9.81\ X 18.85)^{0.5} = 0.11$

$y_s/18.85 = 2.0\ X\ 1.1\ X\ (0.914/18.85)^{0.65}\ (0.11)^{0.43} = 0.12$

$y_s = 18.85\ X\ 0.12 = 2.26$ m

Total Scour

$y_s = 10.92 + 2.26 = 13.2$ m

6.11 PIER SCOUR EXAMPLE PROBLEMS (ENGLISH)

6.11.1 Example Problem 1 - Scour at a Simple Solid Pier (English)

Given:

Pier geometry: a = 4.0 ft, L = 59 ft, round nose
Flow variables: $y_1 = 10.2$ ft, $V_1 = 11.02$ ft/s
Angle of attack = 0 degrees, g = 32.2 ft/s^2
Froude No. = $11.02/(32.2 \times 10.2)^{0.5} = 0.61$
Bed material: $D_{50} = 0.32$ mm (0.0011 ft), $D_{95} = 7.3$ mm (0.024 ft)
Bed Configuration: Plane bed

Determine:

The magnitude of pier scour depth.

Solution:

Use Equation 6.1.

$$\frac{y_s}{y_1} = 2.0 K_1 K_2 K_3 K_4 \left(\frac{a}{y_1}\right)^{0.65} Fr_1^{0.43}$$

$$y_s / 10.2 = 2.0 x 1.0 x 1.0 x 1.1 x 1.0 x (4.0 / 10.2)^{0.65} x 0.61^{0.43} = 0.97$$

$$y_s = 0.97 x 10.22 = 9.9\,ft$$

6.11.2 Example Problem 2 - Angle of Attack (English)

Given:

Same as Problem 1 but angle of attack is 20 degrees

Solution:

Use Equation 6.4 to compute K_2

$$K_2 = (Cos\ \theta + L / a\ Sin\ \theta)^{0.65}$$

If L/a is larger than 12, use L/a = 12 as a maximum in Equation 6.4 (see Table 6.2).

L/a = 18 /1.22 = 14.8 > 12 use 12

$K_2 = (Cos\ 20 + 12\ Sin\ 20)^{0.65} = 2.86$

y_s = 9.9 X 2.86 = 28.4 ft

6.11.3 Example Problem 3 - Coarse Bed Material (English)

Given:

Same as Problem 1 but the bed material is coarser
Bed material: D_{50} = 17.8 mm, (0.058 ft); D_{95} = 96.3 mm, (0.316 ft)
Bed configuration: Plane Bed

Determine:

If the coarse bed material would decrease local scour depth. Determine K_4 and y_s.

Solution:

Use Equations 6.5, 6.6, 6.7, and 6.8
K_4 = 1 if D_{50} <2 mm or D_{95} < 20 mm
if $D_{50} \geq 2$ mm and $D_{95} \geq 20$ mm

then:

$$K_4 = 0.4\,(V_R)^{0.15}$$

$$V_R = \frac{V_1 - V_{icD_{50}}}{V_{cD_{50}} - V_{icD_{95}}} > 0$$

where:

V_{icDx}	=	Approach velocity required to initiate scour at the pier for the grain size Dx , ft/s
V_{icDx}	=	$0.645 \left(\frac{D_x}{a}\right)^{0.053} V_{cD_x}$
V_{cDx}	=	Critical velocity for incipient motion for the grain size D_x ,ft/s
V_{cDx}	=	$112\ y_1^{1/6}\ D_x^{1/3}$
V_{cD50}	=	$11.2\ (10.2)^{1/6}\ (0.058)^{1/3} = 6.38$ ft/s
V_{cD95}	=	$11.2\ (10.2)^{1/6}\ (0.316)^{1/3} = 11.23$ ft/s
V_{icD50}	=	$0.645\ (0.058 / 4.0)^{0.053}\ (6.38) = 3.29$ ft/s
V_{icD95}	=	$0.645\ (0.316 / 4.0)^{0.053}\ (11.23) = 6.33$ ft/s)
V_R	=	$\frac{(1102 - 3.29)}{(6.38 - 6.3)} = 154.6$
K_4	=	$0.4\ (154.6)^{0.15} = 0.85$
y_s	=	0.85 X 9.9 = 8.4 ft

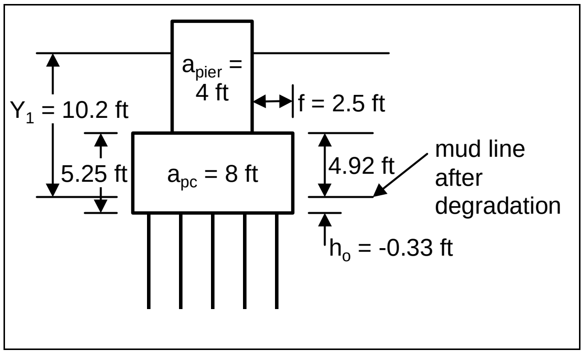
6.11.4 Example Problem 4 - Scour at Complex Piers (Solid Pier on an Exposed Footing) (English)

Given:

The pier in Problem 1 (Section 6.11.1) is on a 8.0 ft wide by 5.25 ft high by 65 ft long rectangular footing. Footing extends 2.5 ft upstream from the pier. The footing is on an unspecified pile foundation. The footing is exposed 4.92 ft by long-term degradation. Determine local pier scour.

Data:

Pier geometry; a_{pier} = 4.0 ft, L = 59 ft, round nose
Pile cap or footing geometry, a_{pc} (or a_f) = 8 ft, L = 65 ft, T = 5.12 ft, f = 2.5 ft
Approach flow: y_1 = 10.2 ft, V_1 = 11.02 ft/s
Angle of attack = 0 degrees
Froude No. = $11.02/(32.2 \times 10.2)^{0.5} = 0.61$
Bed material: D_{50} = 0.32 mm, D_{84} = 7.3 mm, Plane bed
h_0 = 4.92 - 5.25 = -0.33 ft
See sketch below:



Local Scour from Pier Stem

$f = 2.5$ ft
$h_1 = h_0 + T = -0.33 + 5.25 = 4.92$ ft
$K_{h\ pier}$ = function (h_1/a_{pier}, f/a_{pier}) (from Figure 6.5)
$h_1/a_{pier} = 4.92/4.0 = 1.23$
$f/a_{pier} = 2.5/4 = 0.62$
$K_{h\ pier} = 0.06$

$$\frac{y_{s\,pier}}{y_1} = K_{h\,pier}\left[2.0\,K_1K_2K_3K_4\left(\frac{a_{pier}}{y_1}\right)^{0.65}\left(\frac{V_1}{\sqrt{gy_1}}\right)^{0.43}\right]$$

$$\frac{y_{s\,pier}}{y_1} = 0.06\left[2.0(1.0)(1.0)(1.1)(1.0)\left(\frac{4.0}{10.2}\right)^{0.65}\left(\frac{11.02}{\sqrt{32.2 \times 10.2}}\right)^{0.43}\right]$$

$y_{s\ pier} = 0.06 \times [0.97] \times 10.2 = 0.6$ ft

Note: the quantity in the square brackets is the scour ratio for a full depth pier.

Local Scour from the Pile Cap or Footing

Assume the average bed elevation in the vicinity of the pier lowers by ½ the pier stem scour.

$y_2 = y_1 + y_{s\ pier}/2 = 10.2 + 0.3 = 10.5$ ft

$V_2 = V_1(y_1/y_2) = 11.02\ (10.2/10.5) = 10.7$ ft/s

$h_2 = h_0 + y_{s\ pier}/2 = -0.33 + 0.3 = -0.03$ ft

The bottom of the pile cap is below the adjusted mud line; use Case 2 computations for an exposed footing.

$y_f = h_1 + y_{s\ pier}/2 = 4.92 + 0.3 = 5.22$ ft

The velocity on the footing is:

$$\frac{V_f}{V_2} = \frac{\ln\left(10.93\,\frac{y_f}{k_s}+1\right)}{\ln\left(10.93\,\frac{y_2}{k_s}+1\right)} \quad = \quad \frac{\ln\left(10.93\,\frac{5.22}{.024}+1\right)}{\ln\left(10.93\,\frac{10.5}{0.024}+1\right)} = 0.92$$

Note: assume $k_s = D_{84} = 7.3$ mm = 0.024 ft

$V_f = 0.92xV_2 = 0.92 \times 10.7 = 9.84$ ft/s

$$\frac{y_{s\,footing}}{y_f} = 2.0\,K_1K_2K_3K_4K_w\left(\frac{a_f}{y_f}\right)^{0.65}\left(\frac{V_f}{\sqrt{gy_f}}\right)^{0.43}$$

$$\frac{y_{s\,footing}}{y_f} = 2.0(1.1)(1.0)(1.1)(1.0)(1.0)\left(\frac{8.0}{5.22}\right)^{0.65}\left(\frac{9.84}{\sqrt{32.2\times 5.22}}\right)^{0.43} = 2.83$$

Note that $y_2/a_f = 1.31$ (>0.8); use $K_W = 1.0$

$y_{s\ footing} = 2.83y_f = 2.83 \times 5.22 = 14.8$ ft

Total Local Pier Scour Depth

$y_s = y_{s\ pier} + y_{s\ footing} = 14.8 + 0.6 = 15.4$ ft

6.11.5 Example Problem 5 - Scour at a Complex Pier with Pile Cap in the Flow (English)

During the design of the new Woodrow Wilson Bridge over the Potomac River, several complex pier configurations were tested in physical model studies. The purpose of this problem is to analyze local scour for the possible condition that the main channel migrated to the pier configured as shown in Figure 6.16. It was determined that the water surface elevations would be +7.3 ft and + 9.7 ft for the Q_{100} and the Q_{500} events respectively and the velocities in the main channel would be 11.2 ft/sec and 14 ft/sec for the Q_{100} and the Q_{500} events respectively. The following computations are for the Q_{100} event:

Initial parameters:

$y_1 = 51.8$ ft
$V_1 = 11.2$ ft/sec
$a_{pier} = 32$ ft
$a_{pc} = 53.25$ ft
$h_0 = 25.5$ ft
$h_1 = h_0 + T = 41.5$ ft (resolution of the pile cap thickness below)
S = 13.75 ft (center to center spacing of piles)

T= 16 ft (assign half of the tapered portion of the cap to the pile cap and half to the pier)
f = 8.62 ft (Figure 6.16)
zero angle of attack

Pier Stem Component

f/a_{pier} = 8.62/32 = 0.27

h_1/a_{pier} = 41.5/32 = 1.30

$K_{h\ pier}$ = 0.062 (from Figure 6.5)

$$\frac{y_{s\,pier}}{y_1} = K_{hpier}\left[2.0\,K_1\,K_2\,K_3\,K_4\left(\frac{a_{pier}}{y_1}\right)^{0.65}\left(\frac{V_1}{\sqrt{gy_1}}\right)\right]^{0.43}$$

$$\frac{y_{spier}}{518} = 0.062\left[2.0(11)(10)(11)(10)\left(\frac{32}{518}\right)^{0.65}\left(\frac{112}{\sqrt{(32.2)518}}\right)^{0.43}\right] = 0.0629$$

The quantity in the brackets is the scour ratio for a full depth pier that extends below the scour hole.

$y_{s\ pier}$ = 0.0629 x 51.8 ft = 3.2 ft

Pile Cap Component

$h_2 = h_0 + y_{s\ pier}/2$ = 25.5 + 1.6 = 27.1 ft

$y_2 = y_1 + y_{s\ pier}/2$ = 51.8 + 1.6 = 53.4 ft

$V_2 = V_1$ x (y_1/y_2) = 11.2 x (51.8/53.4) = 10.9 ft/s

Note: For Figure 6.6, $y_{2max} = 3.5\ a_{pc}$ = 186.38 > 53.4; use y_2 = 53.4 ft

h_2/y_2 = 0.51

T/y_2 = 16/53.4 = 0.30

$$\frac{a^*_{pc}}{a_{pc}} = 0.07 \qquad \text{(from Figure 6.6)}$$

a^*_{pc} = 0.07 x 53.25 = 3.7 ft

This is the width of a full depth pier that would produce the same scour depth as the isolated pile cap will produce.

$$\frac{y_{s\,pc}}{y_2} = 2.0 K_1 K_2 K_3 K_4 K_w \left(\frac{a^*_{pc}}{y_2}\right)^{0.65} \left(\frac{V_2}{\sqrt{g y_2}}\right)^{0.43}$$

$$\frac{y_{s\,pc}}{53.4} = 2.0(1.1)(1.0)(1.1)(1.0)(1.0)\left(\frac{3.7}{53.4}\right)^{0.65}\left(\frac{10.9}{\sqrt{(32.2)(53.4)}}\right)^{0.43} = 0.24$$

Note that y_2/a^*_{pc} = 14.4 (>0.8); use K_w = 1.0

$y_{s\ pc}$ = 0.24 x 53.4 = 12.8 ft

Pile Group Component

$h_3 = h_0 + (y_{s\ pier} + y_{s\ pc})/2$ = 25.5 +(3.2 + 12.8)/2= 33.5 ft

$y_3 = y_1 + (y_{s\ pier} + y_{s\ pc})/2$= 51.8 +(3.2 + 12.8)/2 = 59.8 ft

$V_3 = V_1 \times (y_1/y_3)$ =11.2 x (51.8/59.8) = 9.7 ft/s

a_{proj} = 4 x 5.5 = 22.0 ft (from Figure 6.8)

a_{proj}/a = 22.0 / 5.5 = 4.0

S/a = 13.75/5.5 = 2.5 (relative center to center spacing of piles)

K_{sp} = 0.58 (from Figure 6.10)

K_m = 1.16 (From Figure 6.11 for three rows per foundation; foundations separated)

$a^*_{pg} = K_{sp} \times K_m \times a_{proj}$ = 0.58 x 1.16 x 22.0 = 14.8 ft

Note: in Figure 6.12, $y_{3\,max} = 3.5 \times a^*_{pg}$ = 51.8 < 59.8; use y_3 = 51.8 ft

h_3/y_3 = 33.5/51.8 = 0.65

$K_{h\ pg}$ = 0.79 (from Figure 6.12)

$$\frac{y_{s\,pg}}{y_3} = K_{h\,pg}\left[2.0 K_1 K_2 K_3 K_4 \left(\frac{a^*_{pc}}{y_3}\right)^{0.65} \left(\frac{V_3}{\sqrt{g y_3}}\right)^{0.43}\right]$$

$$\frac{y_{s\,pg}}{51.8} = 0.79\left[2.0(1.0)(1.0)(1.1)(1.0)\left(\frac{14.8}{51.8}\right)^{0.65}\left(\frac{9.7}{\sqrt{(32.2)(51.8)}}\right)^{0.43}\right] = 0.41$$

$y_{s\ pg}$ =0.41 x 51.8 = 21.24 ft

Total Estimated Scour

$y_s = y_{s\,pier} + y_{s\,pc} + y_{s\,pg}$ = 3.7 +12.8 + 21.24 = 37.74 ft

6.11.6 Example Problem 6 - Scour at Multiple Columns (English)

Calculate the scour depth for a pier that consists of six 16-inch columns spaced at 7.5 ft with an flow angle of attack of 26 degrees. Debris is not a problem and there is no armoring at this site.

Data:

Columns: 6 columns 1.33 ft, spaced 7.5 ft
Velocity: V_1 = 11.16 ft/s; Depth: y_1 = 20.0 ft
Angle of attack: 26 degrees
Spacing coefficient = S/a = 7.5/1.33 = 5.6; S/a > 5.0
Assume K_3 = 1.1 for plane bed condition

Determine the depth of local scour:

Three methods of calculating the scour depth will be illustrated.

a. Scour depth according to Raudkivi[26] is 1.2 times the local scour of a single column.

$$\frac{y_s}{20} = 2.0 x 1.0 x 1.0 x 1.1 x 1.0 \left(\frac{1.33}{20}\right)^{0.65} \left(\frac{11.16}{(32.2 x 20)^{0.5}}\right)^{0.43} = 0.266$$

y_s = 20 X 0.266 X 1.2 = 6.4 ft

b. Compare this value with that computed by collapsing the columns.

Collapsed pier width = 6 X 1.33 = 8.0 ft

Projected pier width = L Sin 26^0 + a Cos 26^0 = 8.0 Sin 26^0 + 1.33 Cos 26^0 = 4.70 ft

$$\frac{y_s}{20} = 2.0\,(1.0)\,(1.0)\,(1.1)\,(1.0) \left(\frac{4.7}{20}\right)^{0.65} \left(\frac{11.16}{(32.2 \times 20)^{0.5}}\right)^{0.43} = 0.603$$

y_s = 12.1 ft

c. The scour depth can be calculated for multiple columns by calculating the depth for a single column and multiplying it by the K_2 factor given in Equation 6.4. For example:

K_2 = (Cos 26^0+ 8.0/1.33 Sin 26^0 $)^{0.65}$ = 2.27

$$\frac{y_s}{20} = 2.0\,(1.0)\,(2.27)\,(1.1)\,(1.0) \left(\frac{1.33}{20}\right)^{0.65} \left(\frac{11.16}{(32.2 \times 20)^{0.5}}\right)^{0.43} = 0.603$$

y_s = 20 X 0.603 = 12.1 ft

Spacing between columns for this pier is greater than 5 times column diameter so method (a) applies. Also, a model study of the pier gave a scour depth of 6.4 ft. Therefore:

$y_s = 20 \text{ X } 0.266 \text{ X } 1.2 = 6.4$ ft

6.11.7 Example Problem 7 - Pier Scour with Pressure Flow (English)

An existing bridge is subjected to pressure flow to the top of a solid guard rail at the 100-year return period flow. There is only a small increase in flow depth at the bridge for the 500-year return period flow due to the large overbank area. A HEC-RAS model of the flow gives the following data:

Data:

$y_1 =$ 32 ft, $V_1 = 9.61$ ft/s, $q_1 = 307.5$ cfs/ft
Pier width a = 3.0 ft, is round nose, solid, aligned with the flow
Sand bed with $D_{50} = 0.4$ mm and $D_{84} = 0.9$ mm
Distance from stream bed to lower chord (H_b) is 26 ft before scour

Calculate the local pier scour:

Vertical Contraction Scour Depth

$y_s/y_1 = -5.08 + 1.27 \; y_1/H_b + 4.44 \; H_b/y_1 + 0.19 \; V_a/V_c$

$V_c = 11.2 \, (y_1)^{1/6} \, (D_{50})^{1/3} = 11.2 \, (32)^{1/6} \, (0.0013)^{1/3} = 2.18$ ft/s

$V_a = q_1/H_b = 307.5/26 = 11.82$ ft/s

$y_s/32 = -5.08 + 1.27 \, (32/26) + 4.44 \, (26/32) + 0.19 \, (11.82/2.18)$

$y_s/32 = 1.12$ and $y_s = 35.9$ ft

Local Pier Scour

$y_2 = H_b + y_s = 26 + 35.9 = 61.9$ ft

$V_2 = V_a \, (H_b/y_2) = 11.82 \, (26/61.9) = 4.96$ ft/s

$y_s/y_1 = 2.0 \, K_1 \, K_2 \, K_3 \, K_4 \, (a/y_1)^{0.65} \, (Fr)^{0.43}$

$K_1 = K_2 = K_4 = 1.0$; $K_3 = 1.1$; $Fr = 4.96 / (32.2 \text{ X } 61.9)^{0.5} = 0.11$

$y_s/61.9 = 2.0 \text{ X } 1.1 \text{ X } (3.0/61.9)^{0.65} \, (0.11)^{0.43} = 0.12$

$y_s = 7.4$ ft

Total Scour

$y_s = 35.9 + 7.4 = 43.3$ ft

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CHAPTER 7

EVALUATING LOCAL SCOUR AT ABUTMENTS

7.1 GENERAL

Scour occurs at abutments when the abutment and embankment obstruct the flow. Several causes of abutment failures during post-flood field inspections of bridge sites have been documented:[69]

- Overtopping of abutments or approach embankments
- Lateral channel migration or stream widening processes
- Contraction scour
- Local scour at one or both abutments

Abutment damage is often caused by a combination of these factors. Where abutments are set back from the channel banks, especially on wide floodplains, large local scour holes have been observed with scour depths of as much as four times the approach flow depth on the floodplain. As a general rule, the abutments most vulnerable to damage are those located at or near the channel banks.

The flow obstructed by the abutment and approach highway embankment forms a horizontal vortex starting at the upstream end of the abutment and running along the toe of the abutment, and a vertical wake vortex at the downstream end of the abutment (Figure 7.1).

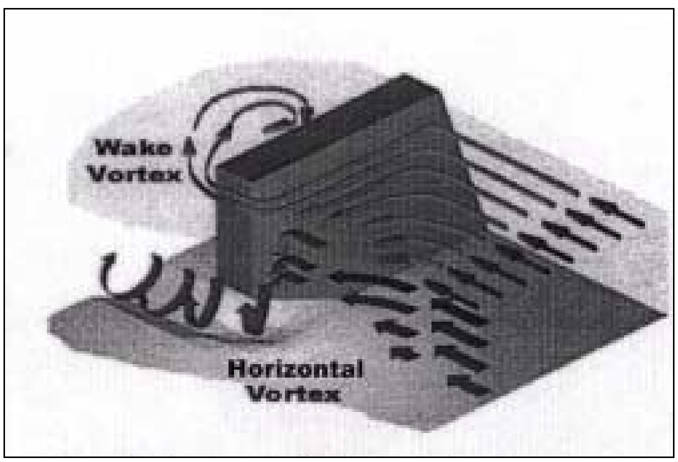

Figure 7.1. Schematic representation of abutment scour.

The vortex at the toe of the abutment is very similar to the horseshoe vortex that forms at piers, and the vortex that forms at the downstream end is similar to the wake vortex that forms downstream of a pier. Research has been conducted to determine the depth and location of the scour hole that develops for the horizontal (so called horseshoe) vortex that occurs at the upstream end of the abutment, and numerous abutment scour equations have been developed to predict this scour depth.

Abutment failures and erosion of the fill also occur from the action of the downstream wake vortex. However, research and the development of methods to determine the erosion from the wake vortex has not been conducted. An example of abutment and approach erosion of a bridge due to the action of the horizontal and wake vortex is shown in Figure 7.2.

Figure 7.2. Scour of bridge abutment and approach embankment.

The types of failures described above are initiated as a result of the obstruction to the flow caused by the abutment and highway embankment and subsequent contraction and turbulence of the flow at the abutments. There are other conditions that develop during major floods, particularly on wide floodplains, that are more difficult to foresee but that need to be considered in the hydraulic analysis and design of the substructure:[69]

- Gravel pits on the floodplain upstream of a structure can capture the flow and divert the main channel flow out of its normal banks into the gravel pit. This can result in an adverse angle of attack of the flow on the downstream highway with subsequent breaching of the embankment and/ or failure of the abutment.

- Levees can become weakened and fail with resultant adverse flow conditions at the bridge abutment.

- Debris can become lodged at piers and abutments and on the bridge superstructure, modifying flow conditions and creating adverse angles of attack of the flow on bridge piers and abutments.

7.2 ABUTMENT SCOUR EQUATIONS

7.2.1 Overview

Equations for predicting abutment scour depths such as Liu et al., Laursen, Froehlich, and Melville are based entirely on laboratory data.[70,48,71,72] The problem is that little field data on abutment scour exist. Liu et al.'s equations were developed by dimensional analysis of the variables with a best-fit line drawn through the laboratory data.[70] Laursen's equations are based on inductive reasoning of the change in transport relations due to the acceleration of the flow caused by the abutment.[48] Froehlich's equations were derived from dimensional analysis and regression analysis of the available laboratory data.[71] Melville's equations were derived from dimensional analysis and development of relations between dimensionless parameters using best-fit lines through laboratory data.[72]

Until recently, the equations in the literature were developed using the abutment and roadway approach length as one of the variables. This approach results in excessively conservative estimates of scour depth. Richardson and Richardson pointed this out in a discussion of Melville's (1992) paper:[73,72]

> "The reason the equations in the literature predict excessively conservative abutment scour depths for the field situation is that, in the laboratory flume, the discharge intercepted by the abutment is directly related to the abutment length; whereas, in the field, this is rarely the case."

Figure 7.3. illustrates the difference. Thus, equations for predicting abutment scour would be more applicable to field conditions if they included the discharge intercepted by the embankment rather than embankment length. Sturm[42,74] concluded that a discharge distribution factor is the appropriate variable to use on local scour depth rather than abutment length.

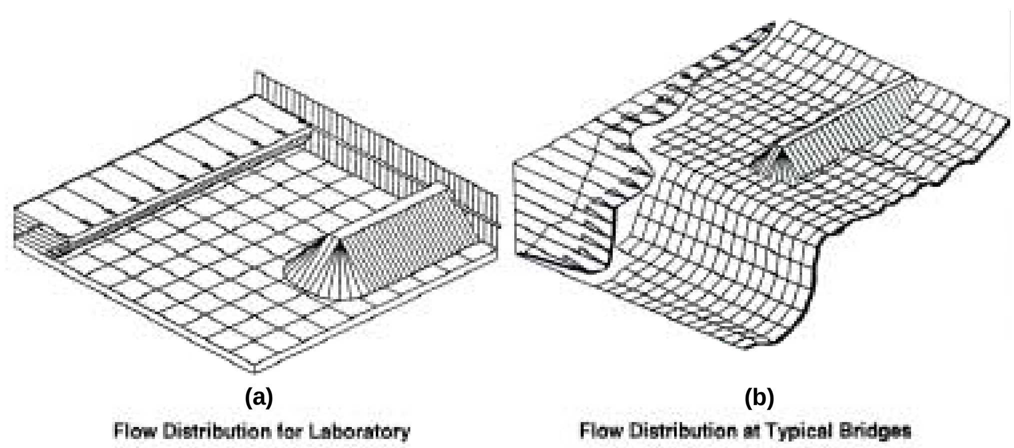

Figure 7.3. Comparison of (a) laboratory flow characteristics to (b) field flow conditions.

Abutment scour depends on the interaction of the flow obstructed by the abutment and roadway approach and the flow in the main channel at the abutment. The discharge returned to the main channel at the abutment is not simply a function of the abutment and roadway length in the field case. Richardson and Richardson noted that abutment scour depth depends on abutment shape, discharge in the main channel at the abutment, discharge intercepted by the abutment and returned to the main channel at the abutment, sediment characteristics, cross-sectional shape of the main channel at the abutment (especially the depth of flow in the main channel and depth of the overbank flow at the abutment), and alignment.[73] In addition, field conditions may have tree-lined or vegetated banks, low velocities, and shallow depths upstream of the abutment. Most of the early laboratory research failed to replicate these field conditions.

Recent research sponsored by the National Cooperative Highway Research Program of the Transportation Research Board has developed an equation to determine abutment scour that includes the discharge intercepted by an abutment and its approach rather than abutment and approach length.[75] The equation and method are presented in Appendix E. In addition, Maryland State Highway Administration has developed a method to determine scour depths at abutments, which is presented in Appendix F.[41,76] Both methods are under development and show promise of improving abutment scour calculations. They should be used with caution, and use of engineering judgment is needed for application at this time.

Abutment foundations should be designed to be safe from long-term degradation, lateral migration, and contraction scour; and protected from local horizontal and wake vortex scour with riprap and/or guidebanks, dikes, or revetments protected with riprap. The two equations provided in this chapter should be used as guides in the design.

7.2.2 Abutment Scour Parameter Determination

Many of the abutment scour prediction equations presented in the literature use the length of an abutment (embankment) projected normal to flow as an independent variable. In practice, the length of embankment projected normal to flow that is used in these relationships is determined from the results of 1-dimensional hydraulic models such as WSPRO[15] or HEC-RAS.[16,17] These models assume an average velocity over the entire cross section (Figure 7.3a). In reality, conveyance and associated velocity and flow depth at the outer extremes of a floodplain are much less, particularly in wide and shallow heavily vegetated floodplains (Figure 7.3b). This flow is typically referred to as "ineffective" flow. When applying abutment scour equations that use the length of embankment projected normal to flow, it is imperative that the length used be the length of embankment blocking "live" flow.

The length of embankment blocking "live" flow can be determined from a graph of conveyance versus distance across a representative cross-section upstream of the bridge (Figure 7.4). If a relatively large portion of a cross-section is required to convey a known amount of discharge in the floodplain, then the length of embankment blocking this flow should probably not be included when determining the length of embankment for use in the abutment scour prediction relationship. Alternately, if the flow in a significant portion of the cross-section has low velocity and/or is shallow, then the length of embankment blocking this flow should probably not be used either. Both WSPRO[15] and HEC-RAS[16,17] can easily compute conveyance versus distance across a cross section.

For example, Figure 7.4 shows the plan view of an embankment blocking three equal conveyance tubes on the right floodplain at a bridge. Since the right conveyance tube occupies the majority of floodplain but conveys only one-third of the floodplain flow, it should not be included in the "live" flow area for determining L'. In this case the length of embankment, L', blocking the "live" flow is approximately the length of the two inner conveyance tubes. In the event that the conveyance versus distance graph does not show a conclusive break point between "live" flow and ineffective flow, an alternative procedure is to estimate L' as the width of the conveyance tube directly upstream of the abutment times the total number of conveyance tubes (including fractional portions) obstructed by the embankment. This length is more representative of the uniform flow conditions in the laboratory experiments used to develop abutment scour equations.

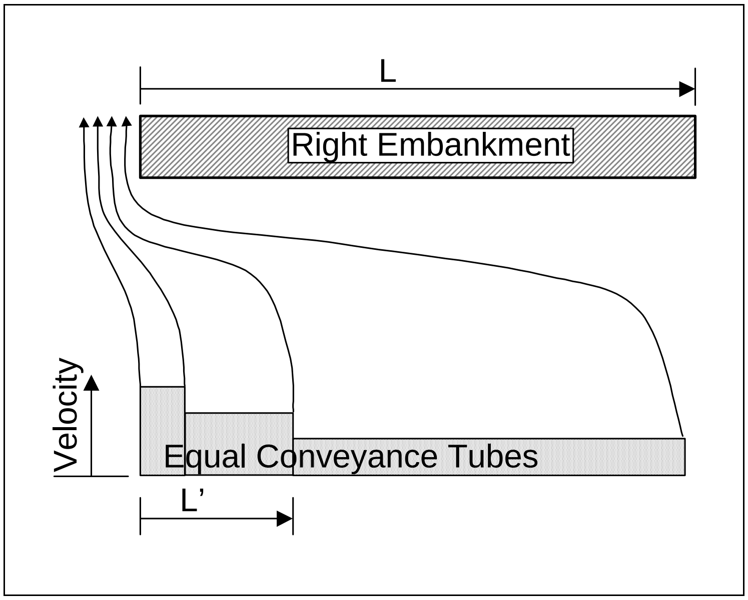

Figure 7.4. Determination of length of embankment blocking live flow for abutment scour estimation.

7.3 ABUTMENT SITE CONDITIONS

Abutments can be set back from the natural stream bank, placed at the bankline or, in some cases, actually set into the channel itself. Common designs include stub abutments placed on spill-through slopes, and vertical wall abutments, with or without wingwalls. Scour at abutments can be live-bed or clear-water scour. The bridge and approach road can cross the stream and floodplain at a skew angle and this will have an effect on flow conditions at the abutment. Finally, there can be varying amounts of overbank flow intercepted by the approaches to the bridge and returned to the stream at the abutment. More severe abutment scour will occur when the majority of overbank flow returns to the bridge opening directly upstream of the bridge crossing. Less severe abutment scour will occur when overbank flows gradually return to the main channel upstream of the bridge crossing.

7.4 ABUTMENT SKEW

The skew angle for an abutment (embankment) is depicted in Figure 7.5. For an abutment angled downstream, the scour depth is decreased, whereas the scour depth is increased for an abutment angled upstream. An equation and guidance for adjusting abutment scour depth for embankment skew are given in Section 7.7.1.

7.5 ABUTMENT SHAPE

There are three general shapes of abutments: (1) spill-through abutments, (2) vertical walls without wing walls, and (3) vertical-wall abutments with wing walls (Figure 7.6). These shapes have varying angles to the flow. As shown in Table 7.1, depth of scour is approximately double for vertical-wall abutments as compared with spill-through abutments. Similarly, scour at vertical wall abutments with wingwalls is reduced to 82 percent of the scour of vertical wall abutments without wingwalls.

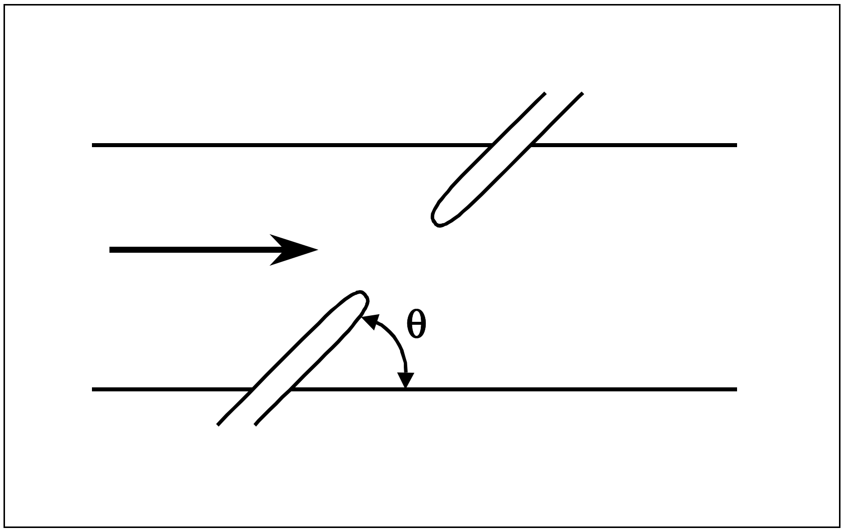

Figure 7.5. Orientation of embankment angle, θ, to the flow.

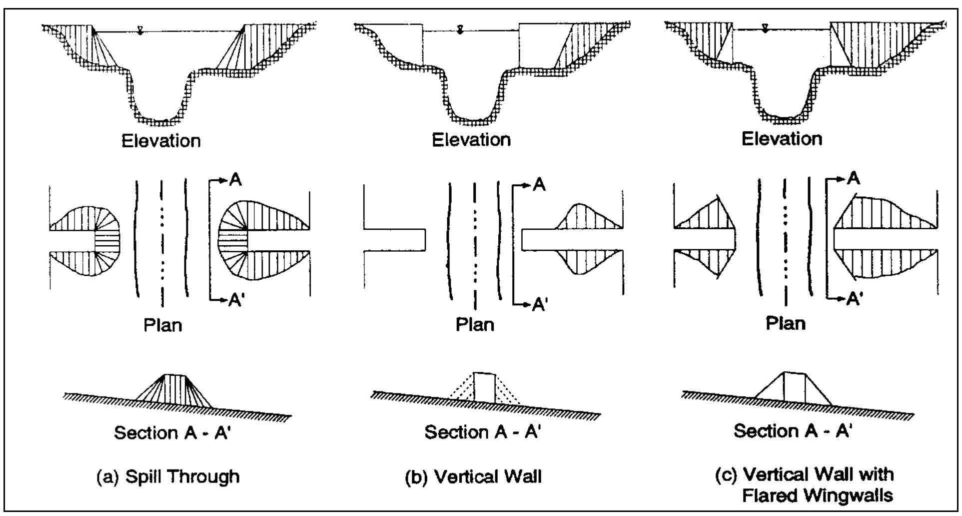

Figure 7.6. Abutment shape.

Table 7.1. Abutment Shape Coefficients.	
Description	K_1
Vertical-wall abutment	1.00
Vertical-wall abutment with wing walls	0.82
Spill-through abutment	0.55

7.6 DESIGNING FOR SCOUR AT ABUTMENTS

The preferred design approach is to place the abutment foundation on scour resistant rock or on deep foundations. Available technology has not developed sufficiently to provide reliable abutment scour estimates for all hydraulic flow conditions that might be reasonably expected to occur at an abutment. **Therefore, engineering judgment is required in designing foundations for abutments. In many cases, foundations can be designed with shallower depths than predicted by the equations when they are protected with rock riprap and/or with a guide bank placed upstream of the abutment designed in accordance with guidelines in HEC-23.[7] Cost will be the deciding factor**.

Based on lessons learned from field evaluations of damaged abutments, consideration should be given to designing deep foundations (piles and shafts) to support both vertical wall abutments and stub abutments on spill-through slopes for the condition where the approach embankment is breached and all supporting soil around the abutment (including the spill through slope) has been removed (see Figure 7.2). Piling for abutments should be driven below the elevation of the long-term degradation and contraction scour. The potential for lateral channel instability should also be considered when designing abutment foundation depths. Some State DOTs evaluate the abutment for scour in a manner similar to that of a pier.

On wide floodplains or on floodplains with complex conditions which could affect future flood flows (confluences, adverse meander patterns and bends, gravel mining pits, ponding of the flow, levee systems, etc.) additional scour countermeasures such as guidebanks, dikes or revetments should be evaluated for inclusion with the initial bridge construction. The intent here is to establish a control to maintain a favorable approach flow condition at the abutment even though upstream conditions may change.

The potential for lateral channel migration, long-term degradation and contraction scour should be considered in setting abutment foundation depths near the main channel. It is recommended that the abutment scour equations presented in this chapter be used to develop insight as to the scour potential at an abutment.

Where spread footings are placed on erodible soil, the preferred approach is to place the footing below the elevation of total scour. If this is not practicable, a second approach is to place the top of footings below the depth of the sum of contraction scour and long-term degradation and to provide scour countermeasures. For spread footings on erodible soil, it becomes especially important to protect adjacent embankment slopes with riprap or other appropriate scour countermeasures. The toe or apron of the riprap serves as the base for the slope protection and must be carefully designed to resist scour while maintaining the support for the slope protection.

In summary, as a minimum, abutment foundations should be designed assuming no ground support (lateral or vertical) as a result of soil loss from long-term degradation, stream instability, and contraction scour. The abutment should be protected from local scour using riprap and/or guide banks. Guidelines for the design of riprap and guide banks are given in HEC-23.[7] To protect the abutment and approach roadway from scour by the wake vortex several DOTs use a 15-meter (50-ft) guide bank extending from the downstream corner of the abutment. Otherwise, the downstream abutment and approach should be protected with riprap or other countermeasures.

In the following sections, two equations are presented for use in estimating scour depths as a guide in designing abutment foundations. The methods can be used for either **clear-water or live-bed** scour.

7.7 LIVE-BED SCOUR AT ABUTMENTS

As a check on the potential depth of scour to aid in the design of the foundation and placement of rock riprap and/or guide banks, Froehlich's[70] live-bed scour equation or the HIRE equation in HDS 6[22] can be used.

7.7.1 Froehlich's Live-Bed Abutment Scour Equation

Froehlich[71] analyzed 170 live-bed scour measurements in laboratory flumes by regression analysis to obtain the following equation:

$$\frac{y_s}{y_a} = 2.27\, K_1\, K_2 \left(\frac{L'}{y_a}\right)^{0.43} Fr^{0.61} + 1 \qquad (7.1)$$

where:

K_1	=	Coefficient for abutment shape (Table 7.1)
K_2	=	Coefficient for angle of embankment to flow
K_2	=	$(\theta/90)^{0.13}$ (see Figure 7.4 for definition of θ) $\theta<90°$ if embankment points downstream $\theta>90°$ if embankment points upstream
L'	=	Length of active flow obstructed by the embankment, m (ft)
A_e	=	Flow area of the approach cross section obstructed by the embankment, m^2 (ft^2)
Fr	=	Froude Number of approach flow upstream of the abutment = $V_e/(gy_a)^{1/2}$
V_e	=	Q_e/A_e, m/s (ft/s)
Q_e	=	Flow obstructed by the abutment and approach embankment, m^3/s (ft^3/s)
y_a	=	Average depth of flow on the floodplain (A_e/L), m (ft)
L	=	Length of embankment projected normal to the flow, m (ft)
y_s	=	Scour depth, m (ft)

It should be noted that Equation 7.1 is not consistent with the fact that as L' tends to 0, y_s also tends to 0. The 1 was added to the equation so as to envelope 98 percent of the data. See Section 7.2.2 and Figure 7.4 for guidance on estimating L'.

7.7.2 HIRE Live-Bed Abutment Scour Equation

An equation based on field data of scour at the end of spurs in the Mississippi River (obtained by the USACE) can also be used for estimating abutment scour.[22] This field situation closely resembles the laboratory experiments for abutment scour in that the discharge intercepted by the spurs was a function of the spur length. The modified equation, referred to herein as the HIRE equation, is applicable when the ratio of projected abutment length (L) to the flow depth (y_1) is greater than 25. This equation can be used to estimate scour depth (y_s) at an abutment where conditions are similar to the field conditions from which the equation was derived:

$$\frac{y_s}{y_1} = 4\ Fr^{0.33}\ \frac{K_1}{0.55}\ K_2 \tag{7.2}$$

where:

- y_s = Scour depth, m (ft)
- y_1 = Depth of flow at the abutment on the overbank or in the main channel, m (ft)
- Fr = Froude Number based on the velocity and depth adjacent to and upstream of the abutment
- K_1 = Abutment shape coefficient (from Table 7.1)
- K_2 = Coefficient for skew angle of abutment to flow calculated as for Froehlich's equation (Section 7.7.1)

7.8 CLEAR-WATER SCOUR AT AN ABUTMENT

Equations 7.1 and 7.2 are recommended for both live-bed and clear-water abutment scour conditions. If a method other than Froehlich's equation is used, it is suggested that scour for both the clear water and live bed condition be computed (see Appendix E and Appendix F). Engineering judgment should then be used to select the most appropriate scour depth.

7.9 ABUTMENT SCOUR EXAMPLE PROBLEMS (SI)

7.9.1 Example Problem 1 (SI)

Determine abutment scour depth for the following conditions to aid in scour evaluation and design of countermeasures. The right abutment is at the bankline with 3.00 m of overbank flow width. The left abutment projects into the channel 61.96 m. Each of these lengths represents the full length of obstruction of active flow. The projection on the left side is the result of stream erosion and widening. The right channel bank is 0.61 m high and the embankment extends back 3.00 m to a 3 m high bank. The bridge and approach are oriented at a 10° angle upstream to the flow from the right side.

Given:

Upstream channel depth = 2.62 m
Discharge = 773.05 m^3/s
Bridge is vertical wall with wingwalls

Original (unscoured) depth of flow at bridge is estimated as 2.16 m

Right Abutment

$$L = L' = 3\,\text{Cos}\,10^\circ = 2.95\,\text{m}$$

$$y_a = 2.62 - 0.61 = 2.01\,\text{m}$$

$$\frac{L}{y_a} = \frac{2.95}{2.01} = 1.47 < 25 \text{ (Use Froehlich Equation)}$$

$$\frac{y_s}{y_a} = 2.27 K_1\, K_2 \left(\frac{L'}{y_a}\right)^{0.43} Fr^{0.61} + 1$$

$$K_1 = 0.82$$

$$K_2 = \left(\frac{\theta}{90}\right)^{.13} = \left(\frac{100}{90}\right)^{0.13} = 1.01 \text{ (Abutment angles } 10^\circ \text{ upstream)}$$

$$A_e = 2.01 \text{x} 2.95 = 5.93\,\text{m}^2$$

$Q_e = 17.8\,\text{m}^3 / \text{s};\ V_e = 3.00\,\text{m/s}$ (Q_e and V_e are obtained from HEC-RAS)

$$Fr = \frac{V_e}{\sqrt{g y_a}} = \frac{3.00}{(9.81 \text{x} 2.01)^{1/2}} = 0.68$$

$$\frac{y_s}{2.01} = 2.27\ (0.82)\ (1.01)\left(\frac{2.95}{2.01}\right)^{.43} (0.68)^{0.61} + 1 = 2.75$$

$$y_s = 2.75 \text{ x } 2.01 = 5.53\,\text{m}$$

Left Abutment

$$L = 61.96\ \text{Cos}\ 10^\circ = 61.02\,\text{m}$$

$$y_1 = 2.16\,\text{m}$$

$$\frac{L}{y_1} = \frac{61.02}{2.16} = 28.25 > 25 \text{ (Use HIRE Equation)}$$

$$\frac{y_s}{y_1} = 4 Fr^{0.33} \frac{K_1}{0.55} K_2$$

$v_1 = 3.72\,\text{m/s}$ (From HEC-RAS stream tube next to abutment)

$$Fr = \frac{v_1}{\sqrt{gy_1}} = \frac{3.72}{(9.81 x 2.16)^{1/2}} = 0.81$$

$$K_1 = 0.82$$

$$K_2 = \left(\frac{80}{90}\right)^{.13} = 0.98$$

$$\frac{y_s}{y_1} = 4(0.81)^{0.33} \frac{0.82}{0.55}(0.98) = 5.45$$

$$y_s = 5.45 \, x \, 2.16 = 11.8 \, m$$

7.10 ABUTMENT SCOUR EXAMPLE PROBLEMS (English)

7.10.1 Example Problem 1 (English)

Determine abutment scour depth for the following conditions to aid in scour evaluation and design of countermeasures. The right abutment is at the bankline with 9.8 ft of overbank flow width. The left abutment projects into the channel 200 ft. Each of these lengths represents the full length of obstruction of active flow. The projection on the left side is the result of stream erosion and widening. The right channel bank is 2 ft high and the embankment extends back 9.8 ft to a 9.8 ft high bank. The bridge and approach are oriented at a 10° angle upstream to the flow from the right side.

Given:

Upstream channel depth = 8.6 ft
Discharge is 27,300 cfs
Bridge is vertical wall with wingwalls

Original (unscoured) depth of flow at bridge is estimated as 7.1 ft

Right Abutment

$$L = L' = 9.8 \, Cos 10° = 9.7 \, ft$$

$$y_a = 8.6 - 2.0 = 6.6 \, ft$$

$$\frac{L}{y_a} = \frac{9.7}{6.6} = 1.47 < 25 \text{ (Use Froehlich Equation)}$$

$$\frac{y_s}{y_a} = 2.27 K_1 K_2 \left(\frac{L'}{y_a}\right)^{0.43} Fr^{0.61} + 1$$

$$K_1 = 0.82$$

$$K_2 = \left(\frac{\theta}{90}\right)^{.13} = \left(\frac{100}{90}\right)^{0.13} = 1.01 \text{ (Abutment angles } 10^\circ \text{ upstream)}$$

$$A_e = 6.6 \,x\, 9.7 = 64.0\, ft^2$$

$Q_e = 629\, cfs;\ V_e = 9.8\, ft/s$ (Q_e and V_e are obtained from HEC-RAS)

$$Fr = \frac{V_e}{\sqrt{g y_a}} = \frac{9.8}{(32.2 \,x\, 6.6)^{1/2}} = 0.67$$

$$\frac{y_s}{6.6} = 2.27\,(0.82)\,(1.01)\left(\frac{9.7}{6.6}\right)^{.43}(0.67)^{0.61} + 1 = 2.74$$

$$y_s = 2.74 \,x\, 6.6 = 18.1 ft$$

Left Abutment

$$L = 200 \,Cos\, 10^\circ = 197.0\, ft$$

$$y_1 = 7.1 ft$$

$$\frac{L}{y_1} = \frac{197.0}{7.1} = 27.7 > 25 \text{ (Use HIRE Equation)}$$

$$\frac{y_s}{y_1} = 4 Fr^{0.33} \frac{K_1}{0.55} K_2$$

$v_1 = 12.2\, ft/s$ (From HEC-RAS stream tube next to abutment)

$$Fr = \frac{v_1}{\sqrt{g y_1}} = \frac{12.2}{(32.2 \,x\, 7.1)^{1/2}} = 0.81$$

$$K_1 = 0.82$$

$$K_2 = \left(\frac{80}{90}\right)^{.13} = 0.98$$

$$\frac{y_s}{y_1} = 4(0.81)^{0.33}\,\frac{0.82}{0.55}\left(0.98\right) = 5.45$$

$$y_s = 5.45 \times 7.1 = 38.7\ \text{ft}$$

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CHAPTER 8

COMPREHENSIVE EXAMPLE SCOUR PROBLEM

8.1 GENERAL DESCRIPTION OF PROBLEM

This example problem is taken from a paper by Arneson et al.[77] FHWA's WSPRO computer program was used to obtain the hydraulic variables. The program uses 20 stream tubes to give a quasi 2-dimensional analysis. Each stream tube has the same discharge (1/20 of the total discharge). The stream tubes provide the velocity distribution across the flow and the program has excellent bridge routines. The problem presented here is worked in SI (metric) units, however, the same problem worked in English units is presented in Appendix H. The solution follows Steps 1-7 of the specific design approach of Chapter 2 (Section 2.4).

A 198.12-m long bridge (Figure 8.1) is to be constructed over a channel with spill-through abutments (slope of 1V:2H). The left abutment is set approximately 60.5 m back from the channel bank. The right abutment is set at the channel bank. The bridge deck is set at elevation 6.71 m and has a girder depth of 1.22 m. Six round-nose piers are evenly spaced in the bridge opening. The piers are 1.52 m thick, 12.19 m long, and are aligned with the flow. The 100-year design discharge is 849.51 m^3/s. The 500-year flow of 1444.16 m^3/s was estimated by multiplying the Q_{100} by 1.7 since no hydrologic records were available to predict the 500-year flow.

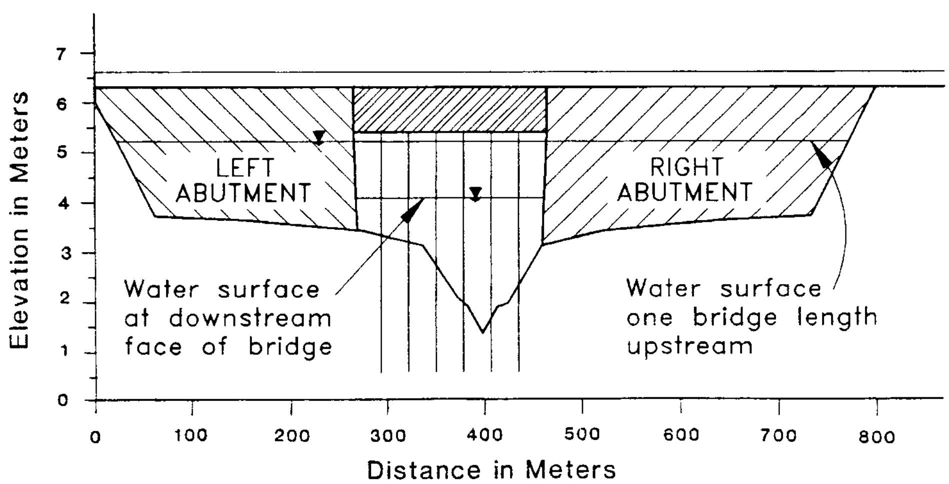

Figure 8.1. Cross section of proposed bridge.

8.2 STEP 1: DETERMINE SCOUR ANALYSIS VARIABLES

From Level 1 and Level 2 analysis: a site investigation of the crossing was conducted to identify potential stream stability problems at this crossing. Evaluation of the site indicates that the river has a relatively wide floodplain. The floodplain is well vegetated with grass and trees; however, the presence of remnant channels indicates that there is a potential for lateral shifting of the channel.

The bridge crossing is located on a relatively straight reach of channel. The channel geometry is relatively the same for approximately 300 m up- and downstream of the bridge crossing. The D_{50} of the bed material and overbank material is approximately 0.002 m (2 mm). The maximum grain size of the bed material is approximately 0.008 m (8 mm). The specific gravity of the bed material was determined to be equal to 2.65.

The river and crossing are located in a rural area with the primary land use consisting of agriculture and forest.

Review of bridge inspection reports for bridges located upstream and downstream of the proposed crossing indicates no long-term aggradation or degradation in this reach. At the bridge site, bedrock is approximately 46 m below the channel bed.

Since this is a sand-bed channel, no armoring potential is expected. Furthermore, the bed for this channel at low flow consists of dunes which are approximately 0.3 to 0.5 m high. At higher flows, above the Q_5, the bed will be either plane bed or antidunes.

The left and right banks are relatively well vegetated and stable; however, there are isolated portions of the bank which appear to have been undercut and are eroding. Brush and trees grow to the edge of the banks. Banks will require riprap protection if disturbed. Riprap will be required upstream of the bridge and extend downstream of the bridge.

8.2.1 Hydraulic Characteristics

Hydraulic characteristics at the bridge were determined using WSPRO.[15] Three cross sections were used for this analysis and are denoted as "EXIT" for the section downstream of the bridge, "FULLV" for the full-valley section at the bridge, and "APPR" for the approach section located one bridge length upstream of the bridge. The bridge geometry was superimposed on the full-valley section and is denoted "BRDG." Values used for this example problem are based on the output from the WSPRO model which is presented in Appendix G (SI). Specific values for scour analysis variables are given for each computation separately and cross referenced to the line numbers of the WSPRO output.

The HP2 option was used to provide hydraulic characteristics at both the bridge and approach sections. This WSPRO option subdivides the cross section into 20 equal conveyance tubes. Figures 8.2 and 8.3 illustrate the location of these conveyance tubes for the approach and bridge cross section, respectively. Figure 8.4 illustrates the average velocities in each conveyance tube and the contraction of the flow from the approach section through the bridge. Figure 8.4 also identifies the equal conveyance tubes of the approach section which are cut off by the abutments.

Hydraulic variables for performing the various scour computations were determined from the WSPRO output (Appendix G) and from Figures 8.2, 8.3, and 8.4. These variables which will be used to compute contraction scour and local scour are presented in Tables 8.1 through 8.6.

Contraction scour will occur both in the main channel and on the left overbank of the bridge opening. For the main channel, contraction scour could be either clear-water or live-bed depending on the magnitude of the channel velocity and the critical velocity for sediment movement. A computation will be performed to determine the sediment transport characteristics of the main channel and the appropriate contraction scour equation.

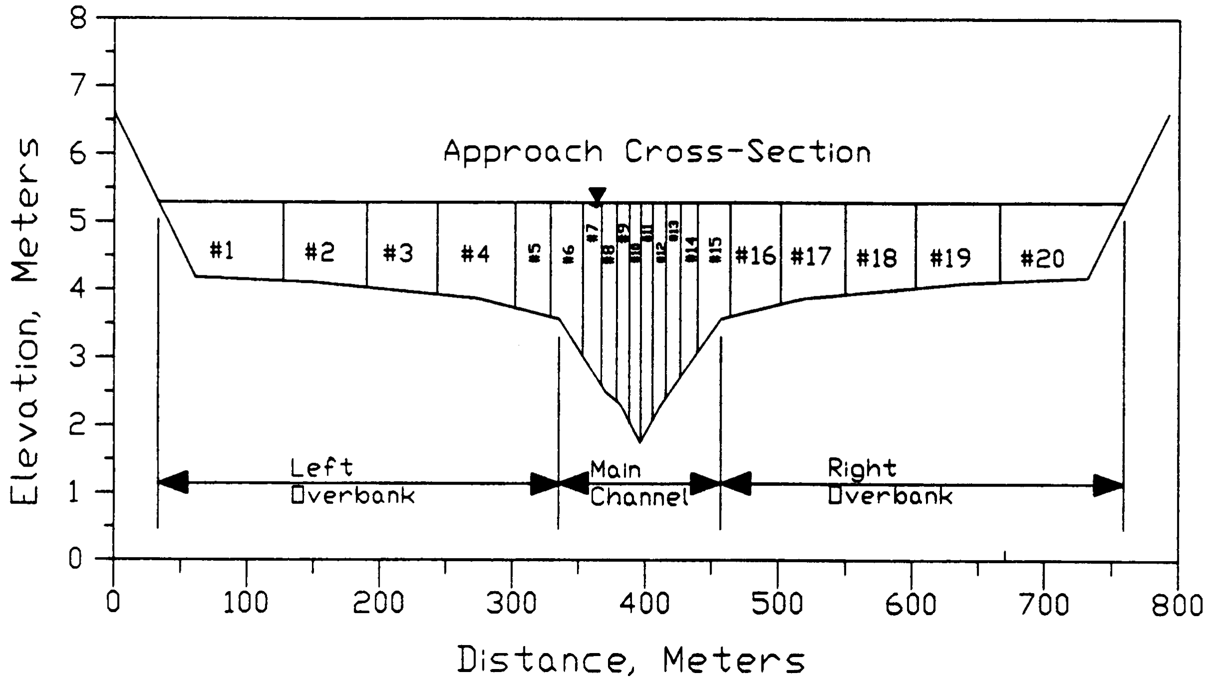

Figure 8.2. Equal conveyance tubes of approach section.

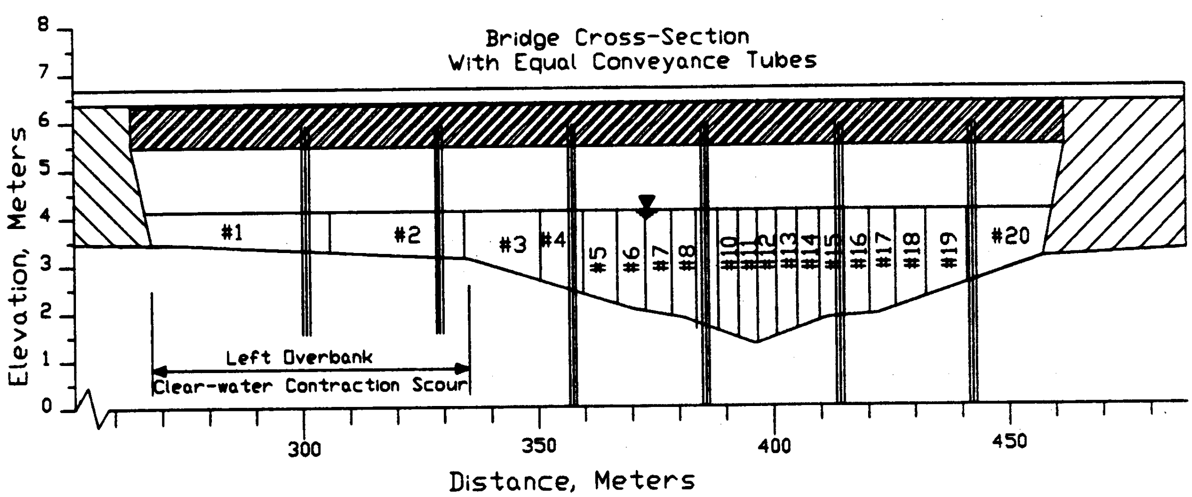

Figure 8.3. Equal conveyance tubes of bridge section.

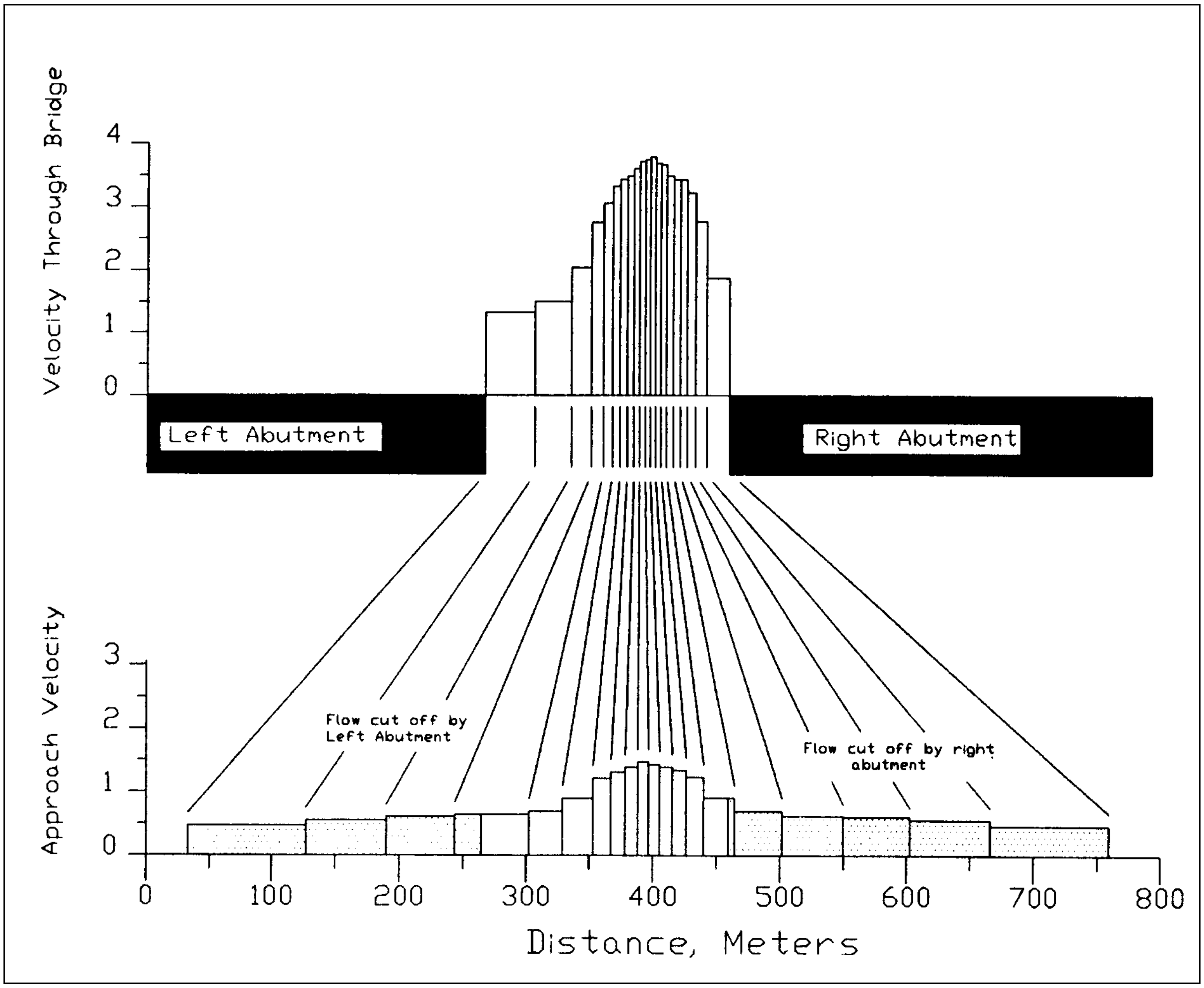

Figure 8.4. Plan view of equal conveyance tubes showing velocity distribution at approach and bridge sections.

Table 8.1. Hydraulic Variables from WSPRO for Estimation of Live-bed Contraction Scour.		
		Remarks
Q (m^3/s)	849.51	Total discharge, line 8 of WSPRO input or Line 26 of WSPRO output.
K_1 (Approach)	19 000	Conveyance of main channel of approach. Line 378 of WSPRO output, SA#2.
K_{total} (Approach)	39 150	Total conveyance of approach section. Line 380 of WSPRO output.
W_1 or TOPW (Approach) (m)	121.9	Topwidth of flow (TOPW). Assumed to represent active live bed width of approach. Line 378 of WSPRO output, SA#2.
A_c (Approach) (m^2)	320	Area of main channel approach section. Line 378, SA#2.
WETP (Approach) (m)	122.0	Wetted perimeter of main channel approach section. Line 378 of WSPRO output, SA#2.
K_c (Bridge)	11 330	Conveyance of main channel through bridge. Line 333 of WSPRO output, SA#2.
K_{total} (Bridge)	12 540	Total conveyance through bridge. Line 334 of WSPRO output.
A_c (Bridge) (m^2)	236	Area of the main channel, bridge section. Line 333 of WSPRO output, SA #2.
W_c (Bridge) (m)	122	Channel width at the bridge. Difference between subarea break-points defining banks at bridge, line 109 of WSPRO output.
W_2 (Bridge) (m)	115.9	Channel width at bridge, less 4 channel pier widths (6.08 m).
S_f (m/m)	0.002	Average unconstricted energy slope (SF). Line 260, or 266 of WSPRO output.

Table 8.2. Hydraulic Variables from WSPRO for Estimation of Clear-water Contraction Scour on Left Overbank.		
		Remarks
Q (m^3/s)	849.51	Total discharge, (see Table 8.1).
Q_{chan} (Bridge) (m^3/s)	767.54	Flow in main channel at bridge. Determined in live-bed computation of step 3A.
Q_2 (Bridge) (m^3/s)	81.97	Flow in left overbank through bridge. Determined by subtracting Q_{chan} (listed above) from total discharge through bridge.
D_m (Bridge Overbank) (m)	0.0025	Grain size of left overbank area. $D_m = 1.25\ D_{50}$.
$W_{setback}$ (Bridge)(m)	68.8	Topwidth of left overbank area (SA #1) at bridge. Line 332, of WSPRO output.
$W_{contracted}$ (Bridge) (m)	65.8	Set back width less two pier widths (3.04 m)
A_{left} (Bridge) (m^2)	57	Area of left overbank at the bridge. Line 332 of WSPRO output, SA #1.

Table 8.3. Hydraulic Variables from WSPRO for Estimation of Pier Scour (Conveyance Tube Number 12).		
		Remarks
V_1 (m/s)	3.73	Velocity in conveyance tube #12. Line 314 of WSPRO output.
Y_1 (m)	2.84	Mean depth of tube #12. Line 315 of WSPRO output.

Table 8.4. Hydraulic Variables from WSPRO for Estimation of Abutment Scour Using Froehlich's Equation for Left Abutment.		
		Remarks
Q (m^3/s)	849.51	Total discharge (Table 8.1)
q_{tube} (m^3/s)	42.48	Discharge per equal conveyance tube, defined as total discharge divided by 20.
#Tubes	3.5	Number of approach section conveyance tubes which are obstructed by left abutment. Determined by super-imposing abutment geometry onto the approach section (Figure 8.4)
Q_e (m^3/s)	148.68	Flow in left overbank obstructed by left abutment and approach embankment. Determined by multiplying # Tubes and q_{tube}.
A_e (left abut.) (M^2)	264.65	Area of approach section conveyance tubes number 1, 2, 3, and half of tube 4. Line 347 of WSPRO output.
L (m)	232.80	Length of abutment projected into flow, determined by adding top widths of approach section conveyance tubes number 1, 2, 3, and half of tube 4. Line 346 of WSPRO output.
L′ (m)	169.4	Length of active flow obstructed by embankment. Width or approach section conveyance tube directly upstream of abutment times the number of conveyance tubes blocked by embankment. (290.5 - 242.1) x 3.5 = 169.4 Note: Conveyance tube widths from line 346 of WSPRO output.

Table 8.5. Hydraulic Variables from WSPRO for Estimation of Abutment Scour Using HIRE Equation for Left Abutment.		
		Remarks
V_{tube} (m/s) (Bridge x-Section)	1.29	Mean velocity of conveyance tube #1, adjacent to left abutment. Line 304 of WSPRO output.
y_1 (m) (Bridge x-Section)	0.83	Average depth of conveyance tube #1. Line 305 of WSPRO output.

Table 8.6. Hydraulic Variables from WSPRO for Estimation of Abutment Scour Using HIRE Equation for Right Abutment.		
		Remarks
V_{tube} (m/s)	2.19	Mean velocity of conveyance tube 20, adjacent to right abutment. Line 319 of WSPRO output.
y_1 (m)	1.22	Average depth of conveyance tube 20. Line 320 of WSPRO output.

In the overbank area adjacent to the left abutment, clear-water scour will occur. This is because the overbank areas upstream of the bridge are vegetated, and because the velocities in these areas will be low. Thus, returning overbank flow which will pass under the bridge adjacent to the left abutment will not be transporting significant amounts of material to replenish the scour on the left overbank adjacent to the left abutment.

Because of this, two computations for contraction scour will be required. The first computation, which will be illustrated in Step 3A will determine the magnitude of the contraction scour in the main channel. The second computation, which is illustrated in Step 3B will utilize the clear-water equation for the left overbank area. Hydraulic data for these two computations are presented in Tables 8.1 and 8.2 for the channel and left overbank contraction scour computations, respectively.

Table 8.3 lists the hydraulic variables which will be used to estimate the local scour at the piers (Step 5). These hydraulic variables were determined from a plot of the velocity distribution derived from the WSPRO output (Figure 8.5). For this example the highest velocities and flow depths in the bridge cross section will be used (at conveyance tube number 12). Only one pier scour computation will be completed because the possibility of thalweg shifting and lateral migration will require that all of the piers be set assuming that any pier could be subjected to the maximum scour producing variables.

Local scour at the left abutment and right abutment will be illustrated in steps 6A and B using the HIRE equation. Scour variables derived from the WSPRO output for these computations are presented in Tables 8.4 and 8.5.

8.3 STEP 2: ANALYZE LONG-TERM BED ELEVATION CHANGE

Evaluation of stage discharge relationships and cross sectional data obtained from other agencies do not indicate progressive aggradation or degradation. Also, long-term aggradation or degradation are not evident at neighboring bridges. Based on these observations, the channel is relatively stable vertically, at present. Furthermore, there are no plans to change the local land use in the watershed. The forested areas of the watershed are government-owned and regulated to prevent wide spread fire damage, and instream gravel mining is prohibited. These observations indicate that future aggradation or degradation of the channel, due to changes in sediment delivery from the watershed, are minimal.

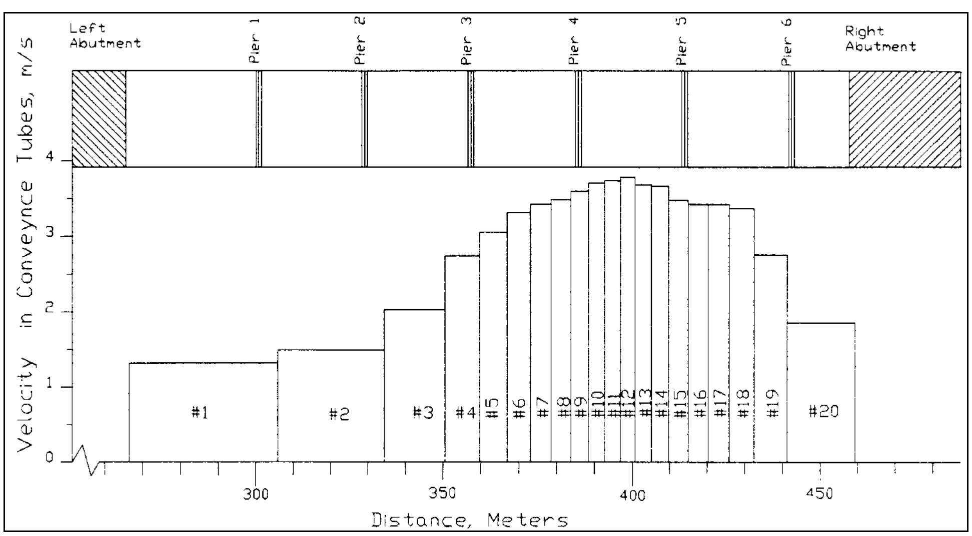

Figure 8.5. Velocity distribution at bridge crossing.

Based on these observations, and due to the lack of other possible impacts to the river reach, it is determined that the channel will be relatively stable vertically at the bridge crossing and long-term aggradation or degradation potential is considered to be minimal. However, there is evidence that the channel is unstable laterally. This will need to be considered when assessing the total scour at the bridge.

8.4 STEP 3A: COMPUTE THE MAGNITUDE OF THE GENERAL (CONTRACTION) SCOUR IN MAIN CHANNEL

As a precursor to the computation of contraction scour in the main channel under the bridge, it is first necessary to determine whether the flow condition in the main channel is either live-bed or clear-water. This is determined by comparing the critical velocity for sediment movement at the approach section to the average channel velocity of the flow at the approach section as computed using the WSPRO output. This comparison is conducted using the average velocity in the main channel of the approach section to the bridge. If the average computed channel velocity is greater than the critical velocity, the live-bed equation should be used. Conversely, if the average channel velocity is less than the critical velocity, the clear-water equation is applicable. The following computations are based on the quantities tabulated in Table 8.1.

The discharge in the main channel of the approach section is determined from the ratio of the conveyance in the main channel to the total conveyance of the approach section. By multiplying this ratio by the total discharge, the discharge in the main channel at the approach section (Q_1) is computed.

$$Q_1 = Q\ (K_1 / K_{total}) = 849.51\, m^3 / s \left(\frac{19\ 000}{35\ 150}\right)$$

$$Q_1 = 412.28\ m^3 / s$$

The average velocity in the main channel of the approach section is determined by dividing the discharge computed in Equation 8.1 by the cross-sectional area of the main channel.

$$V_1 = (Q_1 / A_c) = \left(\frac{412.28}{320}\right) = 1.29\ m / s$$

The average flow depth in the approach section is determined by dividing the flow area by the topwidth of the channel.

$$y_1 = \left(A_1 / TOPW\right) = \left(\frac{320}{121.9}\right) = 2.63\ m$$

The channel velocity is compared to the critical velocity of the D_{50} size for sediment movement (V_c) to determine whether the flow condition is either clear-water or live-bed.

$$V_c = 6.19\ y_1^{1/6}\ D_{50}^{1/3}$$

$$V_c = 6.19\ (2.63\ \text{m})^{1/6}\ (0.002\ \text{m})^{1/3}$$

$$V_c = 0.92\ \text{m/s}$$

Since the average velocity in the main channel is greater than the critical velocity ($V_1 > V_c$), the flow condition will be live-bed. The following computations illustrate the computation of the contraction scour using the live-bed equation.

The following computation determines the mode of bed material transport and the factor k_1. All hydraulic parameters which are needed for this computation are listed in Table 8.1.

The hydraulic radius of the approach channel is:

$$R = \frac{A_c}{WETP} = \frac{320\ \text{m}^2}{122\ \text{m}} = 2.62\ \text{m}$$

Notice that the hydraulic radius of the approach is nearly equal to the average flow depth computed earlier (Equation 8.3). This condition indicates that the channel is wide with its width greater than 10 times the flow depth. **If the width was less than 10 times the average flow depth, the channel could not be assumed to be wide and the hydraulic radius would deviate from the average flow depth**.

The average shear stress on the channel bed is:

$$\tau_o = \gamma\ R\ S$$

$$\tau_o = (9810\ \text{N/m}^3)\ (2.62\ \text{m})\ (0.002\ \text{m/m}) = 51.4\ \text{N/m}^2 = 51.4\ \text{Pa}$$

The shear velocity in the approach channel is:

$$V_* = (\tau_o / \rho)^{0.5} = (54.1/1000)^{0.5} = 0.227\ \text{m/s}$$

Bed material is sand with D_{50} = 0.002 m (2mm).
Fall velocity (ω) = 0.21 m/s from Figure 5.8 at 20°C and D_s = 2 mm

Therefore

$$\frac{V_*}{\omega} = \frac{0.227}{0.21} = 108$$

From the above, the coefficient k_1 is determined (from the discussion for Equation 5.2) to be equal to 0.64 which indicates that the mode of bed material transport is a mixture of suspended and contact bed material discharge.

The discharge in the main channel at the bridge (Q_2) is determined from the ratio of conveyances for the bridge section. This procedure for obtaining the discharge is similar to the procedure used to obtain the discharge in the main channel of the approach which was previously illustrated in Equation 8.1.

$$Q_2 = Q\,(K_2 / K_{total}) = 849.51\,m^3/s \left(\frac{11\,330}{12\,540}\right)$$

$$Q_2 = 767.54\ m^3/s$$

The channel widths at the approach and bridge section are given in Table 8.1. Therefore all parameters to determine live-bed contraction scour have been determined and Equation 5.2 can be employed.

$$\frac{y_2}{y_1} = \left(\frac{Q_2}{Q_1}\right)^{6/7} \left(\frac{W_1}{W_2}\right)^{k_1}$$

$$\frac{y_2}{2.63} = \left(\frac{767.54}{412.28}\right)^{6/7} \left(\frac{1219}{115.9}\right)^{0.64} = 176$$

$$y_2 = (2.63)(176) = 4.63\text{m}$$

Live-bed contraction scour is calculated by subtracting the flow depth in the bridge (y_0) from y_2. The bridge channel flow depth (y_0) is the area divided by the topwidth, y_0 = 236 m^2/122 m = 1.93 m. Therefore, the depth of contraction scour in the main channel is:

$$y_s = y_2 - y_0 = 4.63\ \text{m} - 193\ \text{m} = 2.7\ \text{m}$$

This amount of contraction scour is large and could be minimized by increasing the bridge opening, providing for relief bridges in the overbank, or in some cases, providing for highway approach overtopping.

If this were the design of a new bridge, the excessive backwater (0.61 m) would require a change in the design to meet FEMA backwater requirements. The increase in backwater is obtained by subtracting the elevation given in line 264 from the elevation given in line 281 in Appendix G. However, in the evaluation of an existing bridge for safety from scour, this amount of contraction scour could occur and the scour analysis should proceed.

8.5 STEP 3B: COMPUTE GENERAL (CONTRACTION) SCOUR FOR LEFT OVERBANK

Clear-water contraction scour will occur in the overbank area between the left abutment and the left bank of bridge opening. Although the bed material in the overbank area is soil, it is protected by vegetation. Therefore, there would be no bed-material transport into the set-back bridge opening (clear-water conditions). The subsequent computations are based on the discharge and depth of flow passing under the bridge in the left overbank. These hydraulic variables were determined from the WSPRO output and are tabulated in Table 8.2.

Computation of clear-water contraction scour (Equation 5.4)

$$y_2 = \left[\frac{0.025\ Q^2}{(D_m^{2/3}\ W_{contracted}{}^2)}\right]^{3/7}$$

Computation of contraction scour flow depth in left overbank area under the bridge, y_2:

$$y_2 = \left[\frac{0.025\ (81.97\ m^3 / s)^2}{(0.0025\ m)^{2/3}\ (65.8\ m)^2} \right]^{3/7} = 1.38\ m$$

Computation of average flow depth in left overbank bridge section, y_0:

$$y_0 = \frac{A}{TOPW} = \frac{(57.0\ m^2)}{(68.8\ m)} = 0.83\ m$$

Therefore, the clear-water contraction scour in the left overbank of the bridge opening is:

$$y_s = y_2 - y_0 = 1.38\ m - 0.83\ m = 0.55\ m$$

8.6 STEP 4: COMPUTE THE MAGNITUDE OF OTHER GENERAL SCOUR COMPONENTS

The crossing is on a relatively straight reach with no channel braiding, and there are no downstream controls of water surface elevations. Thus, the other general scour components (bend scour, confluence scour, etc) will not be a factor.

8.7 STEP 5: COMPUTE THE MAGNITUDE OF LOCAL SCOUR AT PIERS

It is anticipated that any pier under the bridge could potentially be subject to the maximum flow depths and velocities derived from the WSPRO hydraulic model (Table 8.3). Therefore, only one computation for pier scour is conducted and assumed to apply to each of the six piers for the bridge. This assumption is appropriate based on the fact that the thalweg is prone to shifting and because there is a possibility of lateral channel migration.

8.7.1 Computation of Pier Scour

The Froude Number for the pier scour computation is based on the hydraulic characteristics of conveyance tube number 12. Therefore:

$$Fr_1 = \frac{V}{(g\ y_1)^{0.5}} = \frac{3.73\ m / s}{[(9.81\ m / s^2)\ (2.84\ m)]^{0.5}}$$

$$Fr_1 = 0.71$$

For a round-nose pier, aligned with the flow and sand-bed material:

$$K_1 = K_2 = K_4 = 1.0$$

For plane-bed condition:

$$K_3 = 1.1$$

Using Equation 6.3:

$$\frac{y_s}{y_1} = 2.0\ K_1\ K_2\ K_3\ K_4 \left(\frac{a}{y_1}\right)^{0.65} Fr_1^{0.43}$$

$$\frac{y_s}{2.84} = 2\ (1)\ (1)\ (1.1)\ (1) \left(\frac{1.52\ m}{2.84\ m}\right)^{0.65} (0.71)^{0.43}$$

$$\frac{y_s}{2.84} = 1.26$$

$$y_s = 3.6\ m$$

From the above computation the maximum local pier scour depth will be 3.6 m.

8.7.2 Correction for Angle of Attack

The above computation assumes that the piers are aligned with the flow (skew angles are less than 5°). However, if the piers were skewed to the flow by more than 5°, the value of y_s/y_1, as computed above, would need to be adjusted by K_2. The following computations illustrate the adjustment for piers skewed 10°.

$$\frac{L}{a} = \frac{12.2\ m}{1.52\ m} = 8$$

K_2 can then be obtained by using Equation 6.4 for an L/a of 8 and a 10° angle of attack. For this example, K_2=1.67. Applying this correction:

$$\frac{y_s}{2.84} = 1.67\ (1.26) = 2.1$$

$$y_s = 6.0\ m$$

Therefore, the maximum local pier scour depth for a pier angled 10° to the flow is 6.0 m.

8.7.3 Discussion of Pier Scour Computation

Although the estimated local pier scour would probably not occur at each pier, the possibility of thalweg shifting, which was identified in the Level 1 analysis, precludes setting the piers at different depths even if there were a substantial savings in cost. This is because any of the piers could be subjected to the worst-case scour conditions.

It is also important to assess the possibility of lateral migration of the channel. This possibility can lead to directing the flow at an angle to the piers, thus increasing local scour.

Countermeasures to minimize this problem could include riprap for the channel banks both up- and downstream of the bridge, and installation of guide banks to align flow through the bridge opening.

The possibility of lateral migration precludes setting the foundations for the overbank piers at a higher elevation. Therefore, in this example the foundations for the overbank piers should be set at the same elevation as the main channel piers.

8.8 STEP 6A: COMPUTE THE MAGNITUDE OF LOCAL SCOUR AT LEFT ABUTMENT

8.8.1 Computation of Abutment Scour Depth Using Froehlich's Equation

For spill-through abutments, K_1 =0.55. For this example, the abutments are set perpendicular to the flow; therefore, K_2=1.0. Abutment scour can be estimated using Froehlich's equation with data derived from the WSPRO output (Table 8.4).

The y_a value at the abutment is assumed to be the average flow depth in the overbank area. It is computed as the cross-sectional area of the left overbank cut off by the left abutment divided by the distance the left abutment protrudes into the overbank flow.

$$y_a = \frac{A_e}{L} = \frac{264.65\,m^2}{232.80\,m} = 1.14\,m$$

The average velocity of the flow in the left overbank (Figure 8.4) which is cut off by the left abutment is computed as the discharge cutoff by the abutment divided by the area of the left overbank cut off by the left abutment.

$$V_e = \frac{Q_e}{A_e} = \frac{148.68\,m^3/s}{264.65\,m^2} = 0.56\,m/s$$

Using these parameters, the Froude Number of the overbank flow is:

$$Fr = \frac{V_e}{(g\,y_a)^{1/2}} = \frac{0.56\,m/s}{[(9.81\,m/s^2)\,(1.14\ m)]^{0.5}} \tag{8.25}$$

Fr = 0.17

Using Froehlich's equation (Equation 7.1):

$$\frac{y_s}{y_a} = 2.27\,K_1\,K_2\,\left(\frac{L'}{y_a}\right)^{0.43}\,Fr^{0.61} + 1$$

$$\frac{y_s}{1.14} = 2.27\,(0.55)\,(1.0)\,\left(\frac{169.4}{1.56}\right)^{0.43}\,(0.17)^{0.61} + 1$$

$$\frac{y_s}{1.14\,m} = 4.64$$

$$y_s = 5.3\,m$$

Using Froehlich's equation, the abutment scour at the left abutment is computed to be 5.9 m.

8.8.2 Computation of Abutment Scour Depth Using the HIRE Equation

The HIRE equation for abutment scour is applicable for this situation because L/y_1 is greater than 25.

The HIRE equation is based on the velocity and depth of the flow passing through the bridge opening adjacent to the abutment end which is listed in Table 8.5. Therefore, the Froude Number of this flow is:

$$Fr_1 = \frac{1.29\,m/s}{[(9.81\,m/s^2)\,(0.83 m)]^{0.5}} = 0.45$$

Using the HIRE equation with K_1 = 0.55 and K_2 = 1.0 (Equation 7.2):

$$\frac{y_s}{0.83\ m} = 4\ Fr_1^{0.33} = 4\ (0.45)^{0.33} = 3.07$$

$$y_s = 2.6\ m$$

From the above computation, the depth of scour at the left abutment as computed using the HIRE equation, is 2.6 m.

8.9 STEP 6B: COMPUTE THE MAGNITUDE OF LOCAL SCOUR AT RIGHT ABUTMENT

The HIRE equation for abutment scour is also applicable for the right abutment since L/y_1 is greater than 25.

The HIRE equation is based on the velocity and depth of the flow passing through the bridge opening adjacent to the end of the right abutment and listed in Table 8.6. The Froude Number of this flow is:

$$Fr_1 = \frac{2.19\,m/s}{[(9.81\,m/s^2)\,(1.22\ m)]^{0.5}} = 0.63$$

Using the HIRE equation with K_1 = 0.55 and K_2 = 1.0:

$$\frac{y_s}{1.22\ \text{m}} = 4\ Fr_1^{0.33} = 4\ (0.63)^{0.33} = 3.43$$

$$y_s = 4.2\ \text{m}$$

From the above computation, the depth of scour at the right abutment, as computed using the HIRE equation is 4.2 m.

8.10 DISCUSSION OF ABUTMENT SCOUR COMPUTATIONS

Abutment scour as computed using the Froehlich equation[70] will generally result in deeper scour predictions than will be experienced in the field. These scour depths could occur if the abutments protruded into the main channel flow, or when a uniform velocity field is cut off by the abutment in a manner that most of the returning overbank flow is forced to return to the main channel at the abutment end. For most cases, however, when the overbank area, channel banks and area adjacent to the abutment are well vegetated, scour depths as predicted with the Froehlich equation will probably not occur.

All of the abutment scour computations (left and right abutments) assumed that the abutments were set perpendicular to the flow. If the abutments were angled to the flow, a correction utilizing K_2 would be applied to Froehlich's equation and to the equation from HDS 6.[22] However the adjustment for skewed abutments is minor when compared to the magnitude of the computed scour depths. For example, if the abutments for this example problem were angled 30° upstream ($\theta = 90^\circ + 30^\circ = 120^\circ$), the correction for skew would increase the computed depth of abutment scour by no more than 3 to 4 percent for the Froehlich and HIRE equation, respectively.

8.11 STEP 7: PLOT TOTAL SCOUR DEPTH AND EVALUATE DESIGN

As a final step, the results of the scour computations are plotted on the bridge cross section and carefully evaluated (Figure 8.6). For this example, only the computations for pier scour with piers aligned with the flow were plotted and the abutment scour computations reflect the results from the HIRE equation. The topwidth of the local scour holes is suggested as 2.0 times y_s.

It is important to evaluate carefully the results of the scour computations. For example, although the total scour plot indicates that the total scour at the overbank piers is less than for the channel piers, this does not indicate that the foundations for the overbank piers can be set at a higher elevation. Due to the possibility of channel and thalweg shifting, all of the piers should be set to account for the maximum total scour. Also, the computed contraction scour is distributed uniformly across the channel in Figure 8.6. However, in reality this may not be what would happen. With the flow from the overbank area returning to the channel, the contraction scour could be deeper at both abutments. The use of guide banks would distribute the contraction scour more uniformly across the channel. This would make a strong case for guide banks in addition to the protection they would provide to the abutments. The stream tube velocities could be used to distribute the scour depths across this section.

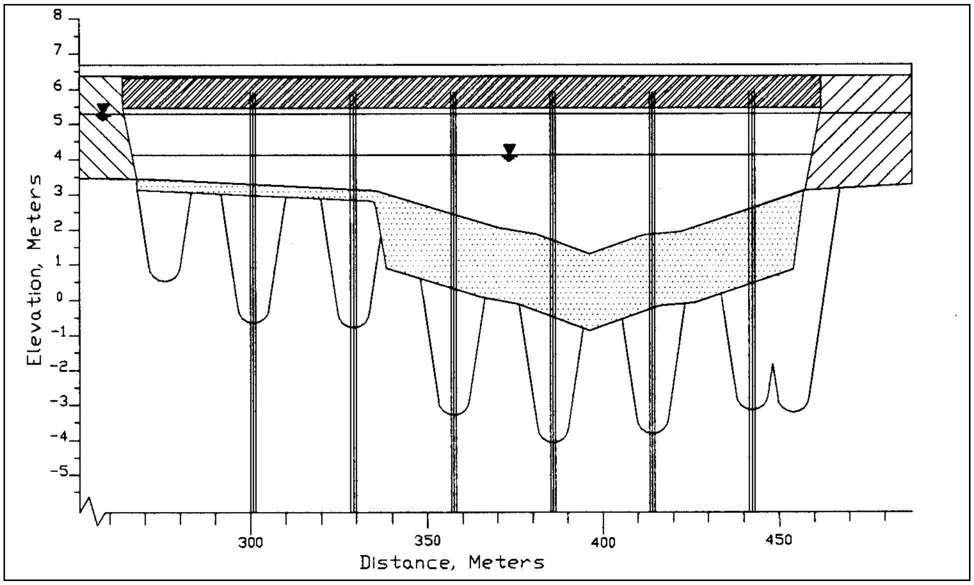

Figure 8.6. Plot of total scour for example problem.

The plot of the total scour also indicates that there is a possibility of overlapping scour holes between the sixth pier and right abutment, and it is not clear from where the right abutment scour should be measured, since the abutment is located at the channel bank. Both of these uncertainties should be avoided for replacement and new bridges whenever possible. Consequently, it would be advisable to set the right abutment back from the main channel. This would also tend to reduce the magnitude of contraction scour in the main channel.

The possibility of lateral migration of the channel will have an adverse effect on the magnitude of the pier scour. This is because lateral migration will most likely skew the flow to the piers. This problem can be minimized by using circular piers. An alternative approach would be to install guide banks to align the flow through the bridge opening.

A final concern relates to the location and depth of contraction scour in the main channel near the second pier and toe of the right abutment. At these locations, contraction scour in the main channel could increase the bank height to a point where bank failure and sloughing would occur. It is recommended that the existing bank lines be protected with revetment (i.e., riprap, gabions, etc.). Since the river has a history of channel migration, the bridge inspection and maintenance crews should be briefed on the nature of this problem so that any lateral migration can be identified.

The plot of the scour prism in Figure 8.6 should be reploted to show the potential for the scour to occur at any location in the bridge opening. This is shown in Figure 8.7

8.12 COMPLETE THE GENERAL DESIGN PROCEDURE

This design problem uses Steps 1 through 7 of the specific design approach (Chapter 2) and completes Steps 1 through 6 of the general design procedure in Chapter 2. The design must now proceed to Steps 7 and 8, which include bridge foundation analysis and consideration of the check for superflood. This is not done for this example problem.

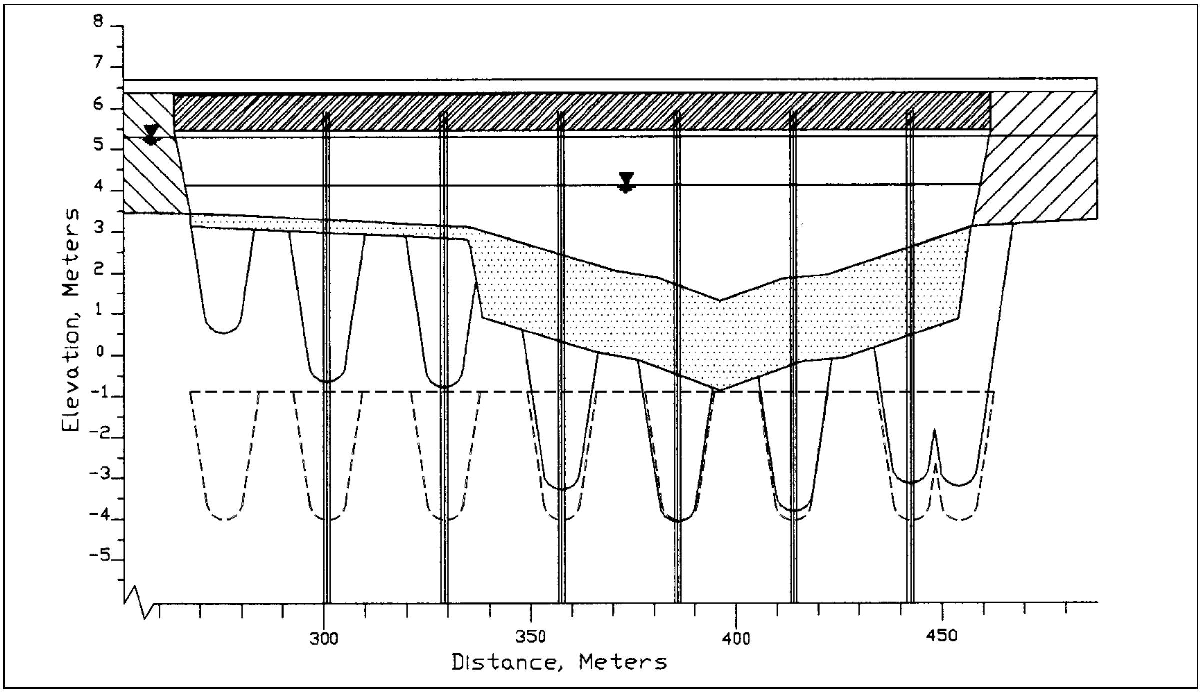

Figure 8.7. Revised plot of total scour for example problem.

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CHAPTER 9

SCOUR ANALYSIS FOR TIDAL WATERWAYS

9.1 INTRODUCTION

In the coastal region, scour at bridges over tidal waterways that are subjected to the effects of astronomical tides and storm surges is a combination of long-term degradation, contraction scour, local scour, and waterway instability. These are the same scour mechanisms that affect non-tidal (riverine) streams. Although many of the flow conditions are different in tidal waterways, the equations used to determine riverine scour are applicable if the hydraulic conditions (depth, discharge, velocity, etc.) are carefully evaluated.[23, 24]

This chapter presents methods and equations for determining stream stability and scour at tidal inlets, tidal estuaries, bridge crossings to islands and streams affected by tides (tidal waterways). Analysis of tidal waterways is very complex. The hydraulic analysis must consider the magnitude of the 100- and 500-year storm surge (storm tide - see Section 9.2 Glossary), the characteristics (geometry) of the tidal inlet, estuary, bay or tidal stream and the effect of any constriction of the flow due to the bridge. In addition, the analysis must consider the long-term effects of the normal tidal cycles on long-term aggradation or degradation, contraction scour, local scour, and stream instability. Coastal analyses require a synthesis of complex meteorological, bathymetric, geographical, statistical, and hydraulic disciplines and knowledge. The methods and equations presented in this chapter provide an overview of application of these elements in the context of tidal scour analyses.

A storm tide or storm surge in coastal waters results from astronomical tides, wind action, and rapid barometric pressure changes. In addition, the change in elevation resulting from the storm surge may be increased by resonance in harbors and inlets, whereby, the tidal range in an estuary, bay, or inlet is larger than on the adjacent coast.

The astronomical tidal cycle with reversal in flow direction can increase long-term degradation, contraction scour, and local scour. If sediment is being moved on the flood and ebb tide, there may be no net loss of sediment in a bridge reach because sediments are being moved back and forth. Consequently, no net long-term degradation may occur. However, local scour at piers and abutments can occur at both the inland and ocean side of the piers and abutments and will alternate with the reversal in flow direction. If, however, there is a loss of sediment in one or both flow directions, there will then be long-term degradation in addition to local scour. Also, the tidal cycles may increase bank erosion, migration of the channel, and thus, increase stream instability.

The complexity of the hydraulic analysis increases if the tidal inlet or the bridge constrict the flow and affect the amplitude of the storm surge (storm tide) in the bay or estuary so that there is a large change in elevation between the ocean and the estuary or bay. A constriction in the tidal inlet can increase the velocities in the constricted waterway opening, decrease interior wave heights and tidal range, and increase the phase difference (time lag) between exterior and interior water levels. Analysis of a constricted inlet or waterway may require the use of an orifice equation rather than tidal relationships.

For the analysis of bridge crossings of tidal waterways, a three-level analysis approach similar to the approach outlined in HEC-20 is suggested.[6] Level 1 includes a qualitative evaluation of the stability of the inlet or estuary, estimating the magnitude of the tides, storm surges, and flow in the tidal waterway, and attempting to determine whether the hydraulic analysis depends on tidal or river conditions, or both. **Level 2** represents the engineering analysis necessary to obtain the velocity, depths, and discharge for tidal waterways to be used in determining long-term aggradation, degradation, contraction scour, and local scour. The hydraulic variables obtained from the Level 2 analysis are used in the riverine equations presented in previous chapters to obtain total scour. Using these riverine scour equations, which are for steady-state equilibrium conditions for unsteady, dynamic tidal flow may result in estimating deeper scour depths than will actually occur (conservative estimate), but this represents the state of knowledge at this time for this level of analysis.

For complex tidal situations, **Level 3** analysis using physical and 2-dimensional computer models may be required. This section will be limited to a discussion of Levels 1 and 2 analyses. In Level 2 analyses, unsteady 1-dimensional or quasi 2-dimensional computer models may be used to obtain the hydraulic variables needed for the scour equations. **The Level 1, 2, and 3 approaches are described in more detail in later sections.**

The steady-state equilibrium scour equations given in previous sections of this manual are suitable for use to determine scour depths in tidal flows. As mentioned earlier, tidal flows resulting from storm surges are unsteady but no more so than most unsteady riverine flows. For both cases, scour depths are conservative.

9.2 OVERVIEW OF TIDAL PROCESS

9.2.1 Glossary

Bay A body of water connected to the ocean with an inlet.

Diurnal tide Tides with an approximate tidal period of 24 hours.

Ebb or ebb tide Flow of water from the bay or estuary to the ocean.

Estuary Tidal reach at the mouth of a river.

Flood or flood tide Flow of water from the ocean to the bay or estuary.

Littoral transport or drift Transport of beach material along a shoreline by wave action. Also, longshore sediment transport.

Run-up, wave Height to which water rises above still-water elevation when waves meet a beach, wall, etc.

Semi-diurnal tide Tides with an approximate tidal period of 12 hours.

Set-up, wave Height to which water rises above still-water elevation as a result of storm wind effects.

Still-water elevation Flood height to which water rises as a result of barometric pressure changes occurring during a storm event.

Storm surge Coastal flooding phenomenon resulting from wind and barometric changes. The storm surge is measured by subtracting the astronomical tide elevation from the total flood elevation (Hurricane surge).

Storm tide Coastal flooding resulting from combination of storm surge and astronomical tide (often referred to as storm surge)

Tidal amplitude Generally, half of tidal range.

Tidal cycle One complete rise and fall of the tide.

Tidal day Time of rotation of the earth with respect to the moon. Assumed to equal approximately 24.84 solar hours in length.

Tidal inlet A channel connecting a bay or estuary to the ocean.

Tidal passage A tidal channel connected with the ocean at both ends.

Tidal period Duration of one complete tidal cycle. When the tidal period equals the tidal day (24.84 hours), the tide exhibits diurnal behavior. Should two complete tidal periods occur during the tidal day, the tide exhibits semi-diurnal behavior.

Tidal prism Volume of water contained in a tidal bay, inlet or estuary between low and high tide levels.

Tidal range Vertical distance between specified low and high tide levels.

Tidal waterways A generic term which includes tidal inlets, estuaries, bridge crossings to islands or between islands, inlets to bays, crossings between bays, tidally affected streams, etc.

Tides, astronomical Rhythmic diurnal or semi-diurnal variations in sea level that result from gravitational attraction of the moon and sun and other astronomical bodies acting on the rotating Earth.

Tsunami Long-period ocean wave resulting from earthquake, other seismic disturbances or submarine land slides.

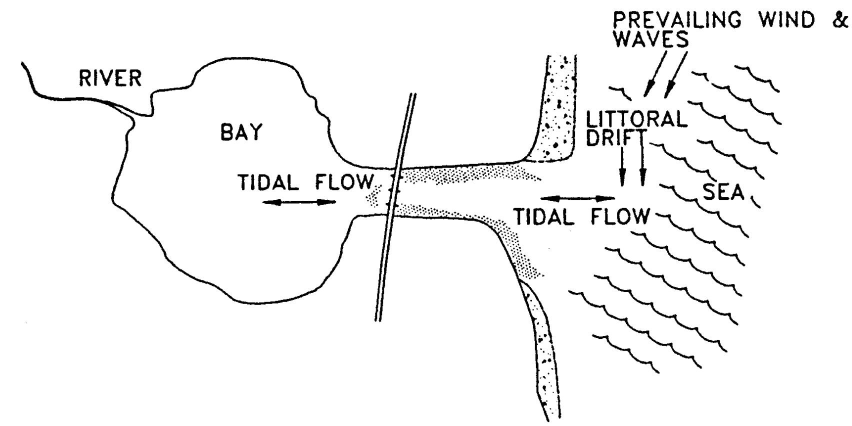
Waterway opening Width or area of bridge opening at a specific elevation, measured normal to principal direction of flow.

Wave period Time interval between arrivals of successive wave crests at a point.

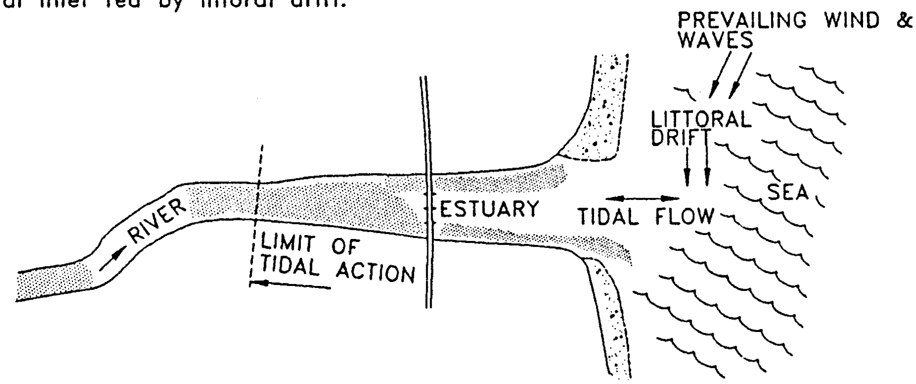
9.2.2 Definition of Tidal and Coastal Processes

Typical bridge crossings of tidal waterways are sketched in Figure 9.1. From this figure, tidal flows can be defined as being between the ocean and a bay (or lagoon), from the ocean into an estuary, or through passages between islands.

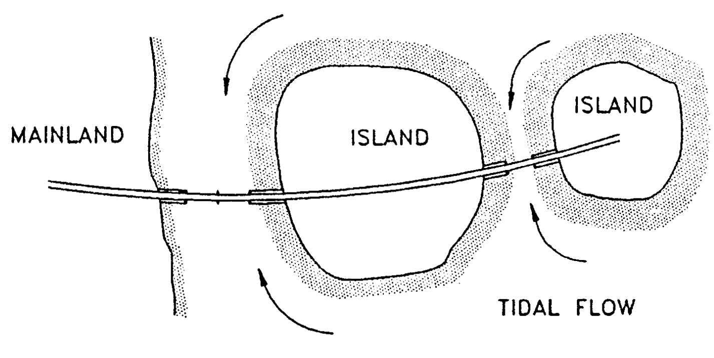
Flow into (flood tide) and out of (ebb tide) a bay or estuary is driven by tides and by the discharge into the bay or estuary from upland areas. Assuming that the flow from upland areas is negligible, the ebb and flood in the bay or estuary will be driven solely by tidal fluctuations and storm surges as illustrated in Figure 9.2. With no inflow of water from rivers and streams, the net flow of water into and out of the bay or estuary will be nearly zero. Increasing the discharge from rivers and streams will lead to a net outflow of water to the ocean.



1. Inlets between the open sea and an enclosed lagoon or bay, where most of the discharge results from tidal flows. Tidal inlet fed by littoral drift.



2. River estuaries where the net discharge comprises river flow as well as tidal flow components



3. Passages between islands, or between an island and the mainland, where a route to the open sea exists in both directions.

Figure 9.1. Types of tidal waterway crossings (after Neill).[78]

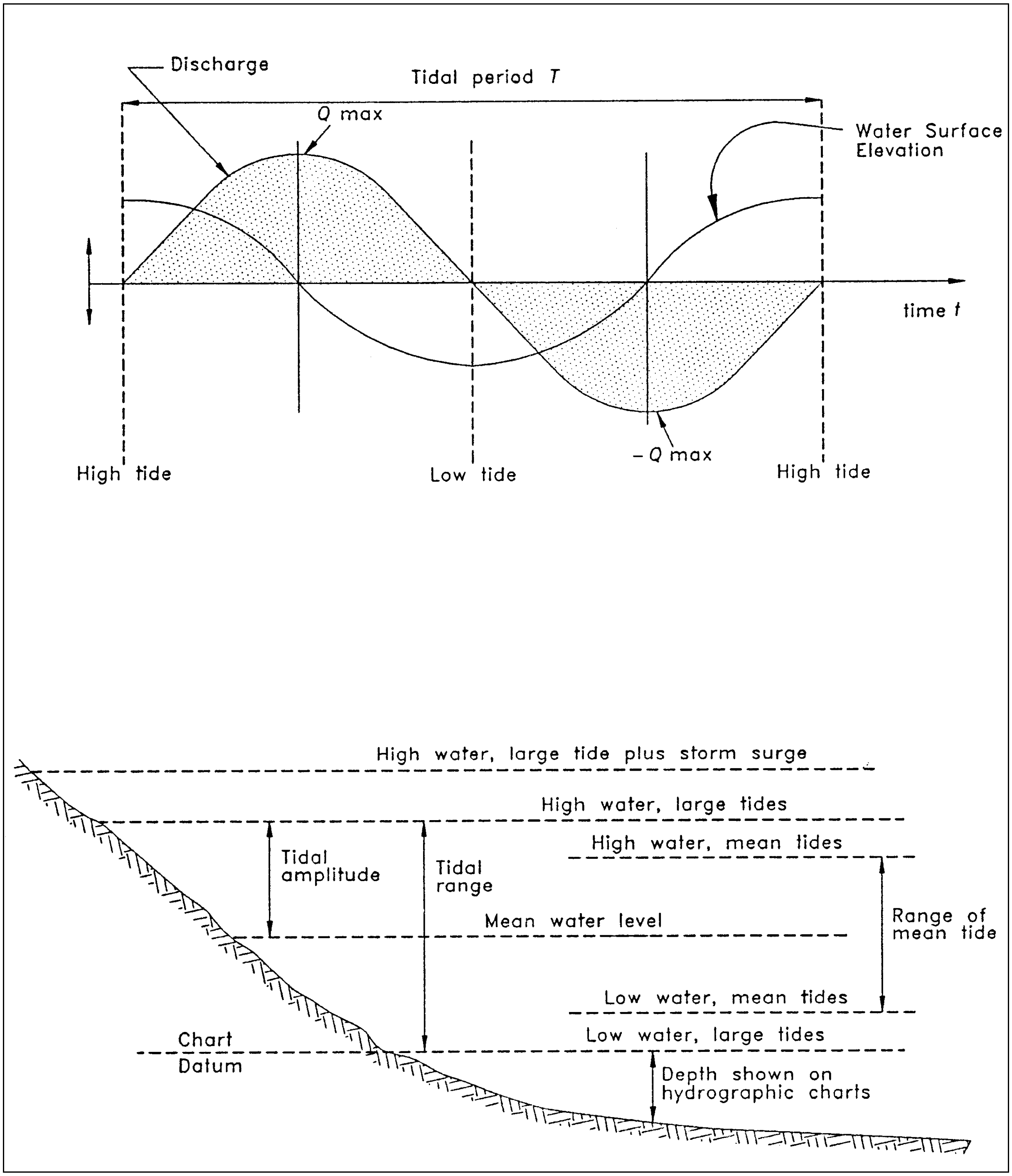


Figure 9.2. Principal tidal terms (after Neill).[78]

Figure 9.2 illustrates the elevation and time variable nature of astronomical tides. For astronomical tides, maximum flood and ebb (or the time of maximum current and discharge) can be assumed to occur at the inflection point of (or halfway between) high tide and low tide, but actually can occur before or after the midtide level depending on the location. The addition of a storm surge to a high astronomical tide can lead to additional water surface elevations (High water, large tide plus storm surge in Figure 9.2), additional current, and associated flooding.

In the most conservative scenario, the greatest potential flood elevation would occur at the time where the high astronomical tide and maximum storm surge height coincide in time. In this circumstance, the maximum discharge would occur when the astronomical tidal period and the period associated with the storm surge event are the same value. The presence of any inland flood discharge would influence this discharge, particularly during the period when the flood levels recede (ebb).

Hydraulically, the above discussion presents two limiting cases for evaluation of the flow velocities in the bridge reach. With negligible flow from the upland areas, the flow through the bridge opening is based solely on the ebb and flood resulting from tidal fluctuations or storm surges. Alternatively, when the flow from the streams and rivers draining into the bay or estuary (inland flood) is large in relationship to the tidal flows (ebb and flood tide), the effects of tidal fluctuations are negligible. For this latter case, the evaluation of the hydraulic characteristics and scour can be accomplished using the methods described in previous chapters for inland rivers.

Bridge scour in the coastal region results from the unsteady diurnal and semi-diurnal flows resulting from astronomical tides, large flows that can result from storm surges (hurricanes, nor'easters), and the combination of riverine and tidal flows. The forces which drive tidal fluctuations are, primarily, the result of the gravitational attraction of the sun and moon on the rotating earth (astronomical tides), wind and storm setup, and geologic disturbances (tsunamis). These different forces which drive tides produce varying tidal periods and amplitudes. In general semi-diurnal astronomical tides having tidal periods of approximately 12 hours occur in the lower latitudes while diurnal tides having tidal periods of approximately 24 hours occur in the higher latitudes. Typically, the storm surge period correlates with the associated storm type. Hurricane surges generally last from 12 to 15 hours. Nor'easters may produce a storm surge lasting several days. In general, storm surge periods may be assumed to be longer than astronomical tidal periods.

The continuous rise and fall of astronomical tides will usually influence long-term trends of aggradation or degradation, contraction and local scour. Worst-case hydraulic conditions for contraction and local scour are usually the result of infrequent tidal events such as storm surges and tsunamis. Storm surges and tsunamis are a single event phenomenon which, due to their magnitude, can present a significant threat to a bridge crossing in terms of scour. The hydraulic variables (discharge, velocity, and depths) and bridge scour in the coastal region can be determined with as much precision as riverine flows. These determinations are conservative and research is needed for both cases to improve scour determinations. Determining the magnitude of the combined flows can be accomplished by simply adding riverine flood flow to the maximum tidal flow, if the drainage basin is small, or routing the design riverine flows to the crossing and adding them to the storm surge flows.

The small size of the bed material (normally fine sand) as well as silts and clays with cohesion and littoral drift (transport of beach sand along the coast resulting from wave action) affect the magnitude of bridge scour. Mass density stratification of the water typically has a minor influence on bridge scour. Peak flows from storm surges may not have durations long enough to reach the ultimate scour depths determined from existing scour equations. Sediment transport equations can be used to compute the rate of contraction scour (see Section 9.6), but the time dependent characteristics of local scour require further research. Diurnal and semi-diurnal astronomical tides can cause long-term degradation if there is no source of sediment except at the crossing. At some locations, this has resulted in long-term degradation of 0.3 to 1.0 m (1.0 to 3.3 ft) per year with no indication of stopping.[79, 80] Existing scour equations can predict the magnitude of this scour, but not the time history.[23, 24]

Mass density stratification (saltwater wedges), which can result when the denser more saline ocean water enters an estuary or tidal inlet with significant freshwater inflow, can result in larger velocities near the bottom than the average velocity in the vertical velocity profile. With careful evaluation, the correct velocity can be determined for use in the scour equations. With storm surges, mass density stratification will not normally occur. The density difference between salt and freshwater, except as it causes saltwater wedges, is not significant enough to affect scour equations. Density and viscosity differences between fresh and sediment-laden water can be much larger in riverine flows than the density and viscosity differences between salt and freshwater.

Salinity can affect the transport of silts and clays by causing them to flocculate and possibly deposit, which may affect stream stability and must be evaluated. Salinity may affect the erodibility of cohesive sediments, but this will only affect the rate of scour, not ultimate scour. Littoral drift is a source of sediment to a tidal waterway.[81, 82] An aggrading or stable waterway may exist if the supply of sediment to the bridge from littoral drift is large. This will have the effect of minimizing contraction scour, and possibly local scour. Conversely, long-term degradation, contraction scour and local scour can be exacerbated if the sediment from littoral drift is reduced or cut off. Evaluating the effect of littoral drift is a sediment transport problem involving historical information, future plans (dredging, jetties, etc.) for the waterway and/or the coast, sources of sediment, and other factors.

Evaluation of total scour at bridges crossing tidal waterways requires the assessment of long-term aggradation or degradation, local scour and contraction scour. Long-term aggradation or degradation estimates can be derived from a geomorphic evaluation coupled with computations of live-bed contraction scour if sediment transport is changed.

Although the hydraulics of flow for tidal waterways is complicated by the presence of two directional flow, the basic concept of sediment continuity is valid. Consequently, a clear understanding of the principle of sediment continuity is essential for evaluating scour at bridges spanning waterways influenced by tidal fluctuations. Technically, the sediment continuity concept states that the sediment inflow minus the sediment outflow equals the time rate of change of sediment volume in a given reach. More simply stated, during a given time period the amount of sediment coming into the reach minus the amount leaving the downstream end of the reach equals the change in the amount of sediment stored in that reach.

As with riverine scour, tidal scour can be characterized by either live-bed or clear-water conditions. In the case of live-bed conditions, sediment transported into the bridge reach will tend to reduce the magnitude of scour. Whereas, if no sediment is in transport to re-supply the bridge reach (clear-water), scour depths can be larger.

In addition to sediments being transported from inland areas, sediments are transported parallel to the coast by ocean currents and wave action. This littoral transport of sediment serves as a source of sediment supply to the inlet, bay or estuary, or tidal passage. During the flood tide, these sediments can be transported into the bay or estuary and deposited.

During the ebb tide, these sediments can be re-mobilized and transported out of the inlet or estuary and either be deposited on shoals or moved further down the coast as littoral transport (Figure 9.3).

Sediment transported to the bay or estuary from the inland river system can also be deposited in the bay or estuary during the flood tide, and re-mobilized and transported through the inlet or estuary during the ebb tide. However, if the bay or estuary is large, sediments derived from the inland river system can deposit in the bay or estuary in areas where the velocities are low and may not contribute to the supply of sediment to the bridge crossing. The result is clear-water scour unless sediment transported on the flood tide (ocean shoals, littoral transport) is available on the ebb. Sediments transported from inland rivers into an estuary may be stored there on the flood and transported out during ebb tide. This would produce live-bed scour conditions unless the sediment source in the estuary was disrupted. Dredging, jetties or other coastal engineering activities can limit sediment supply to the reach and influence live-bed and clear-water conditions.

Application of sediment continuity involves understanding the hydraulics of flow and availability of sediment for transport. For example, a net loss of sediment in the inlet, bay or tidal estuary could be the result of cutting off littoral transport by means of a jetty projecting into the ocean (Figure 9.3). For this scenario, the flood tide would tend to erode sediment from the inlet and deposit sediment in the bay or estuary while the ensuing ebb tide would transport sediment out of the bay or estuary. Because the availability of sediment for transport into the bay is reduced, degradation of the inlet could result. As discussed later, as the cross sectional area of the inlet increases, the flow velocities during the flood tide increase, resulting in further degradation of the inlet. This can result in an unstable inlet which continues to enlarge as a result of sediment supply depletion.

From the above discussion, it is clear that the concept of sediment continuity provides a valuable tool for evaluation of aggradation or degradation trends of a tidal waterway. Although this principle is not easy to quantify without direct measurement or hydraulic and sediment continuity modeling, the principle can be applied in a qualitative sense to assess long-term trends in aggradation or degradation.

9.3 LEVEL 1 ANALYSIS

The objectives of a Level 1 qualitative analysis are to determine the magnitude of the tidal effects on the crossing, the overall long-term stability of the crossing (vertical and lateral stability) and the potential for waterway response to change.

The first step in evaluation of highway crossings is to determine whether the bridge crosses a river which is influenced by tidal fluctuations (tidally affected river crossing) or whether the bridge crosses a tidal inlet, bay or estuary (tidally controlled). The flow in tidal inlets, bays and estuaries is predominantly driven by tidal fluctuations (with flow reversal), whereas, the flow in tidally affected river crossings is driven by a combination of river flow and tidal fluctuations. Therefore, tidally affected river crossings are not subject to flow reversal but the downstream tidal fluctuation acts as a cyclic downstream control. Tidally controlled river crossings will exhibit flow reversal.

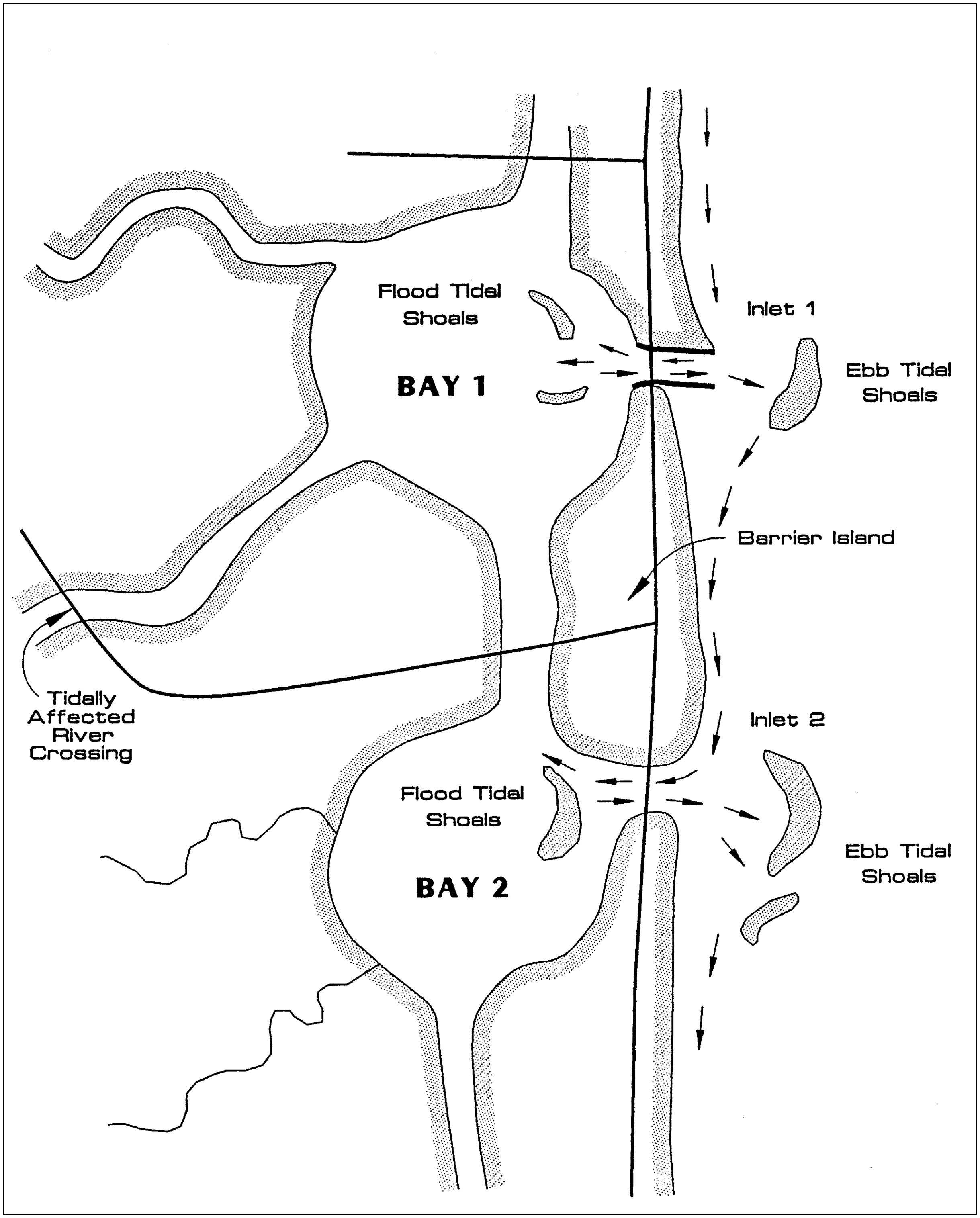


Figure 9.3. Sediment transport in tidal inlets (after Sheppard).[81]

9.3.1 Tidally Affected River Crossings

Tidally affected river crossings are characterized by both river flow and tidal fluctuations. From a hydraulic standpoint, the flow in the river is influenced by tidal fluctuations which result in a cyclic variation in the downstream control of the tail water in the river estuary. The degree to which tidal fluctuations influence the discharge at the river crossing depends on such factors as the relative distance from the ocean to the crossing, riverbed slope, cross-sectional area, storage volume, and hydraulic resistance. Although other factors are involved, relative distance of the river crossing from the ocean can be used as a qualitative indicator of tidal influence. At one extreme, where the crossing is located far upstream, the flow in the river may only be affected to a minor degree by changes in tailwater control due to tidal fluctuations. As such, the tidal fluctuation downstream will result in only minor fluctuations in the depth, velocity, and discharge through the bridge crossing.

As the distance from the crossing to the ocean is reduced, again assuming all other factors as equal, the influence of the tidal fluctuations increases. Consequently, the degree of tail water influence on flow hydraulics at the crossing increases. A limiting case occurs when the magnitude of the tidal fluctuations is large enough to reduce the discharge through the bridge crossing to zero at high tide. River crossings located closer to the ocean than this limiting case have two directional flows at the bridge crossing, and because of the storage of the river flow at high tide, the ebb tide will have a larger discharge and velocities than the flood tide.

For the Level 1 analysis, it is important to evaluate whether the tidal fluctuations will significantly affect the hydraulics at the bridge crossing. If the influence of tidal fluctuations is considered to be negligible, then the bridge crossing can be evaluated based on the procedures outlined for inland river crossings presented previously in this document. If not, then the hydraulic flow variables must be determined using dynamic tidal flow relationships. This evaluation should include extreme events such as the influence of storm surges and inland floods.

From historical records of the stream at the highway crossing, determine whether the worst-case conditions of discharge, depths and velocity at the bridge are the 100- and 500-year return period tide and storm surge, or the 100- and 500-year inland flood or a combination of the two. Historical records could consist of tidal and stream flow data from Federal Emergency Management Agency (FEMA), National Oceanic and Atmospheric Administration (NOAA), USACE, and USGS records; aerial photographs of the area; maintenance records for the bridge or bridges in the area; newspaper accounts of previous high tides and/or flood flows; and interviews in the local area.

If the primary hazard to the bridge crossing is from inland flood events, then scour can be evaluated using the methods given previously in this circular and in HEC-20.[(6)] If the primary hazard to the bridge is from tide and storm surge or tide, storm surge and inland flood runoff, then use the analyses presented in the following sections on tidal waterways. If it is unclear whether the worst hazard to the bridge will result from a storm surge, maximum tide, or from an inland flood, it may be necessary to evaluate scour considering each of these scenarios and compare the results.

9.3.2 Tidal Inlets, Bays, and Estuaries

For tidal inlets, bays and estuaries, the goal of the Level 1 analysis is to determine the stability of the inlet and identify and evaluate long-term trends at the location of the highway crossing. This can be accomplished by careful evaluation of present and historical conditions of the tidal waterway and anticipating future conditions or trends.

Existing cross-sectional and sounding data can be used to evaluate the stability of the tidal waterway at the highway crossing and to determine whether the inlet, bay or estuary is increasing or decreasing in size, or is relatively stable. For this analysis it is important to evaluate these data based on past and current trends. The data for this analysis could consist of aerial photographs, cross section soundings, location of bars and shoals on both the ocean and bay sides of an inlet, magnitude and direction of littoral drift, and longitudinal elevations through the waterway. It is also important to consider the possible impacts (either past or future) of the construction of jetties, breakwaters, or dredging of navigation channels.

Sources of data would be USACE, FEMA, USGS, U.S. Coast Guard (USCG), NOAA, local Universities, oceanographic institutions and publications in local libraries. For example, a publication by Bruun, "Tidal Inlets and Littoral Drift" contains information on many tidal inlets on the east coast for the United States.[82]

A site visit is recommended to gather such data as the conditions of the beaches (ocean and bay side); location and size of any shoals or bars; direction of ocean waves; magnitude of the currents in the bridge reach at mean water level (midway between high and low tides); and size of the sediments. Sounding the channel both longitudinally and in cross section using a conventional "fish finder" sonic fathometer is usually sufficiently accurate for this purpose.

Observation of the tidal inlet to identify whether the inlet restricts the flow of either the incoming or outgoing tide is also recommended. If the inlet or bridge restricts the flow, there will be a noticeable drop in head (change in water surface elevation) in the channel during either the ebb or flood tide. If the tidal inlet or bridge restricts the flow, an orifice equation may need to be used to determine the maximum discharge, velocities and depths (see the Level 2 analysis of this section).

Velocity measurements in the tidal inlet channel along several cross sections, several positions in the cross section and several locations in the vertical can also provide useful information for verifying computed velocities. Velocity measurements should be made at maximum discharge (Q_{max}). Maximum discharge usually occurs around the midpoint in the tidal cycle between high and low tide (Figure 9.2), although constricted inlets usually cause peak discharge to occur closer to high and low tides.

The velocity measurements can be made from a boat or from a bridge located near the site of a new or replacement bridge. If a bridge exists over the channel, a recording velocity meter could be installed to obtain measurements over several tidal cycles. Currently, there are instruments available that make velocity data collection easier. For example, broad-band acoustic Doppler current profiles and other emerging technologies will greatly improve the ability to obtain and use velocity data.

In order to develop adequate hydraulic data for the evaluation of scour, it is recommended that recording water level gages located at the inlet, at the proposed bridge site and in the bay or estuary upstream of the bridge be installed to record tide elevations at 15-minute intervals for several full tidal cycles. This measurement should be conducted during one of the spring tides where the amplitude of the tidal cycle will be largest. The gages should be referenced to the same datum and synchronized. The data from these recording gages are necessary for calibration of tidal hydraulic models such as ACES-INLET[83], or other unsteady 1 or 2-dimensional hydraulic flow models such as UNET, FESWMS-2D, and RMA-2V.[84,45,85,86] These data are also useful for calibration of WSPRO or HEC River Analysis System (RAS) when the bridge crosses tidally affected channels.[15,16,17] A more complete description of the unsteady flow models and data requirements for model application are given in Section 9.4.7.

The data and evaluations suggested above can be used to estimate whether present conditions are likely to continue into the foreseeable future and as a basis for evaluating the hydraulics and total scour for the Level 2 analysis. A stable inlet could change to one which is degrading if the channel is dredged or jetties are constructed on the ocean side to improve the entrance, since dredging or jetties could modify the supply of sediment to the inlet. In addition, plans or projects which might interrupt existing conditions of littoral drift should be evaluated.

It should be noted that in contrast to an inland river crossing, the discharge at a tidal inlet is not fixed. In inland rivers, the design discharge is fixed by the runoff and is virtually unaffected by the waterway opening. In contrast, the discharge at a tidal inlet can increase as the area of the tidal inlet increases, thus increasing long-term aggradation or degradation and local scour. Also, as Neill points out, constriction of the natural waterway opening may modify the tidal regime and associated tidal discharge.[78]

9.4 LEVEL 2 ANALYSIS

9.4.1 Introduction

Level 2 analysis involves the basic engineering assessment of scour problems at highway crossings. Scour equations developed for inland rivers are recommended for use estimating and evaluating scour for tidal flows. However, in contrast to the evaluation of scour at inland river crossings, the evaluation of the hydraulic conditions at the bridge crossing using either WSPRO or HEC-RAS is only suitable for tidally affected crossings where tidal fluctuations result in a variable tailwater control without flow reversal.[15, 16, 17] Other methods, described in this chapter, are recommended for tidally affected and tidally controlled crossings where the tidal fluctuation has a significant influence on the tidal hydraulics.

Several methods to obtain hydraulic characteristics of tidal flows at the bridge crossing are available. These range from simple procedures to more complex 2-dimensional and quasi 2-dimensional unsteady flow models. The use of the simpler hydraulic procedures is discussed and illustrated with example problems in Sections 9.8 and 9.9. An overview of the unsteady flow models which are suitable for modeling tidal hydraulics at bridge crossings is presented in Section 9.5. The use of the simpler hydraulic procedures given in this section can give large values if their underlying assumptions are violated. In these cases, 1- and 2-dimensional computer models can give more realistic values.

9.4.2 Evaluation of Hydraulic Characteristics

The velocity, depth and discharge at the bridge waterway are the most significant variables for evaluating bridge scour in tidal waterways. Direct measurements of the value of these variables for the design storm are seldom available. Therefore, it is usually necessary to develop the hydraulic and hydrographic characteristics of the tidal waterway, estuary or bay, and calculate the discharge, velocities, and depths in the crossing using coastal engineering equations. These values can then be used in the scour equations given in previous sections to calculate long-term aggradation or degradation, contraction scour, and local scour.

Unsteady flow computer models were evaluated under a pooled fund research project administered by the South Carolina Department of Transportation (SCDOT).[(87)] The purpose of this study was to identify the most promising unsteady tidal hydraulic models for use in scour analyses. The study identified UNET, FESWMS-2D, and RMA-2V as being the most applicable for scour analysis.[(84,45,85,86)] The research funded by the South Carolina pooled fund project is being continued to enhance and adapt the selected models so that they are better suited to the assessment of scour at bridges.

The models recommended by the pooled fund study differ in terms of their capabilities, degree of complexity, applicability and method of numerical modeling. UNET is supported by the USACE.[(84)] UNET is a 1-dimensional unsteady flow model and is applicable to channel networks. FESWMS-2D is an unsteady 2-dimensional finite element model developed by the USGS with support from the FHWA.[(45)] FESWMS-2D can be used for steady and unsteady flow analyses and incorporates structure hydraulics. RMA-2V is a 2-dimensional finite element hydrodynamic model that can be used for steady or unsteady flow analyses.[(85, 86)] FESWMS-2D and RMA-2V can also incorporate surface stress due to wind.

Although these unsteady flow models are suitable for determining the hydraulic conditions, their use requires careful application and calibration. The effort required to utilize these models may be more than is warranted for many tidal situations. As such, the use of these models may be more applicable under a Level 3 analysis. However, these models could be used in the context of a Level 2 analysis, if deemed necessary, to better define the hydraulic conditions at the bridge crossing.

Alternatively, either a procedure by Neill for unconstricted waterways, or an orifice equation for constricted tidal inlets can be used to evaluate the hydraulic conditions at bridges influenced by tidal flows.[(78)] A step-wise procedure for using these two methods to determine hydraulic conditions and scour is presented in the following sections. The selection of which procedure to use depends on whether or not the inlet is constricted. In general, narrow inlets to large bays as illustrated in Figure 9.1 can usually be classified as constricted; whereas, estuaries, which are also depicted on Figure 9.1 can be classified as unconstricted. However, these guidelines should not be construed as absolute.

The procedure developed by Neill can be used for unconstricted tidal inlets.[(78)] This method, which assumes that the water surface in the tidal prism is level, and the basin has vertical sides, can be used for locations where the boundaries of the tidal prism can be well defined and where heavily vegetated overbank areas or large mud flats represent only a small portion of the inundated area. Thick vegetation tends to attenuate tide levels due to friction loss, thereby violating the basic assumption of a level tidal prism. The discharges and velocities may be over estimated using this procedure if vegetation will attenuate tidal levels. In some complex cases, a simple tidal routing technique or 2-dimensional flow models may need to be used instead of this procedure (see Section 9.5).

Observation of an abrupt difference in water surface elevation during the normal ebb and flow (astronomical tide) at the inlet (during a Level 1 analysis) is a clear indication that the inlet is constricted. However, the observation of no abrupt change in water surface during astronomical tidal fluctuations does not necessarily indicate that the inlet will be unconstricted when extreme events such as a storm surge occur. In some cases, it may be necessary to compute the tidal hydraulics using both tidal prism and orifice procedures.

Then, judgment should be used to select the worst appropriate hydraulic parameters for the computation of scour.

Velocity measurements made at the bridge site (see Level 1) can be useful in determining whether or not the inlet is constricted as well as for calibration or verification of the tidal computation procedure. Using tidal data at the time that velocity measurements were collected, computed flow depths, velocities and discharge can be compared and verified to measured values. This procedure can form a basis for determining the most appropriate hydraulic computation procedure and for adjusting the parameters in these procedures to better model the tidal flows.

9.4.3 Design Storm and Storm Tide

Normally, long-term aggradation or degradation at a tidal inlet or estuary are influenced primarily by the periodic tidal fluctuations associated with astronomical tides. Therefore, flow hydraulics at the bridge should be determined considering the tidal range as depicted in Figure 9.2 for evaluation of long-term aggradation or degradation.

Extreme events associated with inland floods and storm tides should be used to determine the hydraulics at the bridge to evaluate local and contraction scour. Typically, events with a return period corresponding to the 100- and 500-year storm tide and inland flood need to be considered. Difficulty arises in determining whether the storm tide, inland flood or the combination of storm tide and inland flood should be considered controlling. The effect of the inland flood discharges (if any), would be most significant during the period when storm tide floodwaters recede (ebb), as those discharges would likely add to, and increase the storm tide associated discharges.

When inland flood discharges are small in relationship to the magnitude of the storm tide and are the result of the same storm event, then the flood discharge can be added to the discharge associated with the design tidal flow, or the volume of the runoff hydrograph can be added to the volume of the tidal prism. If the inland flood and the storm tide may result from different storm events, then, a joint probability approach may be warranted to determine the magnitude of the 100- and 500-year flows.

In some cases there may be a time lag between the storm tide discharge and the stream flow discharge at the bridge crossing. For this case, stream flow-routing methods such as the USACE HEC-1 model can be used to estimate the timing of the flood hydrograph derived from runoff of the watersheds draining into the bay or estuary.[88]

For cases where the magnitude of the inland flood is much larger than the magnitude of the storm tide, evaluation of the hydraulics reduces to using the equations and procedures recommended for inland rivers. The selection of the method to use to combine inland flood and storm tide flows is a matter of judgment and must consider the characteristics of the site and the storm events.

9.4.4 Scour Evaluation Concepts

The total scour at a bridge crossing can be evaluated using the scour equations recommended for inland rivers and the hydraulic characteristics determined using the procedures outlined in the previous sections. However, it should be emphasized that the scour equations and subsequent results need to be carefully evaluated considering other (Level 1) information from the existing site, other bridge crossings, or comparable tidal waterways or tidally affected streams in the area.

Evaluation of long-term aggradation or degradation at tidal highway crossings, as with inland river crossings, relies on a careful evaluation of the past, existing and possible future condition of the site. This evaluation is outlined under Level 1 and should consider the principles of sediment continuity. A longitudinal sonic sounder survey of a tide inlet is useful to determine if bed material sediments can be supplied to the tidal waterway from the bay, estuary or ocean. When available, historical sounding data should also be used in this evaluation. Factors which could limit the availability of sediment should also be considered.

Over the long-term in a stable tidal waterway, the quantity of sediment being supplied to the waterway by ocean currents, littoral transport and inland flows and being transported out of the tidal waterway are nearly the same. If the supply of sediment is reduced either from the ocean or from the bay or estuary, a stable waterway can be transformed into a degrading waterway. In some cases, the rate of long-term degradation has been observed to be large and deep. An estimate of the maximum depth that this long-term degradation can achieve can be made by employing the clear-water contraction scour equations to the inlet. For this computation the flow hydraulics should be developed based on the range of mean tide as described in Figure 9.2. It should be noted that the use of this equation would provide an estimate of the worst case long-term degradation which could be expected assuming no sediments were available to be transported to the tidal waterway from the ocean or inland bay or estuary. As the waterway degrades, the flow conditions and storage of sediments in shoals will change, ultimately developing a new equilibrium. The presence of scour resistant rock would also limit the maximum long-term degradation.

Potential contraction scour for tidal waterways also needs to be carefully evaluated using hydraulic characteristics associated with the 100- and 500-year storm surge or inland flood as described in the previous section. For highway crossings of estuaries or inlets to bays, where either the channel narrows naturally or where the channel is narrowed by the encroachment of the highway embankments, the live-bed or clear water contraction scour equations can be utilized to estimate contraction scour.

Soil boring or sediment data are needed in the waterway upstream, downstream, and at the bridge crossing in order to determine if the scour is clear-water or live-bed and to support scour calculations if clear-water contraction scour equations are used. Equation 5.1 and the ratio of V_*/ω can be used to assess whether scour would be clear-water or live-bed.

A mitigating factor which could limit contraction scour concerns sediment delivery to the inlet or estuary from the ocean due to the storm surge and inland flood. A surge may transport large quantities of sediment into the inlet or estuary during the flood tide. Likewise, inland floods can also transport sediment to an estuary during extreme floods. Thus, contraction scour during extreme events may be classified as live-bed because of the sediment being delivered to the inlet or estuary from the combined effects of the storm surge and inland flood. The magnitude of contraction scour must be carefully evaluated using engineering judgment which considers the geometry of the crossing, estuary or bay, the magnitude and duration of the discharge associated with the storm surge or inland flood, the basic assumptions for which the contraction scour equations were developed, and mitigating factors which would tend to limit contraction scour.

Evaluation of local scour at piers can be made by using Equation 6.1 as recommended for inland river crossings. This equation can be applied to piers in tidal flows in the same manner as given for inland bridge crossings. However, the flow velocity and depth will need to be determined considering the design flow event and hydraulic characteristics for tidal flows.

9.4.5 Scour Evaluation Procedure for an Unconstricted Waterway

This method applies only when the tidal waterway or the bridge opening does not significantly constrict the flow and uses the tidal prism method as discussed by Neill.[77]

STEP 1. Determine the net waterway area at the crossing as a function of elevation. Net area is the gross waterway area between abutments minus area of the piers. It is often useful to develop a plot of the area versus elevation.

STEP 2. Determine tidal prism volume as a function of elevation. The volume of the tidal prism at successive elevations is obtained by planimetering successive sounding and contour lines and calculating volume by the average end area method. The tidal prism is the volume of water between low and high tide levels.

STEP 3. Determine the elevation versus time relation for the 100- and 500-year storm tides. The ebb and flood tide elevations can be approximated by either a sine or cosine curve. A sine curve starts at mean water level and a cosine curve starts at the maximum tide level. The equation for storm ebb tide that starts at the maximum elevation is:

$$y = A \, \text{Cos} \, \theta + Z \tag{9.1}$$

where:

Y	=	Amplitude or elevation of the tide above mean water level, m (ft) at time t
A	=	Maximum amplitude of elevation of the tide or storm surge, m (ft). Defined as half the tidal range or half the height of the storm surge
θ	=	Angle subdividing the tidal cycle, one tidal cycle is equal to 360° $\theta = 360 \left(\frac{t}{T} \right)$
t	=	Time from beginning of total cycle, minutes
T	=	Total time for one complete tidal cycle, minutes
Z	=	Vertical offset to datum, m (ft)

The tidal range (difference in elevation between high and low tide levels) is equal to twice the amplitude. One-half the tidal period is equal to the time between high and low tide. These relations are shown in Figure 9.2. A figure similar to Figure 9.2 can be developed to illustrate quantitatively the tidal fluctuations and resultant discharges.

To determine the elevation versus time relation for the 100- and 500-year storm tides, two values must be known:

- storm tidal range
- storm tidal period

As stated earlier, FEMA, USACE, NOAA, and other federal or state agencies compile records which can be used to estimate the 100- and 500-year storm tide elevation, mean sea level elevation, and low tide elevation. These agencies also are the source of data to determine the 100- and 500-year storm tide period.

Storm tides, may have different periods than the astronomical semi-diurnal and diurnal tides which have periods of approximately 12 and 24 hours, respectively. This is because storm tides are influenced by factors other than the gravitational forces of the sun, moon and other celestial bodies. Factors such as the wind, path of the hurricane or storm creating the storm tide, fresh water inflow, shape of the bay or estuary, etc. influence both the storm tide amplitude and period.

STEP 4. Determine the discharge, velocities and depth. Neill has stated the maximum discharge in an ideal tidal estuary may be approximated by the following equation:[(78)]

$$Q_{max} = \frac{3.14\ VOL}{T} \tag{9.2}$$

where:

Q_{max}	=	Maximum discharge in the tidal cycle, m^3/s (ft^3/s)
VOL	=	Volume of water in the tidal prism between high and low tide levels, m^3 (ft^3)
T	=	Tidal period between two successive high tides or two successive low tides, s

A simplification of Equation 9.2, suggested by Chang, is to assume the tidal prism has vertical sides.[(51)] With this assumption, which eliminates the need to compute the volume in the tidal prism by adding the volume of successive elevations, Equation 9.2 becomes:

$$Q_{max} = \frac{3.14\ A_s\ H}{T} \tag{9.2a}$$

where:

A_s	=	Surface area of the tidal prism at mean tide elevation, m^2 (ft^2)
H	=	Elevation difference (tidal range) between high and low tide levels, m (ft)

In the idealized case, Q_{max} occurs in the estuary or bay at mean water elevation and at a time midway between high and low tides when the slope of the tidal energy gradient is steepest (Figure 9.2).

The corresponding maximum average velocity in the waterway is:

$$V_{max} = \frac{Q_{max}}{A_c} \tag{9.3}$$

where:

V_{max} = Maximum average velocity in the cross section (where the bridge will be located) at Q_{max}, m/s (ft/s)

H = Cross-sectional area of the waterway at mean tide elevation, halfway between high and low tide, m^2 (ft^2)

It should be noted that the velocity as determined in the above equations represents the average velocity in the cross section. This velocity will need to be adjusted to estimate velocities at individual piers to account for nonuniformity of velocity in the cross section. As for inland rivers, local velocities can range from 0.9 to approximately 1.7 times the average velocity depending on whether the location in the cross section was near the banks or near the thalweg of the flow.

Neill's studies indicate that the maximum velocity in estuaries is approximately 30 percent greater than the average velocity computed using Equation 9.3. If a detailed analysis of the horizontal velocity distribution is needed, the design discharge could be prorated based on the conveyance in subareas across the channel cross section.

Another useful equation from Neill is:[78]

$$Q_t = Q_{max} \text{ Sin} \left(360 \frac{t}{T}\right) \tag{9.4}$$

where:

Q_t = Discharge at any time t in the tidal cycle, m^3/s (ft^3/s)

The velocities calculated with this procedure can be plotted and compared with any measured velocities that are available for the bridge site or adjacent tidal waterways to evaluate the reasonableness of the results.

STEP 5. Evaluate the effect of flows derived from inland riverine flow on the values of discharge, depth and velocities obtained in step 4. This evaluation may range from simply neglecting the inland flow into a bay (which may be so small that it is insignificant in comparison to the tidal flows), to routing the inland flow into the bay or estuary. If an estuary is a continuation of the stream channel and the storage of water in it is small, the inland flow can simply be added to the Q_{max} obtained from the tidal analysis and the velocities then calculated from Equation 9.3. However, if the inland flow is large and the bay or estuary sufficiently small that the inland flow will increase the tidal prism, the inland flood hydrograph should be routed through the bay or estuary and added to the tidal prism. The USACE HEC-1 could be used to route the flows.[87] In some instances, trial calculations will be needed to determine if and how the inland flow will be included in the discharge through the bridge opening.

STEP 6. Evaluate the discharge, velocities and depths that were determined in steps 4 and 5 above (or the following section for constricted waterways). Use engineering judgment to evaluate the reasonableness of these hydraulic characteristics. Compare these values with values for other bridges over tidal waterways in the area with similar conditions. Compare the calculated values with any measured values for the site or similar sites. Even if the measured discharge values for astronomical tides are much lower than the design storm tide discharge, they will give an appreciation of the magnitude of discharge to be expected.

STEP 7. Evaluate the scour for the bridge using the values of the discharge, velocity and depths determined from the above analysis using the scour equations recommended for inland bridge crossings presented previously. Care should be used in the application of these scour equations, using the guidance given previously for application of the scour equations to tidal situations.

9.4.6 Scour Evaluation Procedure for a Constricted Waterway

The procedures given above except for Steps 2 and 4 (the determination of the tidal prism, discharge, velocity and depth for unconstricted waterways) are followed. To determine these hydraulic variables when the constriction is caused by the channel and not the bridge, the following equation for tidal inlets taken from van de Kreeke[89] or Bruun[90] can be used.

$$V_{max} = C_d \sqrt{2g \Delta H} \tag{9.5}$$

$$Q_{max} = A_c \; V_{max} \tag{9.6}$$

where:

- V_{max} = Maximum velocity in the inlet, m/s (ft/s)
- Q_{max} = Maximum discharge in the inlet, m^3/s (ft^3/s)
- C_d = Coefficient of discharge ($C_d < 1.0$)
- g = Acceleration due to gravity, 9.81 m/s^2 (32.2 ft/s^2)
- ΔH = Maximum difference in water surface elevation between the bay and ocean side of the inlet or channel, m (ft)
- A_c = Net cross-sectional area in the inlet at the crossing, at mean water surface elevation, m^2 (ft^2)

The difference in water surface elevation, ΔH, should be for the normal astronomical tide, the 100-year storm tide and the 500-year storm tide. The difference in height for the normal astronomical tide is used to determine potential long-term degradation at the crossing if the crossing has a deficient or interrupted sediment supply (e.g., by construction of a jetty which cuts off littoral drift). This condition can lead to the inlet becoming unstable and degrading (i.e., enlarging) indefinitely.

The coefficient of discharge (C_d) for most practical applications can be assumed to be equal to approximately 0.8. Alternatively, the coefficient of discharge can be computed using the equations given by van de Kreeke[89] or Bruun:[90]

$$C_d = (1/R)^{1/2} \tag{9.7}$$

where

$$R = K_o + K_b + \frac{2g\, n^2\, L_c}{k_u^2\, h_c^{4/3}} \tag{9.8}$$

and

- R = Coefficient of resistance
- K_o = Velocity head loss coefficient on the ocean side or downstream side of the waterway taken as 1.0 if the velocity goes to 0
- K_b = Velocity head loss coefficient on the bay or upstream side of the waterway. Taken as 1.0 if the velocity goes to 0
- n = Manning's roughness coefficient
- L_c = Length of the waterway (inlet), m (ft)
- h_c = Average depth of flow in the waterway at mean water elevation, m (ft)
- K_u = 1.0 SI
- K_u = 1.486 English

The values of K_o and K_b depend on local hydrodynamic conditions, but are generally greater than 0.5. For a flood tide exiting an inlet to a large bay the coefficient K_b can be taken as 1.0.

If ΔH is not known or cannot be determined easily, a hydrologic routing method developed by Chang et al., which combines the above orifice equations (Equation 9.5 - 9.8) with the continuity equation, can be used.[91] The total flow approaching the bridge crossing at any time (t) is the sum of the riverine flow (Q) and tidal flow. The tidal flow is calculated by multiplying the surface area of the upstream tidal basin (A_s) by the drop in elevation (H_s) over the specified time ($Q_{tide} = A_s\, dH_s/dt$). This total flow approaching the bridge is set equal to the flow calculated from the orifice equation.

$$Q + A_s \frac{dH_s}{dt} = C_d\ A_c\ \sqrt{2g\ \Delta H} \tag{9.9}$$

where:

- A_c = Bridge waterway cross-sectional area, m^2 (ft^2)
- H_s = Water surface elevation in the tidal basin upstream of the bridge, m (ft)
- Q = Riverine discharge m^3/s (ft^3/s)

All other variables are as previously defined.

Equation 9.9 may discretized with respect to time as denoted in Equation 9.10 for the time interval, $\Delta t = t_2 - t_1$. Subscripts 2 and 1 represent the end and beginning of the time interval, respectively.

$$\frac{Q_1 + Q_2}{2} + \frac{A_{S1} + A_{S2}}{2}\frac{H_{S1} - H_{S2}}{\Delta T} = C_d\left(\frac{A_{C1} + A_{C2}}{2}\right)\sqrt{2g\left(\frac{H_{s1} + H_{S2}}{2} - \frac{H_{t1}}{2}\right)} \tag{9.10}$$

For a given initial condition, t_1, all terms with subscript 1 are known. For $t=t_2$, the downstream tidal elevation (H_{t2}), riverine discharge (Q_2), and waterway cross-sectional area (A_{c2}) are also known or can be calculated from the tidal elevation. Only the water-surface elevation (H_{s2}) and the surface area (A_{s2}) of the upstream tidal basin remain to be determined. Because surface area of the tidal basin is a function of the water-surface elevation, the elevation of the tidal basin at time t_2 (H_{s2}) is the only unknown term in Equation 9.10, and this term can be determined by trial-and-error to balance the values on the right and left sides.

Chang et al. suggest the following steps for computing the flow:[91]

Step 1. Determine the period and amplitude of the design tide(s) to establish the time rate of change of the water-surface on the downstream side of the bridge.

Step 2. Determine the surface area of the tidal basin upstream of the bridge as function of elevation by planimetering successive contour intervals and plotting the surface area vs. elevation.

Step 3. Plot bridge waterway area vs. elevation.

Step 4. Determine the quantity of riverine flow that is expected to occur during passage of the storm tide through the bridge.

Step 5. Route the flows through the contracted waterway using Equation 9.10, and determine the maximum velocity of flow.

In most cases, development of a UNET or other 1-dimensional unsteady flow model will be as easy as performing the routing described above.

Using the tidal hydraulics determined as described above for constricted inlets, the scour computations can proceeded according to steps 5, 6, and 7 presented previously for the unconstricted waterway.

9.5 TIDAL CALCULATIONS USING UNSTEADY FLOW MODELS

9.5.1 Tidal Hydraulic Models

Alternatively, the tidal hydraulics at the bridge can be determined using one of several unsteady flow models in lieu of either Neill's procedure, the orifice equation or Chang's procedure. A brief overview of these models is presented below. This information was derived from a pooled fund study (HPR552) administered by the SCDOT.[13,87] All quotes presented in this section are from the final report documenting the first phase of this study.

ACES is an acronym for the Automated Coastal Engineering System and was developed by the USACE in an effort to incorporate many of the various computational procedures typically needed for coastal engineering analysis into an integrated, menu-driven user environment.[83] There are seven separate computation modules for wave prediction, wave theory, littoral processes and other useful modules. One such module denoted as ACES-INLET is a spatially integrated numerical model for inlet hydraulics. This module can be used to determine discharges, depths and velocities in tidal inlets with up to two inlets connecting a bay to the ocean. This module can be used in place of, or in addition to, the procedures given in steps 3 and 4, above, for tidal inlets. **ACES-INLET is applicable only where the project site is at or very near the inlet throat (i.e., for bridges crossing inlets) (Figure 9.1).**

The pooled fund study states:[13]

> "*ACES-Inlet is simple and easy to use. A minimum of data are required and the menu-driven environment makes user input straightforward. The primary limitation of the model is its reliance on numerous empirical coefficients. In addition to requiring keen judgment on the part of the user, the empirical relations greatly oversimplify the inlet dynamics. Model results can be regarded as rough approximations, useful for reconnaissance-level investigations.*"

Other modules incorporated into ACES may be useful in evaluating tidal highway crossings. These modules can be used to estimate wave and tidal parameters, littoral drift, wave run-up and other aspects of tidal flow which could influence the design or evaluation of bridge crossings over tidal inlets connecting bays to the ocean.

UNET is a 1-dimensional unsteady flow model.[84] Although simpler to use than more complex 2-dimensional models, UNET can model networks of open channels, and bifurcations and flow around islands. According to the pooled fund study:

> *"UNET is extremely flexible in modeling of channel networks, storage areas, bifurcations, and junctions. Both external boundaries (hydrographs, stage hydrographs) and internal boundary conditions (gated and uncontrolled spillways, bridges, culverts, and levee systems) can be included. UNET uses a modified HEC-2 file format to facilitate data entry and UNET can use the HEC-DSS database for input and output."*

According to the pooled fund study, the advantages and limitations of UNET are:

> *"UNET uses an efficient implicit numerical formulation solution techniques. Of the reviewed unsteady 1-dimensional flow models, UNET is the only model which intrinsically evaluated bridges, culverts, and embankment overtopping.... Although UNET does not simulate flow separation (2-D), off-channel storage (ineffective flow areas) can be used to represent these areas. The primary limitation of this model is the exclusion of wind effects."*

FESWMS-2DH is a 2-dimensional unsteady flow model developed by the USGS and FHWA.[45] This model uses a finite element numerical simulation and has options for simulation of steady or unsteady flow over highway embankments and through culverts. The model has been incorporated into the SMS[91] user interface. The critique of FESWMS-2DH in the pooled fund study states:

> *"The options for weir flow and culvert flow are particularly well suited to highway application. The variable friction formulation permits realistic modeling of floodplains. FESWMS-2DH has limitations similar to those of other 2- models, e.g. inability to simulate stratified flows or complex near-field phenomena where vertical velocities are not negligible. The relative complexity of the model (as compared to 1-D models) requires some expertise for model setup and use."*

RMA-2V is a widely used 2-dimensional unsteady flow model which uses a finite element numerical procedure.[85,86] The model is incorporated into the SMS user interface which provides additional applications including SED2D which, when linked with RMA-2V, modifies the geometry of the waterway using computations of sediment erosion, sedimentation and transport during each time step of the hydrodynamic model. The critique of RMA-2V in the pooled fund study states:

> *"RMA-2V and the TABS/FastTABS system (now in SMS) offer a rigorous 2-D solution to the shallow water equations coupled with sediment transport capabilities and advanced pre/post processors. The finite element spatial discretization is accurate and can easily represent complex physical systems. Other capabilities include simulation of wetting and drying elements and flow control structures..."*

Of the four unsteady models, ACES and UNET are significantly simpler than either FESWMS or RMA-2V. Because of this, ACES and UNET can be considered to be more adaptable to Level 2 type analysis due to their relative simplicity. Although FESWMS and RMA-2V can be used as part of an advanced Level 2 analysis, their use is more consistent with a Level 3 analysis. As indicated earlier, efforts to enhance and improve these models so that they better support highway applications are ongoing. Future enhancements and versions of these models will likely provide for simpler application and better estimates of the hydraulic conditions which influence scour.

Another advancement in scour analysis of bridges over tidal waterways is the production of a manual on tidal hydraulic modeling for bridges.(87) This manual was developed as part of the second phase of a pooled fund study.(13) The manual includes methods for developing realistic tidal and storm surge boundary conditions, discussions on the applicability of various hydraulic modeling approaches (tidal prism, orifice, routing, hydrodynamic modeling), and examples on the use of 1- and 2-dimensional modeling. Guidance is also being developed on when to include inland runoff with storm surge simulations, effects of wind, time dependency of scour, and wave height determination. Figure 9.4 shows an example of a synthetic storm surge hydrograph added to a daily tide. This is a realistic representation of the surge that could be used as an ocean boundary condition for hydrodynamic modeling. Hydrodynamic modeling has been used on numerous projects to evaluate the scour potential of new and existing bridges.

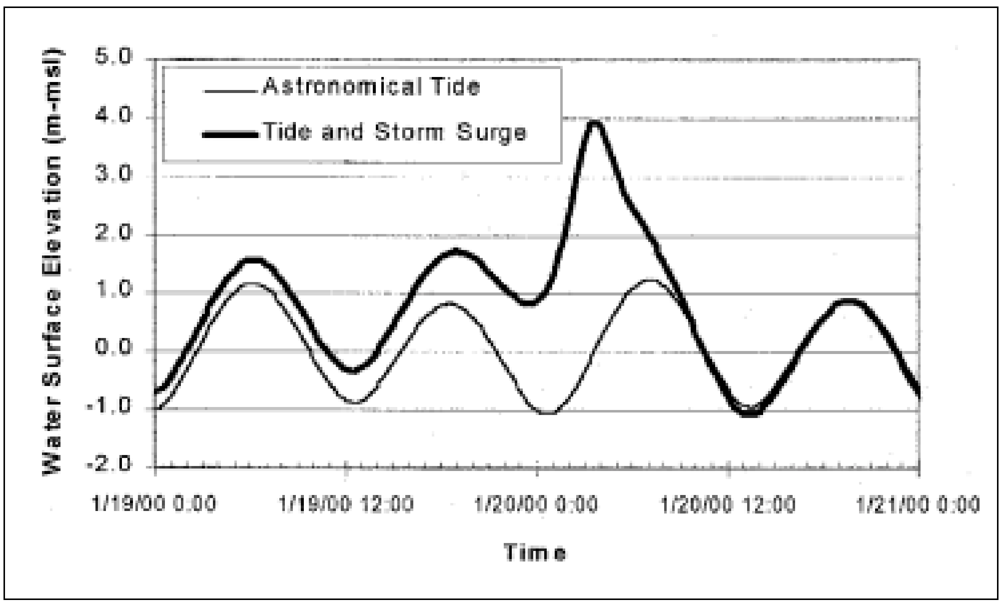


Figure 9.4. Synthetic storm surge hydrograph combined with daily tide.(87)

9.5.2 Data Requirements for Hydraulic Model Verification

Whenever a hydraulic model is employed, it is necessary to calibrate the model to insure that the results will adequately represent the flow conditions which are likely to occur during an extreme event. Because of this, any model, including WSPRO and HEC-RAS should be verified against actual data.(15,16,17) For inland rivers systems model verification is reasonably straightforward. Known discharges and water surface elevations are used to adjust the downstream boundary conditions and resistance parameters until a close agreement between measured data and model output is obtained. Although similar, model verification using unsteady flow models is more difficult due to the unsteady nature of the flow. The following paragraphs discuss data needs for model verification of unsteady flow models.

Ideally, synoptic measurements of the following data are required to validate hydraulic modeling using any of the above mentioned unsteady flow models:

- Tidal elevations in the ocean and back-bay locations
- Velocity measurements are needed in the inlet throat as well as at proposed project sites
- Boundary condition data for any back-bay, open-water boundaries; these data may be elevation, velocity, discharge, or any combination of these parameters
- Wind speed and direction if wind energy influences in the tidal system

The above data may be available from previous studies of the tidal system (for example, USACE or NOAA studies) or may be collected for a specific project.

9.6 TIME DEPENDENT CHARACTERISTICS OF TIDAL SCOUR

In tidal areas, hurricane storm surges often produce extreme hydraulic conditions. Computing **ultimate contraction scour** amounts for these conditions may not be reasonable based on the short duration (often less than 3 hours) of the flow produced by the surge. Based on equations in a Scour Manual published in the Netherlands,[93] (see also Transportation Research Board Research Results Digest[94]), the time development of scour holes can be estimated. To provide confirmation of these results, the Yang[95] sediment transport equation was used to compute contraction scour hole development based on the erosion of the scour hole equal to the transport capacity in the contracted bridge opening. The scour rates for this situation are shown on Figures 9.5 and 9.6. Figure 9.5 shows the complete development of scour with time plotted on a logarithmic axis and Figure 9.6 shows the first 100 hours of development with time plotted on an arithmetic axis. The scour rates predicted by the two methods are extremely similar and indicate that the scour that could be generated in a few hours during a storm surge is significantly less than the ultimate contraction scour condition.

Also shown in Figures 9.5 and 9.6 is the development of a **pier scour** hole for the same hydraulic conditions. The pier scour hole reaches 90 percent of ultimate scour in the first 20 hours while the clear-water **contraction scour** reaches only about 30 percent of ultimate scour.

The Dutch equations are based on clear-water scour and the conditions used to test the Yang equation were close to clear-water. The Dutch Scour Manual[93] indicates that under live-bed conditions scour reaches ultimate conditions more rapidly and that the ultimate scour is less than the equivalent clear-water case which is consistent with current U.S. guidance. Figure 9.7 shows the development of contraction scour (using the Yang equation) under varying amounts of upstream sediment supply relative to the transport capacity in the bridge opening. This approach involves a basic sediment continuity analysis as outlined in HEC-20.[6] For the case shown, if the upstream channel is supplying 50 percent of the contracted section transport capacity, the scour hole reaches the ultimate depth in approximately one hour. Based on this review, it appears that under storm surge conditions contraction scour should be analyzed on a case-by-case basis to assess the level of contraction scour that could occur over a short time. It also suggests that local scour occurs more rapidly and time dependence is a less significant factor.

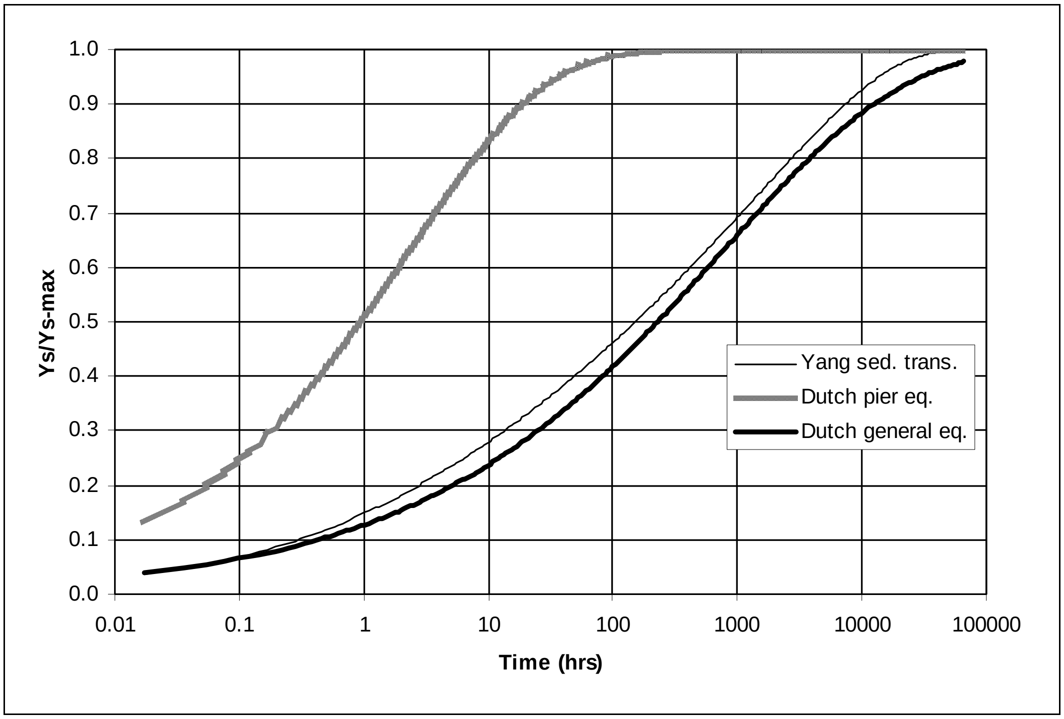


Figure 9.5. Time development of scour.

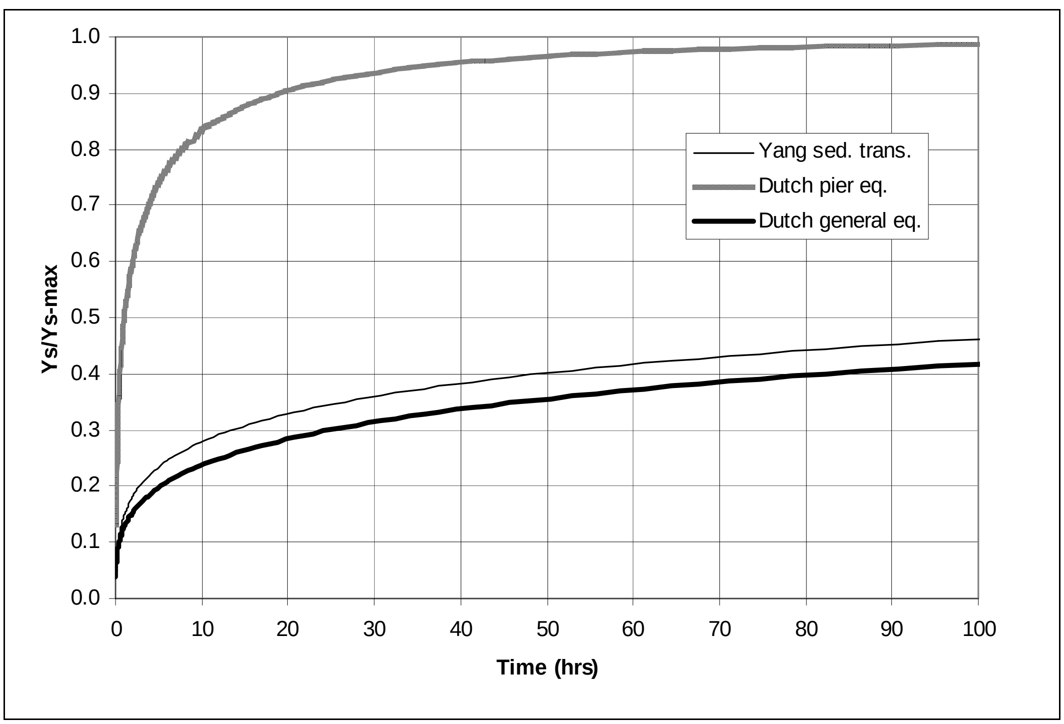


Figure 9.6. Initial scour development.

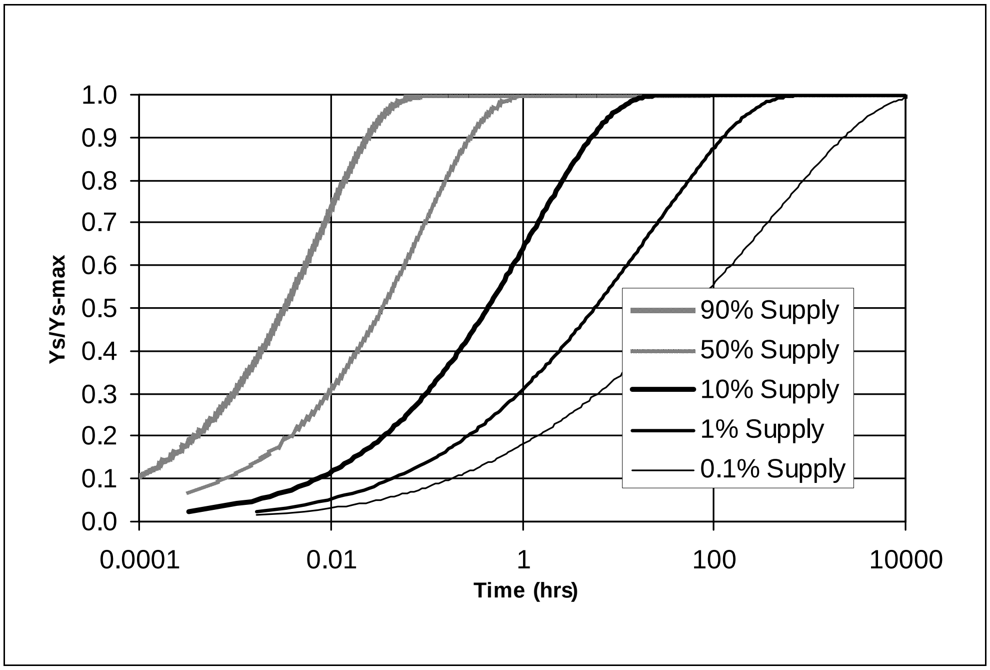


Figure 9.7. Contraction scour development with sediment supply.

9.7 LEVEL 3 ANALYSIS

As discussed in HEC-20, Level 3 analysis involves the use of physical models or more sophisticated computer models for complex situations where Level 2 analysis techniques have proven inadequate.[6] In general, crossings that require Level 3 analysis will also require the use of qualified hydraulic engineers. Level 3 analysis by its very nature is specialized and beyond the scope of this manual.

9.8 TIDAL SCOUR EXAMPLE PROBLEMS (SI)

9.8.1 Example Problem 1 - Tidal Prism Approach (Unconstricted Waterway) (SI)

In this example problem, the discharge, velocity, depths, and scour are to be determined for an existing bridge across a tidal estuary as part of an ongoing scour evaluation. The bridge is 818.39 m long, has vertical wall abutments and 16 bents each consisting of two 3.66 m diameter circular piers supported on piles. Neither the bridge nor the tidal waterway constricts the flow.

For this evaluation, the bridge maintenance engineer has expressed concern about observed scour at one of the piers. This pier is located where the velocities at the pier are approximately 30 percent greater than the average velocities. The water depth at the pier referenced to mean sea level, is 3.75 m. The actual depth of flow at the pier will need to be increased to account for additional water depth caused by the storm surge for the computation of pier scour.

Level 1 Analysis

a. Level 1 analysis has determined that the storm surge for the 100- and 500-year return period produces discharge, velocity and depths that are much larger than those from inland runoff. There is minimal littoral drift and historical tides are low. From FEMA, the storm surge tidal range for the 100-year return period is 2.19 m and for the 500-year return period is 2.87 m. Measured maximum velocity in the waterway at mean sea level for a tide of 0.67 m was only 0.21 m/s.

   Sonic soundings in the waterway indicate that there is storage of sediment in the estuary directly inland from the bridge crossing. This was determined by observing that the elevation of the bed of the waterway at the bridge site was lower than the elevation of the bottom of the estuary further inland. Although no littoral drift is evident, there is storage of sediment at the mouth of the estuary between the ocean and the bridge crossing.

b. Stability of the estuary and crossing was evaluated by examination of the periodic bridge inspection reports which included underwater inspections by divers, evaluation of historical aerial photography, and depth soundings in the estuary using sonic fathometers. From this evaluation it was determined that the planform of the estuary has not changed significantly in the past 30 years. These observations indicate that the estuary and bridge crossing has been laterally stable.

   Evaluation of sounding data at the bridge indicates that there has been approximately 1.52 m of degradation at the bridge over the past 30 years; however, the rate of degradation in the past five years has been negligible. Underwater inspections indicted that local scour around the piers is evident.

c. A search of FEMA, USACE, and other public agencies for inland flood and storm surge data was conducted. These data will be discussed under the Level 2 analysis.

d. Grain size analysis of the bed material indicates that the bed of the estuary is composed of fine sand with a D_{50} of approximately 0.27 mm (0.00027 m).

e. Velocities measured at Q_{max} during a large astronomical tide indicated that the maximum velocity in the bridge section was approximately 30 percent greater than the average velocity.

Level 2 Analysis

STEP 1. A plot of net waterway area as a function of elevation is given in Figure 9.8. Net waterway area is the average area at the bridge crossing less the area of the piers.

STEP 2. A plot of volume of the tidal prism as a function of elevation is also presented in Figure 9.8. The plot was developed by planimetering the area of successive sounding and contour lines and multiplying the average area by the vertical distance between them.

STEP 3. A synthesized storm surge for the 100- and 500-year return period was developed and is presented in Figure 9.8. It was obtained as follows:

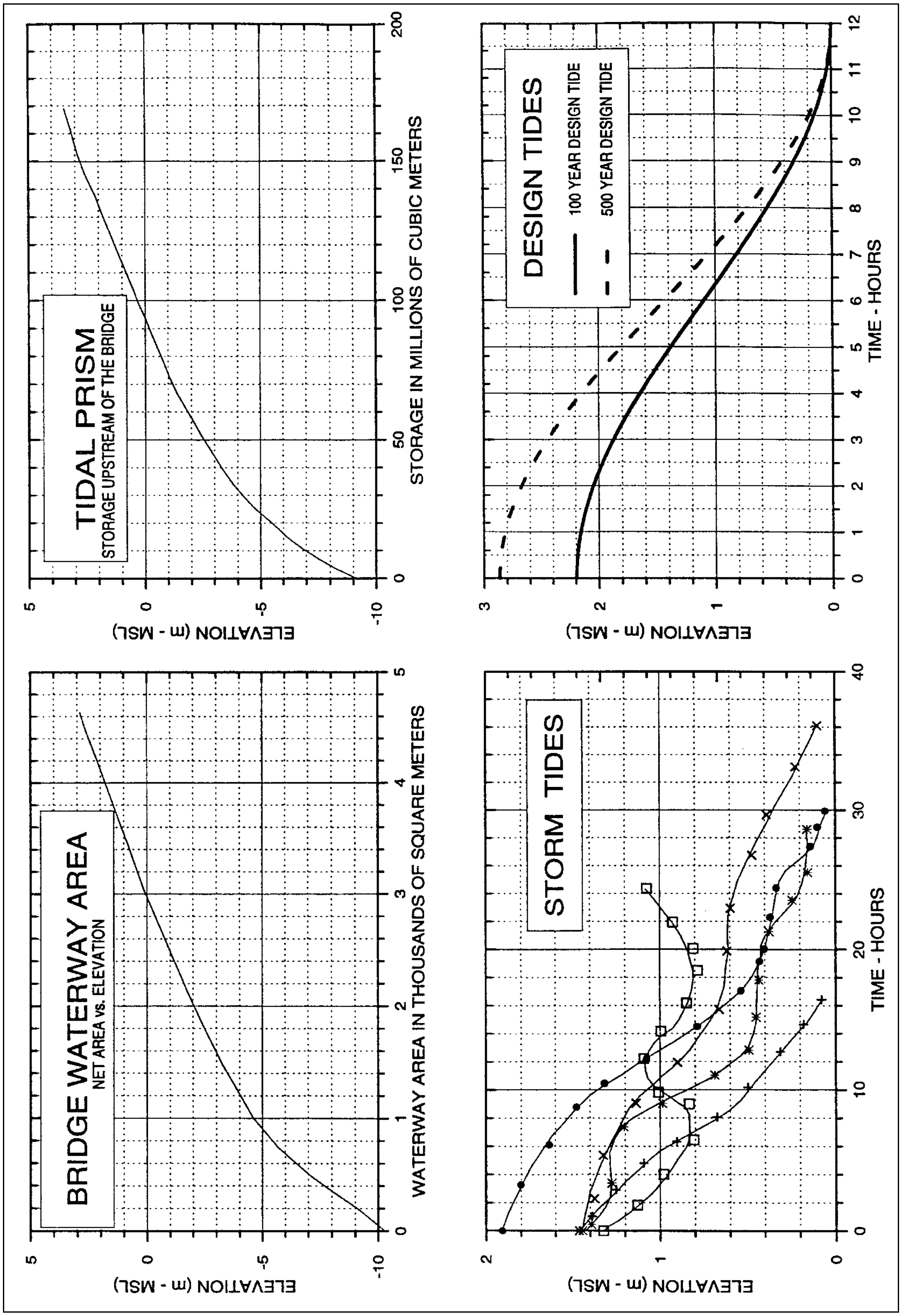


Figure 9.8. Tidal parameters for Example Problem 1 (SI).

An idealized graph for one half the tidal period, beginning at high tide was developed using the cosine equation (Equation 9.1). This plot can be used to develop an idealized tidal cycle for any waterway. Tidal range and period are needed to use the idealized tide cycle to develop a synthesized tidal cycle for this waterway.

The tidal ranges were obtained from a FEMA coastal flood insurance study during the Level 1 analysis (Table 9.1).

Table 9.1. Tidal Ranges Derived from FEMA Flood Study.

Return Period (yr)	High Tide (m)	Low Tide (m)
100	2.19	0
500	2.87	0

The tidal period is more difficult to determine because it is affected by more than the gravitational attraction of the moon and sun. At this waterway location, the direction of the storm and the characteristics of the estuary affected the tidal period. To determine the tidal period, major storm tides were plotted in Figure 9.8. Review of these historical storm tides reveals that (as expected) most events occur over a duration longer than an astronomical tidal period. Only a single event exhibits a seemingly semi-diurnal response. Given these characteristics and behavior, analyses yield a conservative estimate that approximately 12 hours pass between the highest and lowest elevations. This assumption would therefore indicate that the associated storm tide period (T) is 24 hours.

STEP 4. Using the data developed in Steps 1 to 3 and the equations given previously the maximum tidal discharge (Q_{max}) and maximum average tidal velocity (V_{max}) are calculated. The values used in the calculations are given in Table 9.2.

STEP 5. The 100- and 500-year return period peak inland flow into the estuary was obtained from a USGS flood frequency study. These values are also given in Table 9.2.

Table 9.2. Design Discharge and Velocities.

	100-Year Storm Tide	500-Year Storm Tide
Maximum storm tide elevation, m	2.19	2.87
Mean storm tide elevation, m	1.10	1.44
Low storm tide elevation, m	0.0	0.0
Tidal prism volume (millions of cubic meters) Figure 9.8	46.40	60.80
Net waterway area at mean storm tide elevation (A_c), m^2	3620	3809
Tidal period, h	24.0	24.0
Q_{max} (Tidal), m^3/s (Equation 9.2)	1686.3	2209.6
V_{max} (Tidal), m/s (Equation 9.3)	0.47	0.58
Inland peak runoff (discharge), m^3/s	141.03	224.29
Q_{max} (Tidal plus runoff), m^3/s	1827.33	2433.83
V_{max} (Tidal plus runoff), m/s ($V_{max} = Q_{max}/A_c$)	0.50	0.64
Average flow depth - A_c/width, m	4.42	4.65

Average flow depths can be determined by dividing the flow area as listed in Table 9.2 by the channel width (818.4 m). Therefore, the average flow depths for the 100- and 500-year event are 4.42 and 4.65 m, respectively.

The peak discharge from the 100- and 500-year inland flow hydrograph is very small in comparison to the storage volume in the estuary. In this case, adding the inland peak discharge to the maximum tidal discharge will be a conservative estimate of the maximum discharge and maximum average velocity in the waterway. If the inland inflow into the estuary had been large, the flood could be routed through the estuary using standard hydrologic modeling techniques.

STEP 6. A comparison of the calculated velocities with the measured velocities indicate that they are reasonable. The discharge and velocities given in Table 9.2 are acceptable for determining the scour depths. However, the average velocity will have to be adjusted for the nonuniformity of flow velocity in the vicinity of the bridge to obtain the velocities for determining local scour at the piers.

STEP 7. Calculate the components of total scour using the information collected in the Level 1 and Level 2 analyses.

Long-Term Aggradation/Degradation

The Level 1 analysis indicates that the channel is relatively stable at this time. However, there is an indication that over the past 30 years the channel has degraded approximately 1.52 m. Since the degradation rate has been negligible in the last five years, no additional degradation will be anticipated.

Contraction Scour

Contraction scour depends on whether the flow will be clear-water or live-bed. Equation 5.1 is used to determine the critical velocity for the 100-year hydraulics.

$$V_c = 6.19\,(4.42)^{1/6}\,(0.00027)^{1/3} = 0.5\,m/s$$

This indicates that the 100-year storm surge combined with the inland flow may result in velocities greater than or equal to the critical velocity; therefore, contraction scour will most likely be live-bed. This conclusion is made considering that velocities in excess of the average velocity will be expected due to the nonuniformity of the velocity in the bridge opening, as determined during the Level 1 analysis.

Applying the live-bed contraction scour equation, it is noted that the ratio of discharges is equal to unity (i.e., there is no overbank flow). Therefore, the contraction scour will be influenced by the contraction resulting from the bridge piers reducing the flow width at the bridge crossing. Using Equation 5.2, and assuming that the mode of sediment transport is mostly suspended load (k_1=0.69), the estimate of live-bed contraction scour for the 100-year event is:

$$\frac{y_2}{4.42} = \left[\frac{818.39}{759.84}\right]^{0.69} = 105$$

$y_2 = 4.64\,\text{m}$

$y_s = 4.64 - 4.42 = 0.22\,\text{m}$

Therefore, the contraction scour for the 100-year event is approximately 0.22 m. Recomputation for the 500-year event with an average flow depth of 4.65 m results in an estimate of contraction scour of approximately 0.24 m.

Local Scour at Piers

The hydraulic analysis estimates average velocities in the bridge cross section only. Because of this, an estimate of the maximum velocity at the bridge pier is made to account for non-uniform velocity in the bridge cross section. The average velocity will be increased by 30 percent since velocities for normal flows (Level 1) indicated that the maximum velocity was observed to be approximately 30 percent greater than the average. Therefore the maximum velocity for the 100- and 500-year event are 0.65 and 0.83 m/s, respectively.

K_1, K_2, and K_4 equal 1.0. K_3 will be equal to 1.1 since the bed condition at the bridge is plane-bed. The depth of flow at the pier for the 100- and 500-year storm surge is determined by adding the mean storm tide elevation from Table 9.2 to the flow depth at the pier referenced to mean sea level (3.75 m). From this, y_1 will be equal to 4.85 and 5.19 m for the 100- and 500-year storm surge, respectively.

Applying Equation 6.1 for the 100-year event:

$$\frac{y_s}{4.85} = 2.0\,(1.0)\,(1.0)\,(1.1)\,(1.0)\left[\frac{3.66}{4.85}\right]^{0.65}(0.094)^{0.43} = 0.66$$

From the above equation, the local scour at the piers is 3.2 m. Considering the 500-year event, local pier scour is 3.6 m.

9.8.2 Example Problem 2 - Constricted Waterway

This problem presents a Level 2 analysis of a bridge over a tidal inlet where the waterway constricts the flow. In addition, it illustrates how depletion of sediment supplied to the tidal inlet can result in a continual and severe long-term degradation. The length of the inlet is 457.2 m, the width of the bridge opening and inlet is 124.97 m, Manning's n is 0.03, depth of flow at mean water level is 6.1 m and area A_c is 761.81 m^2. The D_{50} of the bed material is 0.30 mm and the D_m (1.25 D_{50}) is 0.375 mm (0.000375 m).

From tidal records, the long-term average difference in elevation from the ocean to the bay, through the waterway, averaged for both the flood and ebb tide is 0.183 m. The difference in elevation for the 100-year storm surge is 0.549 m and for the 500-year storm surge is 0.884 m.

a. Determine the long-term potential degradation that may occur because construction of jetties has cut off the delivery of bed sediments from littoral drift to the inlet.

For this situation, long-term degradation can be approximated by assuming clear-water contraction scour and using the average difference in water surface between the ocean and bay for astronomical tides. The hydraulic computation uses the orifice equations (Equations 9.5 through 9.10).

Using Equation 9.8, determine R (assume K_o = 0.7 and K_b = 1.0 for this location)

$$R = 0.7 + 1.0 + \frac{2(9.81)(0.03)^2\,457.2}{(6.10)^{4/3}}$$

$$R = 2.42$$

From Equation 9.7 determine C_d

$$C_d = \left(\frac{1}{2.42}\right)^{1/2}$$

$$C_d = 0.643$$

Using Equation 9.5, determine V_{max}

$$V_{max} = 0.643\sqrt{(2)(9.81)(0.183)}$$

$$V_{max} = 1.22\,m/s$$

Using Equation 9.6 determine Q_{max}

$$Q_{max} = V_{max}\,A_c = 1.22(761.81)$$

$$Q_{max} = 929.41\,m^3/s$$

Potential long-term degradation for fine bed material is determined using the clear-water contraction scour equation (Equation 5.4):

$$y = \left[\frac{0.025(929.41)^2}{(0.000375)^{2/3}(124.97)^2}\right]^{3/7} = 10.94\,m$$

$$y_s = 10.94 - 6.10 = 4.84\,m$$

Discussion of Potential Long-Term Degradation

This amount of scour would occur in some time period that would depend on the amount of sediment that was available from the bay and ocean side of the waterway to satisfy the transport capacity of the back and forth movement of the water from the flood and ebb tide.

Even if there was no sediment inflow into the waterway, the time it would take to reach this depth of scour is not known.

To determine the length of time would require the use of an unsteady tidal model, and conducting a sediment continuity analysis (see Section 9.6). Using a tidal model and sediment continuity analysis, calculate the amount of sediment eroded from the waterway during a tidal cycle and determine how much degradation this will cause. Then using this new average depth, recalculate the variables and repeat the process. Knowing the time period of the tidal cycle, then the time to reach a scour depth of 4.84 m could be estimated for the case of no sediment inflow into the waterway. Estimates of sediment inflow in a tidal cycle could be used to determine the time to reach the above estimated contraction scour depth when there is sediment inflow.

When the long-term degradation reaches 4.84 m, the scouring may not stop. The reason for this is that the discharge in the waterway is not limited, as in the case of inland rivers, but depends on the amount of flow that can enter the bay in a half tidal cycle. As the area of the waterway increases the flood tide discharge increases because, as an examination of Equations 9.5 and 9.6 show the velocity does not decrease. There may be a slight decrease in velocity because the difference in elevation from the ocean and the bay might decrease as the area increases. However, R in Equation 9.8 decreases with an increase in depth.

Although the above discussion would indicate that long-term degradation would increase indefinitely, this is not the case. As the scour depth increases there would be changes in the relationship between the incoming tide and the tide in the bay or estuary, and also between the tide in the bay and the ocean on the ebb tide. This could change the difference in elevation between the bay and ocean. At some level of degradation the incoming or out-going tides could pick up sediment from either the bay or ocean which would then satisfy the transport capacity of the flow. Also, there could be other changes as scour progressed, such as accumulation of larger bed material on the surface (armor) or exposure of scour resistance rock which would decrease or stop the scour.

In spite of these limiting factors, the above problem illustrates the fact that with tidal flow, in contrast to river flow, as the area of the cross section increases from degradation there may be no decrease in velocity and discharge.

b. Determine V_{max}, Q_{max} for the 100-year storm surge and a depth of 6.1 m.

The values of R and C_d do not change.

$$V_{max} = 0.643\sqrt{(2)(9.81)(0.549)}$$

$$V_{max} = 2.11 m/s$$

$$Q_{max} = 2.11(761.81) = 1607.42 m^3/s$$

These values or similar ones depending on the long-term scour depth, would be used to determine the local scour at piers and abutments using equations given previously. These values could also be used to calculate contraction scour resulting from the storm surge.

9.9 TIDAL SCOUR EXAMPLE PROBLEMS (English)

9.9.1 Example Problem 1 - Tidal Prism Approach (Unconstricted Waterway) (English)

In this example problem, the discharge, velocity, depths, and scour are to be determined for an existing bridge across a tidal estuary as part of an ongoing scour evaluation. The bridge is 2,685 ft long, has vertical wall abutments and sixteen 12 ft diameter circular piers supported on piles. Neither the bridge nor the tidal waterway constricts the flow.

For this evaluation, the bridge maintenance engineer has expressed concern about observed scour at one of the piers. This pier is located where the velocities at the pier are approximately 30 percent greater than the average velocities. The water depth at the pier referenced to mean sea level is 12.30 ft. The actual depth of flow at the pier will need to be increased to account for additional water depth caused by the storm surge for the computation of pier scour.

Level 1 Analysis

a. Level 1 analysis has determined that the storm surge for the 100- and 500-year return period produces discharge, velocity, and depths that are much larger than those from inland runoff. There is minimal littoral drift and historical tides are low. From FEMA, the storm surge tidal range for the 100-year return period is 7.18 ft and for the 500-year return period is 9.42 ft. Measured maximum velocity in the waterway at mean sea level for a tide of 2.20 ft was only 0.70 ft/s.

   Sonic soundings in the waterway indicate thta there is storage of sediment in the estuary directly inland from the bridge crossing. This was determined by observing that the elevation of the bed of the waterway at the bridge site was lower than the elevation of the bottom of the estuary further inland. Although no littoral drift is evident, there is storage of sediment at the mouth of the estuary between the ocean and the bridge crossing.

b. Stability of the estuary and crossing was evaluated by examination of the periodic bridge inspection reports which included underwater inspections by divers, evaluation of historical aerial photography, and depth soundings in the estuary using sonic fathometers. From this evaluation it was determined that the planform of the estuary has not changed significantly in the past 30 years. These observations indicate that the estuary and bridge crossing has been laterally stable.

   Evaluation of sounding data at the bridge indicates that there has been approximately 5.0 ft of degradation at the bridge over the past 30 years; however, the rate of degradation in the past five years has been negligible. Underwater inspections indicted that local scour around the piers is evident.

c. A search of FEMA, USACE, and other public agencies for inland flood and storm surge data was conducted. These data will be discussed under the Level 2 analysis.

d. Grain size analysis of the bed material indicates that the bed of the estuary is composed of fine sand with a D_{50} of approximately 0.27 mm (0.00089 ft).

e. Velocities measured at Q_{max} during a large astronomical tide indicated that the maximum velocity in the bridge section was approximately 30 percent greater than the average velocity.

Level 2 Analysis

STEP 1. A plot of net waterway area as a function of elevation is given in Figure 9.9. Net waterway area is the average area at the bridge crossing less the area of the piers.

STEP 2. A plot of volume of the tidal prism as a function of elevation is also presented in Figure 9.9. The plot was developed by planimetering the area of successive sounding and contour lines and multiplying the average area by the vertical distance between them.

STEP 3. A synthesized storm surge for the 100- and 500-year return period was developed and is presented in Figure 9.9. It was obtained as follows:

An idealized graph for one half the tidal period, beginning at high tide was developed using the cosine equation (Equation 9.1). This plot can be used to develop an idealized tidal cycle for any waterway. Tidal range and period are needed to use the idealized tide cycle to develop a synthesized tidal cycle for this waterway.

The tidal ranges were obtained from a FEMA coastal flood insurance study during the Level 1 analysis (Table 9.3).

Table 9.3. Tidal Ranges Derived from FEMA Flood Study.

Return Period (yr)	High Tide (ft)	Low Tide (ft)
100	7.20	0
500	9.42	0

The tidal period is more difficult to determine because it is affected by more than the gravitational attraction of the moon and sun. At this waterway location, the direction of the storm and the characteristics of the estuary affected the tidal period. To determine the tidal period, major storm tides were plotted in Figure 9.9. Review of these historical storm tides reveals that (as expected) most events occur over a duration longer than an astronomical tidal period. Only a single event exhibits a seemingly semi-diurnal response. Given these characteristics and behavior, analyses yield a conservative estimate that approximately 12 hours pass between the highest and lowest elevations. This assumption would therefore indicate that the associated storm tide period (T) is 24 hours.

STEP 4. Using the data developed in Steps 1 to 3 and the equations given previously the maximum tidal discharge (Q_{max}) and maximum average tidal velocity (V_{max}) are calculated. The values used in the calculations are given in Table 9.4.

STEP 5. The 100- and 500-year return period peak inland flow into the estuary was obtained from a USGS flood frequency study. These values are also given in Table 9.4.

Average flow depths can be determined by dividing the flow area as listed in Table 9.4 by the channel width (2,685 ft). Therefore the average flow depth for the 100- and 500-year event are 14.5 and 15.3 ft, respectively.

The peak discharge from the 100- and 500-year inland flow hydrograph is very small in comparison to the storage volume in the estuary. In this case, adding the inland peak discharge to the maximum tidal discharge will be a conservative estimate of the maximum discharge and maximum average velocity in the waterway. If the inland inflow into the estuary had been large, the flood could be routed through the estuary using standard hydrologic modeling techniques.

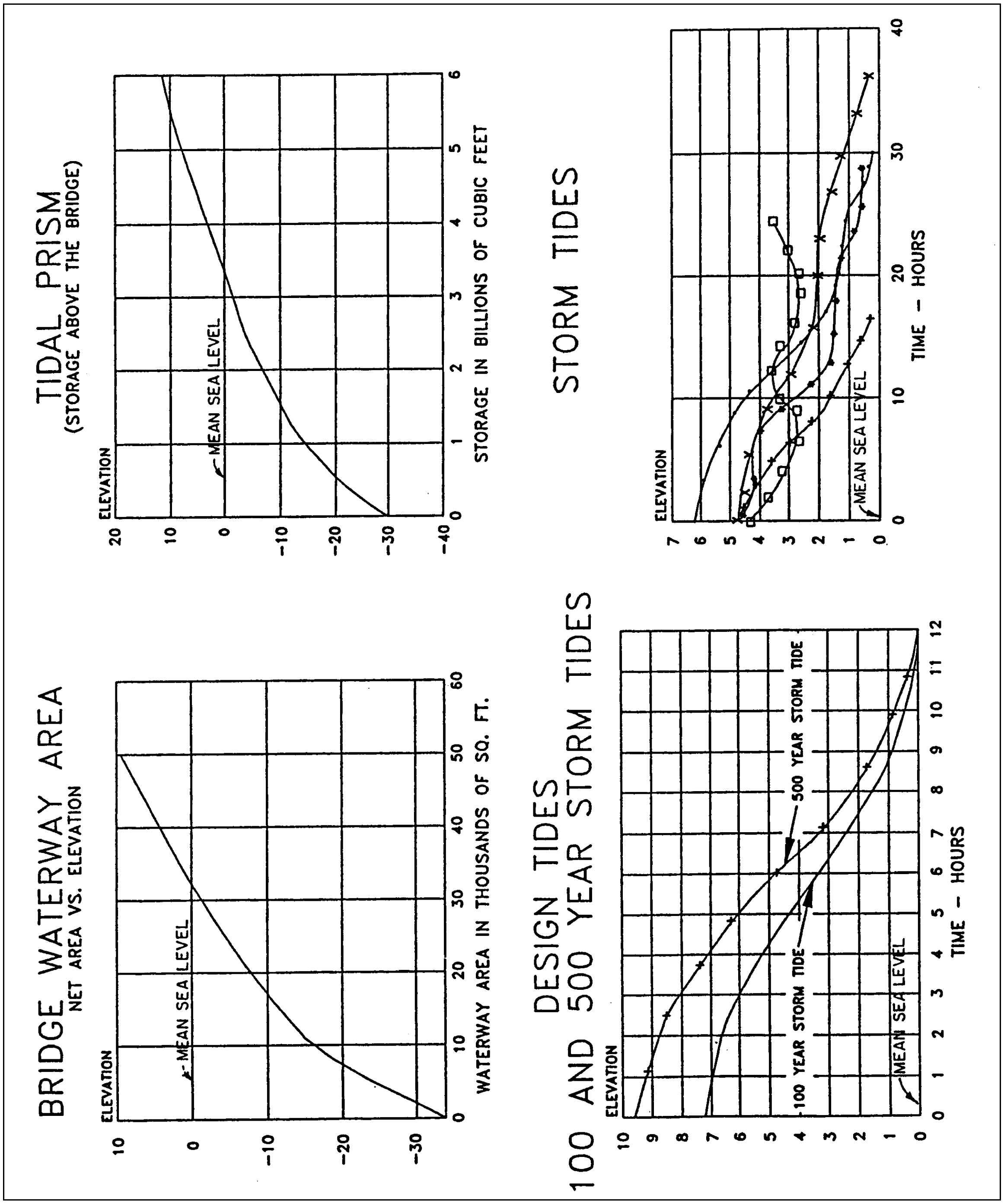


Figure 9.9. Tidal parameters for Example Problem 1 (English).

Table 9.4. Design Discharge and Velocities.		
	100-Year Storm Tide	500-Year Storm Tide
Maximum storm tide elevation, ft	7.19	9.42
Mean storm tide elevation, ft	3.61	4.72
Low storm tide elevation, ft	0.0	0.0
Tidal prism volume, ft^3, Figure 9.9	1,639	2,147
Net waterway area at mean storm tide elevation (A_c), ft^2	39,000	41,000
Tidal period, h	24.0	24.0
Q_{max} (Tidal), ft^3/s (Equation 9.2)	59,550	78,030
V_{max} (Tidal), ft/s (Equation 9.3)	1.54	1.90
Inland peak runoff (discharge), ft^3/s	4,980	7,920
Q_{max} (Tidal plus runoff), ft^3/s	64,530	85,950
V_{max} (Tidal plus runoff), ft/s ($V_{max} = Q_{max}/A_c$)	1.64	2.10
Average flow depth (A_c/width), ft	14.5	15.26

STEP 6. A comparison of the calculated velocities with the measured velocities indicate that they are reasonable. The discharge and velocities given in Table 9.4 are acceptable for determining the scour depths. However, the average velocity will have to be adjusted for the nonuniformity of flow velocity in the vicinity of the bridge to obtain the velocities for determining local scour at the piers.

STEP 7. Calculate the components of total scour using the information collected in the Level 1 and Level 2 analyses.

Long-Term Aggradation/Degradation

The Level 1 analysis indicates that the channel is relatively stable at this time. However, there is an indication that over the past 30 years the channel has degraded approximately 5.0 ft. Therefore, for this evaluation, an estimate of long-term degradation of approximately 5.0 ft for the future will be assumed.

Contraction Scour

Contraction scour depends on whether the flow will be clear-water or live-bed. Equation 5.1 is used to determine the critical velocity for the 100-year hydraulics.

$$V_c = 11.17\,(14.50)^{1/6}\,(0.00089)^{1/3} = 1.68\text{ ft / s}$$

This indicates that the 100-year storm surge combined with the inland flow may result in velocities greater than or equal to the critical velocity; therefore, contraction scour will most likely be live-bed. This conclusion is made considering that velocities in excess of the average velocity will be expected due to the nonuniformity of the velocity in the bridge opening, as determined during the Level 1 analysis.

Applying the live-bed contraction scour equation, it is noted that the ratio of discharges is equal to unity (i.e., there is no overbank flow). Therefore, the contraction scour will be influenced by the contraction resulting from the bridge piers reducing the flow width at the bridge crossing. Using Equation 5.2, and assuming that the mode of sediment transport is mostly suspended load (k_1=0.69), the estimate of live-bed contraction scour for the 100-year event is:

$$\frac{y_2}{14.50} = \left[\frac{2685}{2493}\right]^{0.69} = 1.05$$

$$y_2 = 15.26\ \text{ft}$$

$$y_s = 15.26 - 14.50 = 0.76\ \text{ft}$$

Therefore, the contraction scour for the 100-year event is approximately 0.76 ft. Recomputation for the 500-year event with an average flow depth of 15.26 ft results in an estimate of contraction scour of approximately 0.80 ft.

Local Scour at Piers

The hydraulic analysis estimates average velocities in the bridge cross section only. Because of this, an estimate of the maximum velocity at the bridge pier is made to account for non-uniform velocity in the bridge cross section. The average velocity will be increased by 30 percent since velocities for normal flows (Level 1) indicated that the maximum velocity was observed to be approximately 30 percent greater than the average. Therefore the maximum velocity for the 100- and 500-year event are 2.13 and 2.72 ft/s, respectively.

K_1, K_2, and K_4 equal 1.0. K_3 will be equal to 1.1 since the bed condition at the bridge is plane-bed. The depth of flow at the pier for the 100- and 500-year storm surge is determined by adding the mean storm tide elevation from Table 9.4 to the flow depth at the pier referenced to mean sea level (12.3 ft). From this, y_1 will be equal to 15.9 and 17.0 ft for the 100- and 500-year storm surge, respectively.

Applying Equation 6.1 for the 100-year event:

$$\frac{y_s}{15.9} = 2.0\,(1.0)\,(1.0)\,(1.1)\,(1.0)\left[\frac{12.0}{15.9}\right]^{0.65}(0.094)^{0.43} = 0.66$$

From the above equation, the local scour at the piers is 10.5 ft. Considering the 500-year event, local pier scour is 11.8 ft.

9.9.2 Example Problem 2 - Constricted Waterway (English)

This problem presents a Level 2 analysis of a bridge over a tidal inlet where the waterway constricts the flow. In addition, it illustrates how depletion of sediment supplied to the tidal inlet can result in a continual and severe long-term degradation. The length of the inlet is 1,500 ft, the width of the bridge opening and inlet is 410 ft, Manning's n is 0.03, depth of flow at mean water level is 20.0 ft and area A_c is 8,200 ft^2. The D_{50} of the bed material is 0.30 mm and the D_m (1.25 D_{50}) is 0.375 mm (0.0012 ft).

From tidal records, the long-term average difference in elevation from the ocean to the bay, through the waterway, averaged for both the flood and ebb tide is 0.6 ft. The difference in elevation for the 100-year storm surge is 1.8 ft and for the 500-year storm surge is 2.9 ft.

a. Determine the long-term potential degradation that may occur because construction of jetties has cut off the delivery of bed sediments from littoral drift to the inlet.

For this situation, long-term degradation can be approximated by assuming clear-water contraction scour and using the average difference in water surface between the ocean and bay for astronomical tides. The hydraulic computation uses the orifice equations (Equations 9.5 through 9.10).

Using Equation 9.8, determine R (assume K_o = 0.7 and K_b = 1.0 for this location).

$$R = 0.7 + 10 + \frac{2(32.2)(0.03)^2 1500}{(149)^2 (20.0)^{4/3}}$$

$$R = 2.42$$

From Equation 9.7 determine C_d

$$C_d = \left(\frac{1}{2.42}\right)^{1/2}$$

$$C_d = 0.643$$

Using Equation 9.5, determine V_{max}

$$V_{max} = 0.643\sqrt{(2)(32.2)(0.6)}$$

$$V_{max} = 4.0\,\text{ft / s}$$

Using Equation 9.6 determine Q_{max}

$$Q_{max} = V_{max}\,A_c = 4.0\,(8{,}200)$$

$$Q_{max} = 32{,}800\,\text{cfs}$$

Potential long-term degradation for fine bed material is determined using the clear-water contraction scour equation (Equation 5.4):

$$y = \left[\frac{0.0077\,(32{,}800)^2}{(0.0012)^{2/3}\ (410)^2}\right]^{3/7} = 36.3\,\text{ft}$$

$$y_s = 36.3 - 20.0 = 16.3\,\text{ft}$$

Discussion of Potential Long-Term Degradation

This amount of scour would occur in some time period that would depend on the amount of sediment that was available from the bay and ocean side of the waterway to satisfy the transport capacity of the back and forth movement of the water from the flood and ebb tide.

Even if there was no sediment inflow into the waterway, the time it would take to reach this depth of scour is not known.

To determine the length of time would require the use of an unsteady tidal model, and conducting a sediment continuity analysis (see Section 9.6). Using a tidal model and sediment continuity analysis, calculate the amount of sediment eroded from the waterway during a tidal cycle and determine how much degradation this will cause. Then using this new average depth, recalculate the variables and repeat the process. Knowing the time period of the tidal cycle, then the time to reach a scour depth of 16.3 ft could be estimated for the case of no sediment inflow into the waterway. Estimates of sediment inflow in a tidal cycle could be used to determine the time to reach the above estimated contraction scour depth when there is sediment inflow.

When the long-term degradation reaches 16.3 ft, the scouring may not stop. The reason for this is that the discharge in the waterway is not limited, as in the case of inland rivers, but depends on the amount of flow that can enter the bay in a half tidal cycle. As the area of the waterway increases the flood tide discharge increases because, as an examination of Equations 9.5 and 9.6 show the velocity does not decrease. There may be a slight decrease in velocity because the difference in elevation from the ocean and the bay might decrease as the area increases. However, R in Equation 9.8 decreases with an increase in depth.

Although the above discussion would indicate that long-term degradation would increase indefinitely, this is not the case. As the scour depth increases there would be changes in the relationship between the incoming tide and the tide in the bay or estuary, and also between the tide in the bay and the ocean on the ebb tide. This could change the difference in elevation between the bay and ocean. At some level of degradation the incoming or out-going tides could pick up sediment from either the bay or ocean which would then satisfy the transport capacity of the flow. Also, there could be other changes as scour progressed, such as accumulation of larger bed material on the surface (armor) or exposure of scour resistance rock which would decrease or stop the scour.

In spite of these limiting factors, the above problem illustrates the fact that with tidal flow, in contrast to river flow, as the area of the cross section increases from degradation there may be no decrease in velocity and discharge.

b. Determine V_{max}, Q_{max} for the 100-year storm surge and a depth of 20.0 ft.

The values of R and C_d do not change.

$$V_{max} = 0.643\sqrt{(2)(32.2)18}$$

$$V_{max} = 6.92\,\text{ft/s}$$

$$Q_{max} = 6.92(8200) = 56{,}744\,\text{ft}^3/\text{s}$$

These values or similar ones depending on the long-term scour depth, would be used to determine the local scour at piers and abutments using equations given previously. These values could also be used to calculate contraction scour resulting from the storm surge.

CHAPTER 10

NATIONAL SCOUR EVALUATION PROGRAM

10.1 INTRODUCTION

The State departments of transportation (DOTs) have been conducting scour evaluations of their bridges over water in accordance with the 1991 FHWA Technical Advisory T 5140.23.[9] A scour screening started in 1988 as the result of Technical Advisory T 5140.20 which was superseded by T 5140.23[9] (see Appendix I). The evaluation is to be conducted by an interdisciplinary team of hydraulic, geotechnical and structural engineers who can make the necessary engineering judgments to determine the vulnerability of a bridge to scour. In general, the program consisted of screening all bridges over water to determine their scour vulnerability, and setting priorities for their evaluation. Each DOT structured its own evaluation program using guidelines furnished by FHWA. The screening and evaluation has helped bridge owners in rating each bridge in the National Bridge Inventory (NBI) using rating codes for item 113, Scour Critical Bridges.[10] A description of Item 113 rating codes is given in Appendix J along with the other codes for rating bridge foundations, i.e., Item 60 - Substructures, Item 61 - Channel and Channel Protection, Item 71 - Waterway Adequacy, Item 92 - Critical Feature Inspection, Item 93 - Critical Feature Inspection Date.

As of November 2000, virtually all bridges (99.9 percent) had received an initial screening and more than 90 percent of all bridges had been evaluated for scour. More than half of the DOTs have reported a 90 percent or better completion percentage for the evaluation of all their bridges over waterways.

10.1.1 The Scour Evaluation Program

The scour evaluation program consisted of:

1. Screening all bridges over water to determine:

    a. Whether or not a bridge is vulnerable to scour damage; i.e., whether the bridge is a low risk, scour susceptible, or scour critical bridge; and

    b. Priorities for making bridge scour evaluations.

    c. Scour screening to involve an office review and, if needed, a field inspection.

2. Evaluations consisted of:

    a. Review of bridge plans (when available) to determine foundation types, the elevation of footings and pile tips and the subsurface soils or rock on which the bridge is founded. If plans are not available, other sources of information, such as bridge inspection reports, were reviewed for available information. In some cases, the bridge foundations were unknown (see Appendix K). State DOTs have reported over 89,000 bridges with unknown foundations, meaning that the foundation type, material and/or tip elevations are unknown.

b. Development of hydrologic and hydraulic information for use in estimating scour at the bridge foundations.

c. Review of office files, inspection reports and other available information regarding previous actions taken to maintain and protect the bridge over its service life.

d. Conducting a field inspection to evaluate present conditions and to assess potential problems, which may occur during a future flood event.

e. Evaluation by the interdisciplinary team of the ability of the bridge to resist the anticipated scour based on the above findings, and the rating of the bridge under Item 113, Scour Critical Bridges.

f. An interdisciplinary team consisting of a DOT's structural engineer, geotechnical engineer, hydraulic engineer, and bridge engineer.

3. Developing a plan of action for bridges identified by the interdisciplinary team as scour critical.

Scour evaluation required a broader scope of study and effort than those considered in a bridge inspection. The major purpose of the bridge inspection is to identify changed conditions which may reflect an existing or potential problem. The scour evaluation program has served as the mechanism to design new bridge foundations for scour and to evaluate the condition of existing bridge foundations through an engineering process.

In the following sections the results, to date, of the DOTs screening and evaluation of their bridges is given followed by a general description of the screening and evaluation process.

10.2 SCOUR EVALUATION RESULTS (1988 to 2000)

Bridges screened by the bridge owner as scour susceptible or scour critical needed to be evaluated for scour vulnerability. The evaluation was conducted by either (1) an assessment based on an office review of inspection reports and judgment and/or (2) an analysis using guidelines presented in this manual and HEC-20,[6] "Stream Stability at Highway Structures." Generally, the evaluation was accomplished by an interdisciplinary team comprised of hydraulic, structural, geotechnical engineers. Figure 10.1 shows a summary of the status of scour evaluations as of November 2000. Bridges with unknown foundations and over tidal waters are currently being evaluated by many State DOTs.

10.3 SCOUR SCREENING AND EVALUATION PROCESSES

Each DOT developed its own program for conducting its scour evaluations. In general the following approach was used by the DOTs to assess the vulnerability of existing bridges to scour:

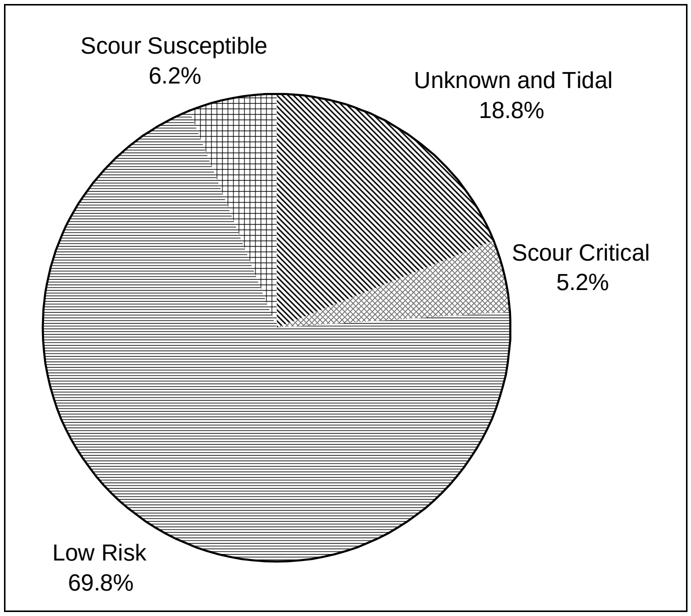


Figure 10.1. Scour evaluation status (as of November 15, 2000).

STEP 1. All bridges over waterways were screened into five categories: (1) low risk, (2) scour susceptible, (3) scour critical, (4) unknown foundations, or (5) tidal. Bridges which were particularly vulnerable to scour failure were identified immediately and the associated scour problem addressed. These particularly vulnerable "scour susceptible" bridges were:

a. Bridges currently experiencing scour or that have a history of scour problems during past floods as identified from maintenance records and experience, bridge inspection records, etc.

b. Bridges over streams with erodible streambeds with design features that make them vulnerable to scour, including:

- Piers and abutments designed with spread footings or short pile foundations;

- Superstructures with simple spans or nonredundant support systems that render them vulnerable to collapse in the event of foundation movement; and

- Bridges with inadequate waterway openings or with designs that collect ice and debris. Particular attention was given to structures where there are no relief bridges or embankments for overtopping, and where all water must pass through or over the structure.

c. Bridges on aggressive streams and waterways, including those with:

- Active degradation or aggradation of the streambed;

- Significant lateral movement or erosion of streambanks;

- Steep slopes or high velocities;

- Instream sand and gravel and other materials mining operations in the vicinity of the bridge; and

- Histories of flood damaged highways and bridges.

d. Bridges located on stream reaches with adverse flow characteristics, including:

- Crossings near stream confluences, especially bridge crossings of tributary streams near their confluence with larger streams;

- Crossings on sharp bends in a stream; and

- Locations on alluvial fans.

STEP 2. Scour susceptible bridges and bridges with unknown foundations (See Appendix K) were prioritized by conducting a preliminary office and field examination of the list of bridges compiled in Step 1, using the following factors as a guide:

a. The potential for bridge collapse or for damage to the bridge in the event of a major flood; and

b. The functional classification of the highway on which the bridge is located, and the effect of a bridge collapse on the safety of the traveling public and on the operation of the overall transportation system for the area or region.

STEP 3. Field and office scour evaluations were conducted on the bridges prioritized in Step 2 using an Interdisciplinary Team of hydraulic, geotechnical, and structural engineers:

a. The evaluation procedure estimated scour for a superflood, a flood exceeding the 100-year flood, and then analyzed the foundations for vertical and lateral stability for this condition of scour. This evaluation approach was the same as the check procedure set forth in Section 2.2, Step 8. An overtopping flood was used where applicable. The difference between designing a new bridge and assessing an old bridge is simply that the location and geometry of a new bridge and its foundation are not fixed as they are for an existing bridge. Thus, the same steps for predicting scour at the piers and abutments were carried out for an existing bridge as for a new bridge. As with the design of a new bridge, engineering judgment was exercised in establishing the total scour depth for an existing bridge. The maximum scour depths that the existing foundation can withstand was compared with the total scour depth. An engineering assessment was made as to whether the bridge should be classified as a scour critical bridge; that is, whether the bridge foundations will be unstable if the estimated scour were to occur.

b. The results of the scour evaluation study was entered into the bridge inventory in accordance with the instructions in the FHWA "Recording and Coding Guide" (see Appendix J).[10] The following codes were used:

- Bridges assessed as "low risk" for Item 113 (Scour Critical Bridges) were coded as an "9, 8, 7, 5, or 4."

- Bridges with unknown foundations (except for interstate bridges) were coded as a "U" in Item 113, indicating that a scour evaluation/calculation has not been made.

- Bridges over tidal waterways were coded "T" and monitored with the regular inspection cycle and with appropriate underwater inspections. These bridges in the most part have been evaluated.

- Bridges assessed to be "scour susceptible" are coded as "6" for Item 113 until such time that further scour evaluations determine foundation conditions.

- Interstate bridges with unknown foundations or over tidal waterways are coded as 6.

- Bridges considered scour critical based on an assessment or calculation are coded as a 3 for Item 113. Bridges coded as scour critical, based on an observed condition are coded as 2, 1, or 0.

STEP 4. Bridges identified as scour critical from the office and field review or during a bridge inspection in Step 2 should have a plan of action developed for correcting the scour problem (see Chapter 12). This plan of action should include:

a. Specific instructions regarding the type and frequency of inspections to be made at the bridge, particularly in regard to monitoring the performance and closing of the bridge, if necessary, during and after flood events.

b. A schedule for the timely design and construction of scour countermeasures determined to be needed for the protection of the bridge.

STEP 5. After completing the scour evaluations for the list of potential problems compiled in Step 1, the remaining waterway bridges included in the State's bridge inventory should be evaluated. In order to provide a logical sequence for accomplishing the remaining bridge scour evaluations, another bridge list should be established, giving priority status to the following:

a. The functional classification of the highway on which the bridge is located with highest priorities assigned to arterial highways and lowest priorities to local roads and streets.

b. Bridges that serve as vital links in the transportation network and whose failure could adversely affect area or regional traffic operations.

The ultimate objectives of the scour evaluation program are to (1) evaluate all bridges over streams in the National Bridge Inventory, (2) determine those foundations which are stable for estimated scour conditions and those which are not, and (3) provide scour protection for scour critical bridges until the bridge can be made safe from scour. This may include scour protection to reduce the risk such as riprap, closing the bridge during high water, monitoring of scour critical bridges during, and inspection after flood events. The final objective (4) would be to replace the bridge or install properly designed scour countermeasures in a timely manner, depending upon the perceived risk involved.

STEP 6. Bridge owners have come to recognize that the rating of bridges for Item 113, Scour Critical Bridges, and the prioritization of bridges for installation of scour countermeasures are not a one-time effort. There is a continuing need to review the Item 113 rating of all bridges during routine inspections and especially after flood events.

A rating of "low risk" for a structure may be changed to "scour critical" after the occurrence of a single flood for a number of reasons including (1) lateral migration of the channel, (2) head cutting and channel degradation with resultant exposure of pile foundations, (3) shifting of the channel thalweg so that a severe angle of attack develops for a pier or abutment which increases local scour. Similarly, a scour critical bridge protected with riprap may require immediate attention after a flood if the riprap is displaced and scour undermines pier or abutment foundations. The bridge inspector should be trained to recognize changes to the river and the effect of such changes on the bridge foundation. The inspector can code Item 113 for the observed scour condition if scour calculations are available to compare the observed with the existing condition. The inspector is charged with notifying his (her) supervisors when significant changes are noticed. The interdisciplinary team should promptly inspect the changed conditions so that appropriate action, commensurate with the perceived risk, can be initiated. The bridge should then be immediately recoded for Item 113 and the related items pertaining to scour and bridge and channel stability set forth in Appendix J.

10.4 UNKNOWN FOUNDATIONS

Bridges are classified as having unknown foundations when the type (spread footing, piles, columns), material (steel, concrete, or timber), dimensions (length, width, or thickness), reinforcing, and/or elevation are unknown. They are classified as "U" in Item 113 of the Coding Guide (Appendix J). The screening program in the national evaluation program has identified about 89,000 bridges with unknown foundations. Research under the National Cooperative Highway Research Program (NCHRP) has investigated nondestructive testing methods which in many cases can determine pile length. Appendix K provides a status report and guidance for a plan of action for protecting bridges with unknown foundations from scour.

CHAPTER 11

INSPECTION OF BRIDGES FOR SCOUR

11.1 INTRODUCTION

There are two main objectives to be accomplished in inspecting bridges for scour:

1. Accurately record the present condition of the bridge and the stream, and

2. Identify conditions that are indicative of potential problems with scour and stream stability for further review and evaluation by others.

In order to accomplish these objectives, the inspector needs to recognize and understand the interrelationship between the bridge, the stream, and the floodplain. Typically, a bridge spans the main channel of a stream and perhaps a portion of the floodplain. The road approaches to the bridge are typically on embankments which obstruct flow on the floodplain. This overbank or floodplain flow must, therefore, return to the stream at the bridge and/or overtop the approach roadways. Where overbank flow is forced to return to the main channel at the bridge, zones of turbulence are established and scour is likely to occur at the bridge abutments. Further, piers and abutments may present obstacles to flood flows in the main channel, creating conditions for local scour because of the turbulence around the foundations. After flowing through the bridge, the flood water will expand back to the floodplain, creating additional zones of turbulence and scour.

The following sections present guidance for the bridge inspector's use in developing an understanding of the overall flood flow patterns at each bridge inspected. Guidance on the use of this information for rating the present condition of the bridge and evaluating the potential for damage from scour is also presented. When an actual or potential scour problem is identified by a bridge inspector, the bridge should be further evaluated by an Interdisciplinary Team using the approach discussed in Chapter 10. The results of this evaluation should be recorded under Item 113 of the "Recording and Coding Guide" (Appendix J).[8, 9, 10]

If the bridge is determined to be scour critical, a Plan of Action (Chapter 12) should be developed for installing scour countermeasures. Also, the rating of the bridge substructure (Item 60 of the Recording and Coding Guide) should be consistent with the rating of Item 113 for the observed scour on the substructure.[10]

11.2 OFFICE REVIEW

It is desirable to make an office review of bridge plans and previous inspection reports prior to making the bridge inspection. Information obtained from the office review provides a better basis for inspecting the bridge and the stream. Items for consideration in the office review include:

1. Has an engineering scour evaluation study been made? If so, is the bridge scour-critical?

2. If the bridge is scour-critical, has a Plan of Action been developed?

3. What do comparisons of streambed cross sections taken during successive inspections reveal about the streambed? Is it stable? Degrading? Aggrading? Moving laterally? Are there scour holes around piers and abutments?

4. What equipment is needed (rods, poles, sounding lines, sonar, etc.) to obtain streambed cross sections?

5. Are there sketches and aerial photographs to indicate the planform location of the stream and whether the main channel is changing direction at the bridge?

6. What type of bridge foundation was constructed? (Spread footings, piles, drilled shafts, etc.) Are footing and pile tip elevations known? Do the foundations appear to be vulnerable to scour? What are the sub-surface soil conditions? (sand, gravel, silt, clay rock?)

7. Do special conditions exist requiring particular methods and equipment (divers, boats, electronic gear for measuring stream bottom, etc.) for underwater inspections?

8. Are there special items that should be looked at? (Examples might include damaged riprap, stream channel at adverse angle of flow, problems with debris, etc.)

11.3 BRIDGE INSPECTION

11.3.1 Safety Considerations

The bridge inspection team should understand and practice prudent safety precautions during the conduct of the bridge inspection. Warning signs should be set up at the approaches to the bridge to alert motorists of the activity on the bridge. This is particularly important if streambed measurements are to be taken from the bridge, since most bridges have minimal clearances between the parapet and the edge of the travel lane. Inspectors should wear brightly colored vests so that they are conspicuous to motorists.

When measurements are made in the stream, the inspector should be secured by a safety line whenever there is deep or fast flowing water and a boat should be available in case of emergency. If waders become overtopped, they will fill and may drag the inspector downstream and under water in a matter of a few seconds.

The inspection team should leave word with their office regarding their schedule of work for the day. The team should also carry a cell phone with them so that they can get immediate help in the event of an emergency.

11.3.2 FHWA Recording and Coding Guide

During the bridge inspection, the condition of the bridge waterway opening, substructure, channel protection, and scour countermeasures should be evaluated, along with the condition of the stream.

The FHWA Recording and Coding Guide (Appendix J) contains guidance for the following items:[10]

1. Item 60: Substructure
2. Item 61: Channel and Channel Protection
3. Item 71: Waterway Adequacy
4. Item 113: Scour Critical Bridges

The guidance in the Recording and Coding Guide for rating the present condition of Items 61, 71, and 113 is set forth in detail. Guidance for rating the present condition of Item 60, Substructure, is general and does not include specific details for scour; however, the rating given to Item 60 should be consistent with the one given for Item 113 whenever a rating of 2 or below is determined for Item 113.

The following sections present approaches to evaluating the present condition of the bridge foundation for scour and the overall scour potential at the bridge.

11.3.3 General Site Considerations

In order to appreciate the relationship between the bridge and the river it is crossing, observation should be made of the conditions of the river up- and downstream of the bridge:

- Is there evidence of general degradation or aggradation of the river channel resulting in unstable bed and banks?
- Is there evidence of on-going development in the watershed and particularly in the adjacent floodplain that could be contributing to channel instability?
- Are there active gravel or sand mining operations in the channel near the bridge?
- Are there confluences with other streams? How will the confluence affect flood flow and sediment transport conditions?
- Is there evidence at the bridge or in the up- and downstream reaches that the stream carries large amounts of debris? Is the bridge superstructure and substructure streamlined to pass debris, or is it likely that debris will hang up on the bridge and create adverse flow patterns with resulting scour?
- The best way of evaluating flow conditions through the bridge is to look at and photograph the bridge from the up- and downstream channel. Is there a significant angle of attack of the flow on a pier or abutment?

11.3.4 Assessing the Substructure Condition

Item 60, Substructure, is the key item for rating the bridge foundations for vulnerability to scour damage. When a bridge inspector finds that a scour problem has already occurred, it should be considered in the rating of Item 60. Both existing and potential problems with scour should be reported so that a scour evaluation can be made by an interdisciplinary team. The scour evaluation is reported on Item 113 in the Recording and Coding Guide.(10) If the bridge is determined to be scour critical, the rating of Item 60 should be consistent to that of Item 113 to ensure that existing scour problems have been considered. The following items are recommended for consideration in inspecting the present condition of bridge foundations:

1. Evidence of movement of piers and abutments;
    - Rotational movement (check with plumb line)
    - Settlement (check lines of substructure and superstructure, bridge rail, etc., for discontinuities; check for structural cracking or spalling)
    - Check bridge seats for excessive movement

2. Damage to scour countermeasures protecting the foundations (riprap, guide banks, sheet piling, sills, etc.). Examples of damage could include riprap placed around piers and/or abutments that has been removed or replaced with river run bed material. A common cause of damage to abutment riprap protection is runoff from the ends of the bridge which flows down to the riprap and undermines it. This condition can be corrected by installing bridge-end drains.

3. Changes in streambed elevation at foundations (undermining of footings, exposure of piles), and

4. Changes in streambed cross section at the bridge, including location and depth of scour holes.

   - Note and measure any depressions around piers and abutments

   - Note the approach flow conditions. Is there an angle of attack of flood flow on piers or abutments?

In order to evaluate the conditions of the foundations, the inspector should measure the elevation of the streambed to a common bench mark at the bridge cross section during each inspection. These cross-section elevations should be plotted to a common datum and successive cross sections compared. Careful measurements should be made of scour holes at piers and abutments, probing soft material in scour holes to determine the location of a firm bottom. If equipment or conditions do not permit measurement of the stream bottom, this condition should be noted for further action.

11.3.5 Assessing Scour Potential at Bridges

The items listed in Table 11.1 are provided for bridge inspectors' consideration in assessing the adequacy of the bridge to resist scour. In making this assessment, inspectors need to understand and recognize the interrelationships between Item 60 (Substructure), Item 61 (Channel and Channel Protection), Item 71 (Waterway Adequacy), and 113 (Scour-Critical Bridges). As noted earlier, additional follow-up by an interdisciplinary team should be made utilizing Item 113 (Scour Critical Bridges) when the bridge inspection reveals a potential problem with scour (Appendix J).

11.3.6 Underwater Inspections

Perhaps the single most important aspect of inspecting the bridge for actual or potential damage from scour is taking and plotting of measurements of stream bottom elevations in relation to the bridge foundations. Where conditions are such that the stream bottom cannot be accurately measured by rods, poles, sounding lines or other means, other arrangements, such as underwater inspections, need to be made to determine the stream bottom elevation around the foundations and to determine the condition of the foundations. Other approaches to determining the cross section of the streambed at the bridge include:

1. Use of divers

2. Use of electronic scour detection equipment (HEC-23[7])

Table 11.1. Assessing the Scour Potential at Bridges.

1. UPSTREAM CONDITIONS

a. Banks

STABLE: Natural vegetation, trees, bank stabilization measures such as riprap, paving, gabions; channel stabilization measures such as dikes and jetties.

UNSTABLE: Bank sloughing, undermining, evidence of lateral movement, damage to stream stabilization measures etc.

b. Main Channel

- Clear and open with good approach flow conditions, or meandering or braided with main channel at an angle to the orientation of the bridge.
- Existence of islands, bars, debris, cattle guards, fences that may affect flow.
- Aggrading or degrading streambed.
- Evidence of movement of channel with respect to bridge (make sketches, take pictures).
- Evidence of ponding of flow.

c. Floodplain

- Evidence of significant flow on floodplain.
- Floodplain flow patterns - does flow overtop road and/or return to main channel?
- Existence and hydraulic adequacy of relief bridges (if relief bridges are obstructed, they will affect flow patterns at the main channel bridge).
- Extent of floodplain development and any obstruction to flows approaching the bridge and its approaches.
- Evidence of overtopping approach roads (debris, erosion of embankment slopes, damage to riprap or pavement, etc.).
- Evidence of ponding of flow.

d. Debris

- Extent of debris in upstream channel.

e. Other Features

- Existence of upstream tributaries, bridges, dams, or other features, that may affect flow conditions at bridges.

Table continues

Table 11.1. Assessing the Scour Potential at Bridges (continued).

2. CONDITIONS AT BRIDGE

a. Substructure

- Is there evidence of scour at piers?
- Is there evidence of scour at abutments (upstream or downstream sections)?
- Is there evidence of scour at the approach roadway (upstream or downstream)?
- Are piles, pile caps or footings exposed?
- Is there debris on the piers or abutments?
- If riprap has been placed around piers or abutments, is it still in place?

b. Superstructure

- Evidence of overtopping by flood water (Is superstructure tied down to substructure to prevent displacement during floods?)
- Obstruction to flood flows (Does superstructure collect debris or present a large surface to the flow?)
- Design (Is superstructure vulnerable to collapse in the event of foundation movement, e.g., simple spans and nonredundant design for load transfer?)

c. Channel Protection and Scour Countermeasures

- Riprap (Is riprap adequately toed into the streambed or is it being undermined and washed away? Is riprap pier protection intact, or has riprap been removed and replaced by bed-load material? Can displaced riprap be seen in streambed below bridge?)
- Guide banks (Spur dikes) (Are guide banks in place? Have they been damaged by scour and erosion?)
- Stream and streambed (Is main current impinging upon piers and abutments at an angle? Is there evidence of scour and erosion of streambed and banks, especially adjacent to piers and abutments? Has stream cross section changed since last measurement? In what way?)

d. Waterway Area Does waterway area appear small in relation to the stream and floodplain? Is there evidence of scour across a large portion of the streambed at the bridge? Do bars, islands, vegetation, and debris constrict the flow and concentrate it in one section of the bridge or cause it to attack piers and abutments? Do the superstructure, piers, abutments, and fences, etc., collect debris and constrict flow? Are approach roads regularly overtopped? If waterway opening is inadequate, does this increase the scour potential at bridge foundations?

Table continues

Table 11.1 Assessing the Scour Potential at Bridges (continued).
3. DOWNSTREAM CONDITIONS a. Banks STABLE: Natural vegetation, trees, bank stabilization measures such as riprap, paving, gabions, channel stabilization measures such as dikes and jetties. UNSTABLE: Bank sloughing, undermining, evidence of lateral movement, damage to stream stabilization measures, etc. b. Main Channel • Clear and open with good "getaway" conditions, or meandering or braided with bends, islands, bars, cattle guards, debris, and fences that retard and obstruct flow. • Aggrading or degrading streambed. • Evidence of movement of channel with respect to the bridge (make sketches and take pictures). • Evidence of extensive bed erosion. c. Floodplain • Clear and open so that contracted flow at bridge will return smoothly to floodplain, or restricted and blocked by dikes, development, trees, debris, or other obstructions. • Evidence of scour and erosion due to downstream turbulence. d. Other Features • Downstream dams or confluence with larger stream which may cause variable tailwater depths. (This may create conditions for high velocity flow through bridge.)

For the purpose of evaluating resistance to scour of the substructure under Item 60 of the Recording and Coding Guide, the questions remain essentially the same for foundations in deep water as for foundations in shallow water:[10]

1. What is the configuration of the stream cross section at the bridge?

2. Have there been any changes as compared to previous cross section measurements? If so, does this indicate that (1) the stream is aggrading or degrading; or (2) local or contraction scour is occurring around piers and abutments?

3. What are the shapes and depths of scour holes?

4. Is the foundation footing, pile cap, or the piling exposed to the stream flow; and if so, what is the extent and probable consequences of this condition?

5. Has riprap around a pier been moved or removed?

Technical Advisory T5140.21[96] contains additional guidance for underwater inspections by divers.

11.3.7 Notification Procedures

A positive means of promptly communicating inspection findings to proper agency personnel must be established. **Any condition that a bridge inspector considers to be of an emergency or potentially hazardous nature should be reported immediately**. That information as well as other conditions which do not pose an immediate hazard, but still warrant further action, should be conveyed to the interdisciplinary team for review.

A report form is, therefore, needed to communicate pertinent problem information to the hydraulic, structural, and geotechnical engineers. An existing report form may currently be used by bridge inspectors within a DOT to advise maintenance personnel of specific needs. Regardless of whether an existing report is used or a new one is developed, a bridge inspector should be provided the means of advising the interdisciplinary team of problems in a timely manner.

11.3.8 Post-Inspection Documentation

Following completion of the bridge inspection, the new channel cross section should be compared with the cross sections taken during previous inspections. The results of the comparison should be evaluated and documented. Many bridge inspectors now utilize lap top computers to facilitate the documentation of the inspection findings. Computers will also facilitate plotting of successive channel cross-sections to enable rapid evaluation of the changes. A bridge scour expert system, CAESAR,[97] is available to assist in this process.

11.4 CASE HISTORIES OF BRIDGE INSPECTION PROBLEMS

11.4.1 Introduction

Since 1987 there have been three bridge failures with loss of life that illustrate the importance of bridge inspections. In two of the failures inspectors failed to observe changed conditions that if corrected may have saved the bridge. In one case, the inspectors documented the changes, but there was no follow-up action to evaluate the changes and to protect the bridge. In the following sections, the inspection problems associated with these bridge failures are described and issues related to inspection are highlighted.

11.4.2 Schoharie Creek Bridge Failure

On April 5, 1987 the New York State Thruway Authority Bridge (I-90) over Schoharie Creek collapsed killing 10 persons[98,99] (see also HEC-23,[7] Design Guideline 8). The National Transportation Safety Board investigated the collapse and gave as the probable cause as:

> "............the failure of the New York State Thruway Authority to maintain adequate rip rap around the bridge piers, which led to severe erosion in the soil beneath the spread footings. Contributing to the accident were ambivalent plans and specifications used for construction of the bridge, an inadequate NYSTA bridge inspection program, and inadequate oversight by the New York State Department of Transportation and the Federal Highway Administration. Contributing to the severity of the accident was the lack of structural redundancy in the bridge."

The bridge was built in 1953 on piers with spread footings and no piles. The footings were 1.5 m (5 ft) deep, 5.5 m (18 ft) wide and 25 m (82 ft) long. The tops of the footings were at the streambed and incised into a substrate consisting of ice contact stratified draft (glacial till). The footings were protected by riprap. In 1955 the bridge survived a larger flood (2084 m^3/s (73,600 cfs)) than the 1987 flood (1759 m^3/s (62,100 cfs)). However, from 1953 to 1987 the bridge was subjected to many floods which progressively removed riprap from the piers, enabling the spread footings to be undermined during the April 1987 flood (Figures 11.1 and 11.2).

The NYSTA inspected the bridge annually or biennially with the last inspection on April 1, 1986. A 1979 inspection by a consultant hired by NYSDOT indicated that most of the riprap around the piers was missing (Figures 11.1 and 11.2); however, the 1986 inspection failed to detect any problems with the condition of the riprap at the piers. Based on the Safety Board findings, the conclusions from this failure are that inspectors and their supervisors must recognize that riprap does not necessarily make a bridge safe from scour, and inspectors must be trained to recognize when riprap is missing and the significance of this condition.

11.4.3 Hatchie River Bridge Failure

On April 1, 1989 the northbound U.S. Route 51 bridge over the Hatchie River in Tennessee collapsed killing eight persons[100,101] (see also HEC-23,[7] Design Guideline 1). The National Transportation Safety Board investigated the collapse and gave as the probable cause:

> ".........the northward migration of the main river channel which the Tennessee Department of Transportation failed to evaluate and correct. Contributing to the severity of the accident was the lack of redundancy in the design of the bridge spans."

A 2-lane bridge on Route 51 was opened to traffic in 1936. It was (1,219 m (4,000 ft)) long and spanned the main channel (approximately 91 m (300 ft)) and the majority of the floodplain. In 1974 a second 2-lane (southbound) bridge was added. Its length was 305 m (1,000 ft) and centered approximately on the main channel downstream from the northbound bridge. The earthfull approaches to the new southbound bridge blocked the floodplain flow that had formerly moved through the open bents of the 1936 (northbound) bridge. This concentrated the flow in both bridges and caused the main channel to move northward and into the floodplain bents of the northbound bridge.

Each of the floodplain bents of the 1936 (northbound) bridge was on a pile cap (bottom elevation 237.9 ft) supported by five untreated wooden piles 6 m (20 ft) long. The main channel bridge was on piers with a pile cap (bottom elevation 223.67 ft) supported on 6 m (20 ft) long precast concrete piles. The northward movement of the channel exposed the piles of the bent next to the channel to local pier scour and it collapsed dropping three spans. The channel migration was documented by Tennessee DOT and U.S. Army Corps of Engineers (USACE) data.[101] At the time of the collapse the flow was not large 244 m^3/s (8,620 cfs) but the flow was overbank and of long duration. The maximum flood peak for the 1989 flood season was (813 m^3/s (28,700 cfs)) with a 3-year recurrence interval.

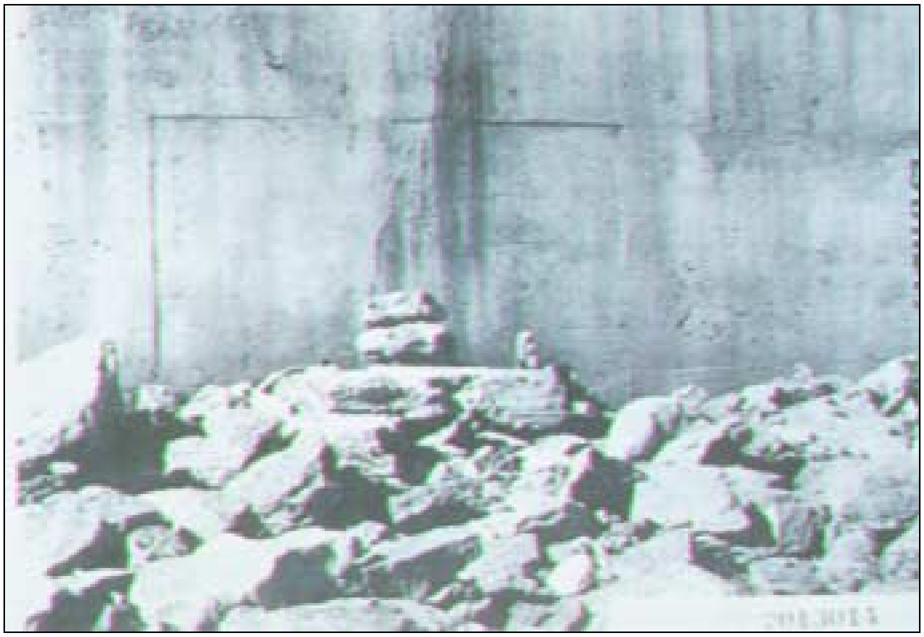


Figure 11.1. Photograph of riprap at pier 2, October 1956.[98,99]

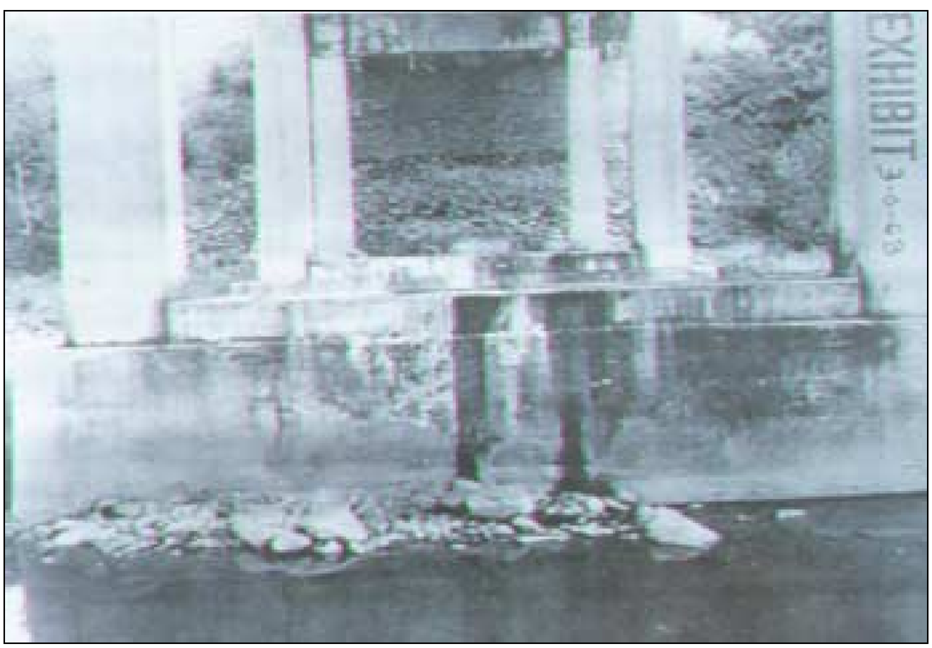


Figure 11.2. Photograph of riprap at pier 2, August 1977 (flow is from right to left).[98,99]

Since 1975, the bridge had been inspected on 24 to 26 month intervals and the last inspection was in September 1987. The NTSB report stated "the 1979, 1985, and 1987 inspection reports accurately identified the channel migration around column bent 70," (the floodplain bent that failed). The report further stated "....on-site inspections of the northbound U.S. 51 Bridge adequately identified the exposure of the column bent footings and piles due to the northward migration of the Hatchie River channel." The report also noted that the inspectors did not have design or as-built plans with then during the inspection. Because of this, the inspectors were mistaken in the thickness of the pile cap and calculated that 0.3 m (1 ft) of the bent piles was exposed. Whereas, the piles were actually exposed .9 m (3 ft) in 1987. The Safety Board noted other (unrelated) bridge collapses where inspectors did not have design or as-built plans, and as a result, deficiencies were overlooked that contributed to bridge failures. Therefore, the Safety Board believes that "it is essential for inspectors to have available bridge design or as-built plans during the on-site bridge inspection."

The NTSB noted that although TDOT inspectors measured the streambed depth at each substructural element and the USACE maintained historical channel profile data at the bridge "a channel profile of the river was not being maintained by TDOT." As a result the TDOT evaluator of the inspection report used only the 1985 and 1987 measurements and would not have been able to determine the extent of channel migration. In other words, if the profiles had been plotted, the evaluator should have easily detected the lateral migration.

The Safety Board also noted that an underwater inspection did not occur in 1987 because the bridge foundation was submerged less than 3 m (10 ft), TDOT criteria at that time. In 1990, TDOT changed the criteria to 1 m (3.5 ft). The Safety Board stated "a diver inspection of the bridge should have been conducted following the 1987 inspection because of the exposure of the untreated timber piles noted in the inspection report."

In conclusion, inspectors should have design or as-built plans on site during an inspection and should measure and plot a profile of the river cross section at the bridge. Submerged bridge elements that can not be examined visually or by feel should have an underwater inspection. Good communication must be established between inspectors, evaluators and decision makers. Changes in the river need to be evaluated through comparisons of successive channel cross sections to determine whether the changes are (1) random and insignificant or (2) represent a significant pattern of change to the channel which may endanger the stability of the bridge.

11.4.4 Arroyo Pasajero Bridge Failure

On March 10, 1995 the two I-5 bridges over Los Gatos Creek (Arroyo Pasajero) in the California Central Valley near Coalinga collapsed killing seven persons and injuring one. CALTRANS retained a team of engineers from FHWA, USGS, and private consultants to investigate the accident. No report was prepared by CALTRANS but three of the investigators, in the interest of bridge engineering, prepared a paper which was published by ASCE.[102] The probable cause of the failure was:

> The minimum scour depth from long-term degradation 3 m (10 ft) from inspection records, contraction scour 2.6 m (8.5 ft) calculated using Laursen's live bed equation, and local pier scour 2 m (6.7 ft) determined from a model study, exposed 2.7 m (8.9 ft) of the cast in place columns below the point where there was steel reinforcement. The force of the flood waters (at an angle of attack of 15 to 26 degrees) on the unreinforced columns, with their area increase by a web wall and debris, caused the bridge to fail.

The bridges, built in 1967, were 37 m (122 ft) long, with vertical wall abutments (with wing walls) and three piers. Each pier consisted of six 406 mm (16 inch) cast in place concrete columns. The columns were spaced 2.3 m (7.5 ft) on centers. They were embedded 12.5 m (41 ft) below original ground surface but only had steel reinforcing for 5.2 m (17 ft) below the original ground surface. The abutments were on pile-supported footings and the piles were 11.3 m (36.7 ft) long. A flood in 1969 lowered the bed 1.83 m (6 ft) and damaged one column. In repairing the damage CALTRANS maintenance constructed a web wall 2.4 or 3.6 m (8 or 12 ft) high, 11.6 m (38 ft) long and 0.6 m (2 ft) wide around the columns to reinforce them. The elevation of the bottom of the web wall was unknown.

Los Gatos Creek is an ephemeral stream (dry most of the time) which drains from the eastern side of the coastal range onto an alluvial fan whose head is approximately 3.2 km (2 mi) upstream of the two bridges. About 548 m (1,800 ft) upstream of the bridges Chino creek (also ephemeral) joins Los Gatos Creek. At the time of construction Chino Creek spread over and infiltrated into its alluvial fan. Some time after construction a channel was constructed connecting the two streams and increasing the drainage area of Los Gatos Creek by about 33 percent.

The Los Gatos Creek channel upstream of the bridge is from 91 to 122 m (300 to 400 ft) wide, but only 46 to 76 m (150 to 250 ft) wide downstream. The 37 m (122 ft) wide bridge severely constricts the channel and the March 10, 1995 flood ponded upstream of the bridge. From 1955 to 1995, differential land subsidence between bench marks approximately 2.4 km (1.5 miles) upstream and 8.5 km (5.3 mi) downstream was measured as 3.5 m (11.5 ft). The bed of the stream is sand and the bedform is plane bed. Discharges are hard to quantify for this stream. For the 1995 flood, the USGS using slope area methods determined that the discharge ranged from 462 to 1141 m^3/s (16,300 to 40,300 cfs) and the most probable discharge was 773 m^3/s (27,300 cfs) with a recurrence interval of 75 years based on historical data.

The factors involved in the I-5 bridge failure were:

- Increase in channel slope by subsidence
- Change in the original design by maintenance adding a web wall between columns to repair damage from an earlier flood. With an angle of attack from 15 to 26 degrees this action potentially increased local pier scour depth by a factor of 3.6 to 4.4
- Increase in drainage area of 33 percent above the bridge by land use change and the construction of a channel to link two streams (Chino Creek to Los Gatos Creek)
- Long-term degradation of 3 m (10 ft) since the bridge was built
- Significant contraction of the flow, i.e., channel width of 91 to 122 m (300 to 400 ft) wide to a bridge width of 37 m (122 ft)

In conclusion, the various factors that contributed to this failure illustrate the complexities of inspection and the need for all elements of a DOT (inspection, maintenance, design and management) to be involved in the process. Inspectors must continually observe the conditions at the bridge, and the stream channel above and below the bridge, and communicate actions, conditions, and changes in the bridge and stream to the different sections of the organization.

11.4.5 Conclusions

These three cases illustrate the difficulty and necessity for inspection of bridges. They also illustrate the need for good communication between DOT inspection, maintenance, design and management. Inspectors must have design or as-built plans on site; must take, plot, and compare cross sections of the channel at the bridge, and they must observe and carefully document the conditions of the bridge and the channel upstream and downstream. Maintenance must inform inspection, design and others when they make changes to a bridge or channel. A "can do" attitude is great but sometimes the consequences can be bad. Communication is very important. Design needs to inform inspection and maintenance of design assumptions and what to look for. Maintenance, because they are the "eyes" of the DOT team, must look for changes and inform others.

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CHAPTER 12

SPECIAL CONSIDERATIONS FOR SCOUR AND STREAM INSTABILITY

12.1 INTRODUCTION

Most bridge owners have now implemented comprehensive programs, inspections and operational procedures to make their bridges less vulnerable to damage or failure from scour. New bridges are designed to resist damage from scour, while existing bridges are inspected regularly and evaluated to determine if a present or potential condition exists that may render the bridge vulnerable to damage during a future flood. When such a condition is found to exist, the bridge is coded as a scour critical bridge, and a plan of action should be developed to address the best way of mitigating the scour problem. Features that make a bridge less vulnerable to damage or failure from scour or stream instability are generally referred to as countermeasures. Countermeasures can be (1) incorporated in the initial design or (2) added after the initial construction.

This chapter outlines special considerations for reducing the risk or making a bridge safe from scour and stream instability. General guidance regarding the use of scour and stream instability countermeasures is provided. Guidance regarding the selection, design and implementation of specific stream instability and scour countermeasures is given in HEC-23.(7) In addition, considerations for evaluating scour in unusual situations, such as scour in cohesive soils or rock, are introduced (with details provided in separate appendices). Cohesive soil and rock can reduce the magnitude of both local scour and general scour at bridge foundations.

12.2 PLAN OF ACTION

A **plan of action** should be developed for each existing bridge found to be scour critical. The two primary components of the plan of action are instructions regarding the type and frequency of inspections to be made at the bridge, and a schedule for the timely design and construction of countermeasures to make a bridge safe from scour and stream stability problems. Depending on the risk, the plan might include development and implementation of a monitoring and/or inspection program, or immediate installation of countermeasures to reduce the risk of failure from scour or stream instability. The plan could include instructions for closure of a bridge, if needed.

HEC-23(7) (Chapter 2) outlines management and inspection strategies that should be considered when developing a plan of action for a scour critical bridge. Issues related to closing and re-opening a bridge are also discussed.

Developing a schedule for the timely design and construction of countermeasures requires defining the preferred countermeasure alternative. It is typical that several different alternatives might be appropriate for a given scour or stream stability problem at a bridge. These alternatives could include hydraulic countermeasures, structural countermeasures or monitoring, either individually or in some combination. To evaluate the engineering feasibility of possible alternatives, conceptual designs and preliminary cost estimates should be prepared. The various alternatives developed should be presented in the plan of action, and a narrative provided describing why the preferred alternative was chosen.

To facilitate selection of alternatives to be considered in the plan of action, a matrix describing the various countermeasures and their attributes has been developed and is presented in HEC-23.[7] HEC-23 also includes general guidance for design of countermeasures, and specific design guidelines for a variety of stream instability and scour countermeasures.

12.3 NEW BRIDGES

For new bridges, the best solutions for minimizing scour damage include:

1. Locating bridges to avoid adverse flood flow patterns

2. Streamlining bridge elements to minimize obstructions to the flow

3. Designing foundations to resist scour, using the guidance in Chapters 2 through 10

4. Designing bridge pier foundations to resist scour without relying on the use of riprap or other countermeasures

5. Designing abutment foundations on piles or on rock, where practicable; for spread footings on soil, placing the footing deep enough to minimize the scour hazard; or protecting the abutment by well designed riprap and/or other suitable countermeasures

6. Incorporating measures to control stream instability (guidebanks, spurs, check dams, etc.) as a part of the initial construction when the potential exists for significant lateral movement or degradation of the channel (see HEC-23)[7]

7. Providing as-built plans (depicting bridge layout, foundations, pile tip elevations, etc.), bridge soils and scour reports and other documented hydrologic and hydraulic design information in a permanent file for the use of bridge maintenance and inspection units. Most DOTS include this information as a part of the permanent bridge plans. The information on design assumptions and site conditions can serve as base line data to evaluate future changes in a river channel and to determine if the changes could affect the safety of the bridge (See examples given in Section 11.4).

12.4 EXISTING BRIDGES

For existing bridges, some of the countermeasures available for protecting the bridge from scour and stream instability are listed below in a rough order of cost (see HEC-23[7] for selection and design guidance):

1. Bridge inspection and scour monitoring programs; closing bridges when necessary

2. Providing riprap at piers and monitoring

3. Providing riprap at abutments and monitoring

4. Constructing guide banks (spur dikes)

5. Constructing river training countermeasures and channel improvements

6. Strengthening the bridge foundations

7. Constructing sills or drop structures (check dams)

8. Constructing relief bridges or lengthening existing bridges

12.5 INSPECTING AND MONITORING BRIDGES FOR SCOUR

Periodic inspections of all bridges serve as the foundation for the bridge owner's management plan to assure the public safety. This includes underwater inspection of foundations located in deep water. Underwater inspection is required when the bridge foundations cannot be visibly inspected by wading.[96] A river and its floodplain are constantly changing, whereas the bridge and its foundation are fixed. A measuring system is necessary to track the lateral and vertical movement of the channel bed over time. The measurements will serve to help in the determination of whether changes are random and within acceptable tolerances, or whether definite trends are occurring which may threaten the stability of the bridge (see Chapter 11).

Gradual river changes are common. As a consequence, the engineer may wait too long to take action. As the degree of encroachment and scour hazard increases, the number of alternative countermeasures available decreases, and costs of correction are correspondingly increased. Threshold values for vertical and horizontal river bed changes should be provided to the inspector. The bridge inspector should report immediately in a special report, as well as the routine inspection report, when changes exceed the threshold values.

Special attention should be given to the condition of scour critical bridges during these periodic inspections. Further, special scour monitoring efforts should be put into effect as necessary to assure that these bridges remain stable. There is a wide range of monitoring procedures which can be used, depending on the condition of the scour critical bridge. The plan of action prepared for each scour critical bridge will serve as the basis for (1) selecting the appropriate monitoring procedures and (2) providing special instructions to the bridge inspector regarding the procedures. Monitoring may include:

- Increasing the frequency and intensity of bridge inspections, using portable scour measuring devices where necessary to check scour critical bridge elements

- Stationing inspectors at the bridge during and immediately after flood events, and providing them with portable equipment to measure scour depths

- Installing permanent scour monitoring equipment at bridge piers and abutments (see HEC-23,[7] Chapter 7)

- Preparing geotechnical stability analyses of bridge piers or abutments to determine the scour depth at which the bridge becomes unstable and should be closed

- Closing the bridge to traffic when conditions become unsafe

The plan of action for a bridge should include special instructions to the bridge inspector, as to when a bridge should be closed to traffic. Guidance should also be given to DOT and other State officials on bridge closures. Contingency plans should be prepared in advance of any bridge closure so that rerouting of traffic can be handled in an orderly fashion.

12.6 COUNTERMEASURES TO REDUCE THE RISK

There are a number of scour critical bridges for which the installation of countermeasures to reduce the risk from scour represents the most practical and cost effective solution. Typical examples of these measures which could reduce, but not eliminate, the scour threat include:

- Placement of riprap around exposed foundations (see Appendix J for guidance)
- Use of grout bags and grout to underpin footings that have been undermined (see HEC-23[7] design guidelines)
- Installation of bendway weirs or spurs at a bend that is migrating towards a bridge abutment so as to redirect the flow away from the abutment (see HEC-23[7] design guidelines)
- Placement of guide banks to move scour away from the abutment foundation

Such countermeasures, if properly installed, may serve successfully for many years in protecting the bridge. While they reduce the risk from scour, they may be subject to failure over an extended period of time or even during a single flood event. They need to be carefully checked during routine inspections and after flood events, especially when used at scour critical bridges.

Installing a scour countermeasure to reduce the risk can serve effectively at bridges where it is not practical or economically justified to undertake repairs to make the bridge safe from scour or to replace the bridge. Examples include:

- Bridge that has only a few years of service life remaining before it is scheduled for replacement
- Small bridges with limited under clearances where it is difficult to install measures to make the bridge safe
- Structures on low volume roads where the risks to the public from a bridge failure are minimal

12.7 COUNTERMEASURES TO MAKE A BRIDGE SAFE FROM SCOUR

Countermeasures to make a bridge safe from scour are distinguished from countermeasures to reduce the risk primarily by the scope of the work involved in their design, installation, and cost. Typically, such countermeasures will be designed on the basis of a hydrologic and hydraulic study of the river to withstand scour associated with a design flood (for scour) and a check flood (for scour). Measures to make a bridge safe from scour include structural changes to the foundations of the bridge. They may also include riprap revetments when designed in accordance with appropriate hydrologic and hydraulic criteria as set forth in HEC-23.[7]

12.8 SCHEDULING CONSTRUCTION OF SCOUR COUNTERMEASURES

It is important for the bridge owner to develop realistic schedules for the installation of scour countermeasures. Lead-time must be provided for the design of the countermeasure and for obtaining necessary permits. Regulatory agencies will usually appreciate the need for emergency work to keep a bridge from failing, and will cooperate in expediting approval of the work (see HEC-23,[7] Chapter 4). However, they are understandably reluctant to consider every scour countermeasure project as emergency work. Coordination with the regulatory agency personnel on a regular basis is needed to assure that the designs for scour countermeasures are prepared in accordance with regulatory requirements. If the installation of a scour countermeasure will require special design procedures that are not in keeping with the normal permit requirements, then this issue needs to be discussed early on in one of the coordination meetings.

The scheduling of scour countermeasure projects should be based on the relative priorities of competing projects. In turn, these priorities should be based, primarily, on the perceived risk to the safety of the persons who travel on the affected highways.

12.9 SCOUR IN COHESIVE SOILS

The maximum depth of local scour at piers in cohesive soils is the same as in non-cohesive soils.[103,104,105] Time is the difference. Maximum scour depth is reached in hours or one runoff event in non-cohesive sand, but may take days and many runoff events in cohesive clays. Local pier scour in cohesive clays may be 1,000 times slower than non-cohesive sand.[103] In addition, by inference, contraction scour and local scour at abutments in cohesive soils do not reach maximum depth as rapidly, but the ultimate scour depth will be the same as for non-cohesive soil.

The equations and methodologies presented in previous chapters, which predict the maximum scour depth in non-cohesive soil, may, in some circumstance be too conservative. The pier scour equation represents an envelope curve of the deepest scour observed during the various laboratory studies and field data. There is much merit in using a conservative approach, taking into consideration the wide range of soil characteristics, the intricate interactions between soil and water, and the uncertainties inherent in predicting flood flows and their flow patterns through the bridge over its service life. When applied with engineering judgment, this conservative approach is usually reasonable and cost effective.

On the other hand, there are site conditions and bridges where an alternative method for scour evaluation would be appropriate. Examples include bridge foundations on highly scour-resistant cohesive soils where the useful life of the bridge is short in relation to the expected number of scouring floods and rate of scour in cohesive soils, bridges scheduled to be replaced in a couple of years, or bridges on low traffic volume roads which are monitored. Significant savings can be achieved for bridges under these conditions, when the characteristics of the cohesive soils to resist scour are taken into account in the design of the foundation. Consequently, guidelines and a technique for evaluating scour in cohesive soils, based on recent research,[103,104] are presented in Appendix L.

12.10 SCOUR IN ROCK

As noted, the equations and methods given in previous chapters are for determining scour depths for the design of bridge foundations in non-cohesive soils. In Chapter 2, recommendations are given for bridge foundations on rock highly resistant to scour. **The problem is determining if rock is resistant to scour**. The determination if the bridge foundations are founded in scour resistance rock and the design of foundations in rock require the expertise of geologist and geotechnical engineers. In addition to standard geologic and geotechnical tests, core or block samples can be taken and subjected to flume studies. The Erosion Function Apparatus (EFA), described in the Appendix L, or a simply constructed or available flume can be used to determine the scourability of the rock material. In Appendix M, four recommendations are given for determining if rock formations are scour resistant; however, additional research is needed in this area.

12.11 OTHER LITERATURE ON SCOUR

Additional information and guidance on stream stability and scour at bridges can be found in several recent publications on these topics. These include a scour manual on European practice from the Netherlands,(93) a book on bridge scour which summarizes the present state of knowledge and practice in New Zealand,(106) and a compendium of papers collected from American Society of Engineers (ASCE) water resources conferences which summarizes research and practice, primarily in the United States, from 1991 to 1998.(107) Highlights of the contents of these publications are indicated in the following paragraphs.

The purpose of the Dutch scour manual(93) is to provide the civil engineer with practical methods to calculate the dimensions of scour holes and to furnish an introduction to the most relevant literature. The manual contains guidelines which can be used to solve problems related to scour in engineering practice and also reflects the results of research projects on the phenomena of scour which have been conducted in the Netherlands during the last several decades.

The manual summarizes and extends the theoretical work of Breusers and Raudkivi, and suggests that the Breusers equilibrium method can be applied directly in engineering practice for all situations where local scour is expected and for nearly all types of structures. Highlights of the manual include:

- Basic concepts
- Sills and jets
- Abutments and spur dikes
- Bridge piers
- Coastal and offshore structures
- Case studies

The New Zealand book on bridge scour covers the description and analysis of scour at bridge foundations. The central focus is the combination of old and new design methods into a comprehensive methodology for bridge-scour design. The book is based upon an extensive summary of existing research results and design experience and it is intended to serve as both a handy reference text and a manual for the practicing bridge designer. A unique aspect of the book is its presentation of thirty-one detailed case studies of scour-induced bridge failure to provide designers with an understanding of processes involved and cases against which design methodologies can be tested. Highlights of the book include:

- New Zealand case histories of bridge scour damage
- Data requirements and basic engineering analyses
- General scour including bend scour and confluence scour
- Contraction and local scour
- Design method for total scour
- Applications and scour countermeasures

The ASCE Compendium contains all the abstracts of the stream stability and scour papers from the proceedings of the Hydraulics Division of the American Society of Civil Engineers annual conferences from 1991 to 1998. Most of the abstracts are from sessions sponsored by the Hydraulic Division's Sedimentation Committee Task Committee on "Bridge Scour Evaluation." In addition, selected authors were invited to write an extended or updated paper on the subject of their original paper. These 75 new papers are included in the Compendium. The abstracts and papers are assembled into the following topics:

- U.S. national bridge scour evaluation program
- Stream stability and geomorphology
- Local scour at bridge piers and abutments
- Contraction scour
- Instrumentation for measuring and monitoring scour
- Field measurements of bridge scour
- Computer and physical modeling of bridge scour
- Bridge scour in tidal waterways
- Countermeasures for stream instability and bridge scour
- Economics and risk analysis of bridge scour
- Research needs

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CHAPTER 13

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APPENDIX A

METRIC SYSTEM, CONVERSION FACTORS, AND WATER PROPERTIES

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APPENDIX A

Metric System, Conversion Factors, and Water Properties

The following information is summarized from the Federal Highway Administration, National Highway Institute (NHI) Course No. 12301, "Metric (SI) Training for Highway Agencies." For additional information, refer to the Participant Notebook for NHI Course No. 12301.

In SI there are seven base units, many derived units and two supplemental units (Table A.1). Base units uniquely describe a property requiring measurement. One of the most common units in civil engineering is length, with a base unit of meters in SI. Decimal multiples of meter include the kilometer (1000m), the centimeter (1m/100) and the millimeter (1 m/1000). The second base unit relevant to highway applications is the kilogram, a measure of mass which is the inertial of an object. There is a subtle difference between mass and weight. In SI, mass is a base unit, while weight is a derived quantity related to mass and the acceleration of gravity, sometimes referred to as the force of gravity. In SI the unit of mass is the kilogram and the unit of weight/force is the newton. Table A.2 illustrates the relationship of mass and weight. The unit of time is the same in SI as in the English system (seconds). The measurement of temperature is Centigrade. The following equation converts Fahrenheit temperatures to Centigrade, $°C = 5/9\ (°F - 32)$.

Derived units are formed by combining base units to express other characteristics. Common derived units in highway drainage engineering include area, volume, velocity, and density. Some derived units have special names (Table A.3).

Table A.4 provides useful conversion factors from English to SI units. The symbols used in this table for metric units, including the use of upper and lower case (e.g., kilometer is "km" and a newton is "N") are the standards that should be followed. Table A.5 provides the standard SI prefixes and their definitions.

Table A.6 provides physical properties of water at atmospheric pressure in SI system of units. Table A.7 gives the sediment grade scale and Table A.8 gives some common equivalent hydraulic units.

Table A.1. Overview of SI Units.		
	Units	Symbol
Base units		
length	meter	m
mass	kilogram	kg
time	second	s
temperature*	kelvin	K
electrical current	ampere	A
luminous intensity	candela	cd
amount of material	mole	mol
Derived units		
Supplementary units		
angles in the plane	radian	rad
solid angles	steradian	sr
*Use degrees Celsius (°C), which has a more common usage than kelvin.		

Table A.2. Relationship of Mass and Weight.			
	Mass	Weight or Force of Gravity	Force
English	slug pound-mass	pound pound-force	pound pound-force
metric	kilogram	newton	newton

Table A.3. Derived Units With Special Names.			
Quantity	Name	Symbol	Expression
Frequency	hertz	Hz	s^{-1}
Force	newton	N	$kg \cdot m/s^2$
Pressure, stress	pascal	Pa	N/m^2
Energy, work, quantity of heat	joule	J	$N \cdot m$
Power, radiant flux	watt	W	J/s
Electric charge, quantity	coulomb	C	$A \cdot s$
Electric potential	volt	V	W/A
Capacitance	farad	F	C/V
Electric resistance	ohm	Ω	V/A
Electric conductance	siemens	S	A/V
Magnetic flux	weber	Wb	$V \cdot s$
Magnetic flux density	tesla	T	Wb/m^2
Inductance	henry	H	Wb/A
Luminous flux	lumen	lm	$cd \cdot sr$
Illuminance	lux	lx	lm/m^2

Table A.4. Useful Conversion Factors.			
Quantity	From English Units	To Metric Units	Multiplied By*
Length	mile yard foot inch	km m m mm	1.609 0.9144 0.3048 25.40
Area	square mile acre acre square yard square foot square inch	km^2 m^2 hectare m^2 m^2 mm^2	2.590 4047 0.4047 0.8361 0.09290 645.2
Volume	acre foot cubic yard cubic foot cubic foot 100 board feet gallon cubic inch	m^3 m^3 m^3 L (1000 cm^3) m^3 L (1000 cm^3) cm^3	1233 0.7646 0.02832 28.32 0.2360 3.785 16.39
Mass	lb kip (1000 lb)	kg metric ton (1000 kg)	0.4536 0.4536
Mass/unit length	plf	kg/m	1.488
Mass/unit area	psf	kg/m^2	4.882
Mass density	pcf	kg/m^3	16.02
Force	lb kip	N kN	4.448 4.448
Force/unit length	plf klf	N/m kN/m	14.59 14.59
Pressure, stress, modulus of elasticity	psf ksf psi ksi	Pa kPa kPa MPa	47.88 47.88 6.895 6.895
Bending moment, torque, moment of force	ft-lb ft-kip	N · m kN · m	1.356 1.356
Moment of mass	lb · ft	m	0.1383
Moment of inertia	lb · ft^2	kg · m^2	0.04214
Second moment of area	in^4	mm^4	416200
Section modulus	in^3	mm^3	16390
Power	ton (refrig) Btu/s hp (electric) Btu/h	kW kW W W	3.517 1.054 745.7 0.2931
*4 significant figures; underline denotes exact conversion			

Table A.4. Useful Conversion Factors (continued).			
Quantity	From English Units	To Metric Units	Multiplied by*
Volume rate of flow	ft^3/s cfm cfm mgd	m^3/s m^3/s L/s m^3/s	0.02832 0.0004719 0.4719 0.0438
Velocity, speed	ft/s	m/s	<u>0.3048</u>
Acceleration	f/s^2	m/s^2	<u>0.3048</u>
Momentum	lb · ft/sec	kg · m/s	0.1383
Angular momentum	lb · ft^2/s	kg · m^2/s	0.04214
Plane angle	degree	rad mrad	0.01745 17.45
*4 significant figures; underline denotes exact conversion			

Table A.5. Prefixes.					
Submultiples			Multiples		
deci	10^{-1}	d	deka	10^{1}	da
centi	10^{-2}	c	hecto	10^{2}	h
milli	10^{-3}	m	kilo	10^{3}	k
micro	10^{-6}	μ	mega	10^{6}	M
nano	10^{-9}	n	giga	10^{9}	G
pica	10^{-12}	p	tera	10^{12}	T
femto	10^{-15}	f	peta	10^{15}	P
atto	10^{-18}	a	exa	10^{18}	E
zepto	10^{-21}	z	zetta	10^{21}	Z
yocto	10^{-24}	y	yotto	10^{24}	Y

Table A.6. Physical Properties of Water at Atmospheric Pressure in SI Units.

Temperature		Density	Specific Weight	Dynamic Viscosity	Kinematic Viscosity	Vapor Pressure	Surface Tension[1]	Bulk Modulus
Centigrade	Fahrenheit	kg/m^3	N/m^3	$N \cdot s/m^2$	m^2/s	N/m^2 abs.	N/m	GN/m^2
0°	32°	1,000	9,810	1.79×10^{-3}	1.79×10^{-6}	611	0.0756	1.99
5°	41°	1,000	9,810	1.51×10^{-3}	1.51×10^{-6}	872	0.0749	2.05
10°	50°	1,000	9,810	1.31×10^{-3}	1.31×10^{-6}	1,230	0.0742	2.11
15°	59°	999	9,800	1.14×10^{-3}	1.14×10^{-6}	1,700	0.0735	2.16
20°	68°	998	9,790	1.00×10^{-3}	1.00×10^{-6}	2,340	0.0728	2.20
25°	77°	997	9,781	8.91×10^{-4}	8.94×10^{-7}	3,170	0.0720	2.23
30°	86°	996	9,771	7.97×10^{-4}	8.00×10^{-7}	4,250	0.0712	2.25
35°	95°	994	9,751	7.20×10^{-4}	7.24×10^{-7}	5,630	0.0704	2.27
40°	104°	992	9,732	6.53×10^{-4}	6.58×10^{-7}	7,380	0.0696	2.28
50°	122°	988	9,693	5.47×10^{-4}	5.53×10^{-7}	12,300	0.0679	
60°	140°	983	9,643	4.66×10^{-4}	4.74×10^{-7}	20,000	0.0662	
70°	158°	978	9,594	4.04×10^{-4}	4.13×10^{-7}	31,200	0.0644	
80°	176°	972	9,535	3.54×10^{-4}	3.64×10^{-7}	47,400	0.0626	
90°	194°	965	9,467	3.15×10^{-4}	3.26×10^{-7}	70,100	0.0607	
100°	212°	958	9,398	2.82×10^{-4}	2.94×10^{-7}	101,300	0.0589	

[1]Surface tension of water in contact with air

Table A.7. Physical Properties of Water at Atmospheric Pressure in English Units.

Temperature		Density	Specific Weight	Dynamic Viscosity	Kinematic Viscosity	Vapor Pressure	Surface Tension[1]	Bulk Modulus
Fahrenheit	Centigrade	$Slugs/ft^3$	Weight lb/ft^3	$lb\text{-}sec/ft^2$	ft^2/sec	lb/in^2	lb/ft	lb/in^2
32	0	1.940	62.416	0.374×10^{-4}	1.93×10^{-5}	0.09	0.00518	287,000
39.2	4.0	1.940	62.424					
40	4.4	1.940	62.423	0.323	1.67	0.12	.00514	296,000
50	10.0	1.940	62.408	0.273	1.41	0.18	.00508	305,000
60	15.6	1.939	62.366	0.235	1.21	0.26	.00504	313,000
70	21.1	1.936	62.300	0.205	1.06	0.36	.00497	319,000
80	26.7	1.934	62.217	0.180	0.929	0.51	.00492	325,000
90	32.2	1.931	62.118	0.160	0.828	0.70	.00486	329,000
100	37.8	1.927	61.998	0.143	0.741	0.95	.00479	331,000
120	48.9	1.918	61.719	0.117	0.610	1.69	.00466	332,000
140	60.0	1.908	61.386	0.0979	0.513	2.89		
160	71.1	1.896	61.006	0.0835	0.440	4.74		
180	82,2	1.883	60.586	0.0726	0.385	7.51		
200	93.3	1.869	60.135	0.0637	0.341	11.52		
212	100	1.847	59.843	0.0593	0.319	14.70		

[1]Surface tension of water in contact with air

Table A.8. Sediment Particles Grade Scale.

Size				Approximate Sieve Mesh Openings Per Inch		Class
Millimeters		Microns	Inches	Tyler	U.S. Standard	
4000-2000	-----	-----	160-80	-----	-----	Very large boulders
2000-1000	-----	-----	80-40	-----	-----	Large boulders
1000-500	-----	-----	40-20	-----	-----	Medium boulders
500-250	-----	-----	20-10	-----	-----	Small boulders
250-130	-----	-----	10-5	-----	-----	Large cobbles
130-64	-----	-----	5-2.5	-----	-----	Small cobbles
64-32	-----	-----	2.5-1.3	-----	-----	Very coarse gravel
32-16	-----	-----	1.3-0.6	-----	-----	Coarse gravel
16-8	-----	-----	0.6-0.3	2 1/2	-----	Medium gravel
8-4	-----	-----	0.3-0.16	5	5	Fine gravel
4-2	-----	-----	0.16-0.08	9	10	Very fine gravel
2-1	2.00-1.00	2000-1000	-----	16	18	Very coarse sand
1-1/2	1.00-0.50	1000-500	-----	32	35	Coarse sand
1/2-1/4	0.50-0.25	500-250	-----	60	60	Medium sand
1/4-1/8	0.25-0.125	250-125	-----	115	120	Fine sand
1/8-1/16	0.125-0.062	125-62	-----	250	230	Very fine sand
1/16-1/32	0.062-0.031	62-31	-----	-----	-----	Coarse silt
1/32-1/64	0.031-0.016	31-16	-----	-----	-----	Medium silt
1/64-1/128	0.016-0.008	16-8	-----	-----	-----	Fine silt
1/128-1/256	0.008-0.004	8-4	-----	-----	-----	Very fine silt
1/256-1/512	0.004-0.0020	4-2	-----	-----	-----	Coarse clay
1/512-1/1024	0.0020-0.0010	2-1	-----	-----	-----	Medium clay
1/1024-1/2048	0.0010-0.0005	1-0.5	-----	-----	-----	Fine clay
1/2048-1/4096	0.0005-0.0002	0.5-0.24	-----	-----	-----	Very fine clay

Table A.9. Common Equivalent Hydraulic Units.

Volume

Unit	Equivalent							
	cubic inch	liter	u.s. gallon	cubic foot	cubic yard	cubic meter	acre-foot	sec-foot-day
liter	61.02	1	0.264 2	0.035 31	0.001 308	0.001	810.6 E - 9	408.7 E - 9
U.S. gallon	231.0	3.785	1	0.133 7	0.004 951	0.003 785	3.068 E - 6	1.547 E - 6
cubic foot	1728	28.32	7.481	1	0.037 04	0.028 32	22.96 E - 6	11.57 E - 6
cubic yard	46 660	764.6	202.0	27	1	0.746 6	619.8 E - 6	312.5 E - 6
$meter^3$	61 020	1000	264.2	35.31	1.308	1	810.6 E - 6	408.7 E - 6
acre-foot	75.27 E + 6	1 233 000	325 900	43 560	1 613	1 233	1	0.504 2
sec-foot-day	149.3 E + 6	2 447 000	646 400	86 400	3 200	2 447	1.983	1

Discharge (Flow Rate, Volume/Time)

Unit	Equivalent					
	gallon/min	liter/sec	acre-foot/day	$foot^3$/sec	million gal/day	$meter^3$/sec
gallon/minute	1	0.063 09	0.004 419	0.002 228	0.001 440	63.09 E - 6
liter/second	15.85	1	0.070 05	0.035 31	0.022 82	0.001
acre-foot/day	226.3	14.28	1	0.504 2	0.325 9	0.014 28
$feet^3$/second	448.8	28.32	1.983	1	0.646 3	0.028 32
million gal/day	694.4	43.81	3.068	1.547	1	0.043 82
$meter^3$/second	15 850	1000	70.04	35.31	22.82	1

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APPENDIX B

EXTREME EVENTS

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APPENDIX B

EXTREME EVENTS

B.1 INTRODUCTION

In 1994, AASHTO introduced an entirely new set of specifications based on the concept of load and resistance factor design (LRFD) methodology. The factors were developed from the theory of reliability based upon current statistical knowledge of loads and structural performance. In the evaluation of scour at bridge structures, there are two conditions, or limit states, that are of primary interest in design:

1. Service Limit States, or limit states relating to stress, deformation and cracking

2. Strength Limit States, or limit states relating to strength and stability

The design flood for scour is used in the evaluation of these limit states.

The Extreme-Event Limit States relate to events with return periods in excess of the design life of the bridge. There are generally three such limit states that may involve consideration of the effect of scour at bridges:

1. A flood event exceeding a 100-year flood (The check flood for scour or superflood is used to evaluate scour for this event as described in Chapter 2, a 500-year flood is recommended for the check flood for scour)

2. An earthquake

3. A vessel collision with the bridge

In addition to the above, there are other conditions possibly relating to scour that the designer may determine are significant for a specific watershed, such as ice loads or debris from logging operations, etc.

Events 2 and 3, above, are related to scour with regard to the possibility that they could occur at the same time that a flood event is occurring. The loss of foundation support due to scour could then impact on the stability of the foundation in resisting the earthquake or vessel collision forces. Recommendations for the consideration of the joint-probability of one of these events with a flood event are discussed below.

B.2 CHANGES IN FOUNDATIONS DUE TO LIMIT STATE FOR SCOUR

The consequences of changes in foundation conditions resulting from the design flood for scour shall be considered at strength and service limit states in accordance with the standards set forth in the AASHTO LRFD Specifications.(1)

The consequences of changes in foundation conditions due to scour resulting from the check flood for bridge scour and from hurricanes shall be considered at the extreme event limit state.

Scour is not a force effect, but by changing the conditions of the substructure it may have a significant effect in altering the force effects acting on structures. The AASHTO LRFD Specifications, Section 3, sets forth detailed requirements for applying loads and load factors to bridge foundations. The extreme event limit states and the loads to be applied for these limit states are explained in this section.

The strength and service limit states are used in the design of a bridge foundation. Structures designed to resist damage from scour will be designed under this provision using normal design considerations and factors of safety selected by the foundation engineer. The assumption is made that all material in the scour prism has been removed and is unavailable for foundation support.

Scour shall be considered in extreme event load combinations as outlined below:

Extreme Event I - Load combination including earthquake

This extreme event limit state includes water loads and earthquakes. The probability of a major flood and an earthquake occurring at the same time is very small. Therefore, consideration of basing water loads and scour depths on mean discharges may be warranted (when considering the joint probability of an earthquake and scour). Mean discharges are considered to be normal (non-flood) flows representing the typical or daily flows in the river.

Extreme Event II - Load combination related to ice load, collision by vessels and vehicles, and certain hydraulic loads with a reduced live load other than that which is a part of the vehicular collision load

This extreme event limit state is a load combination for extreme events such as ice loads, collision by vessels and vehicles, and the check flood for scour. Its application for the check flood for scour involves a reduced live load on the structure of 50 percent. The assumption is made that all material in the scour prism has been removed and is unavailable for foundation support. The structure is to remain stable for this condition, but is not required to have any reserve capacity to resist loads.

The recurrence interval of these extreme events is expected to exceed the design life of the bridge. The joint probability of these events is extremely low, and, therefore, the events are specified to be applied separately.

The Engineer is cautioned to consider the following when applying the above noted AASHTO specifications to the evaluation of the joint probability of a flood and another extreme event. These considerations incorporate recommendations from some of the papers presented at a conference on "The Design of Bridges for Extreme Events" sponsored by the Federal Highway Administration in December 1996.(2)

- There are several current studies underway to evaluate the joint probability of extreme events. Until further and more definitive conclusions are drawn from these studies, judgment is necessary in evaluating site-specific factors on a case by case basis that could affect the safety of the traveling public.

- A differentiation must be made between long-term scour (degradation) and short-term scour (local scour and general (contraction) scour). It is reasonable to consider expected long-term degradation in evaluating the joint probability of occurrence of scour with an earthquake or vessel collision event since it is associated with a period of many years.

On the other hand, live-bed local scour and contraction scour may occur only for a period of hours or days before the scour hole refills; consequently, the joint probability of this type of scour with an earthquake or vessel collision is very low. In some cases, clear-water scour holes may occur and not refill or refill very slowly. While the joint probability of the occurrence of a 100-year flood/clear-water scour hole and another extreme event is very low, the engineer may wish to consider a clear-water scour hole associated with a lesser flood event.

- The probability of the simultaneous occurrence of an extreme vessel collision load (by a ship or barge transiting the navigable channel at normal operating speeds) and short-term scour resulting from a 100-year flood is very low and can be neglected as a load combination. The probability of the simultaneous occurrence of a vessel collision load from a single (empty) hopper barge floating in the waterway at the speed of the current and both long- and short-term scour is valid and should be considered in the design where applicable.

B.3 REFERENCES

1. American Association of State Highway and Transportation Officials, 1994, "LRFD Bridge Design Specifications and Commentary," First Edition, Washington, D.C.

2. Federal Highway Administration, 1996, "The Design of Bridges for Extreme Events," Conference Proceedings," Washington, D.C.

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APPENDIX C

CONTRACTION SCOUR AND CRITICAL VELOCITY EQUATIONS

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APPENDIX C

Contraction Scour and Critical Velocity Equations

C.1 CONTRACTION SCOUR

Contraction scour occurs when the flow area of a stream at flood stage is reduced, either by a natural contraction or bridge. It also occurs when overbank flow is forced back to the channel by roadway embankments at the approaches to a bridge. From continuity, a decrease in flow area results in an increase in average velocity and bed shear stress through the contraction. Hence, there is an increase in erosive forces in the contraction and more bed material is removed from the contracted reach than is transported into the reach. This increase in transport of bed material from the reach lowers the natural bed elevation. As the bed elevation is lowered, the flow area increases and, in the riverine situation, the velocity and shear stress decrease until relative equilibrium is reached; i.e., the quantity of bed material that is transported into the reach is equal to that removed from the reach, or the bed shear stress is decreased to a value such that no sediment is transported out of the reach.

In coastal waterways which are affected by tides, as the cross-sectional area increases the discharge from the ocean may increase and thus the velocity and shear stress may not decrease. Consequently, relative equilibrium may not be reached. Thus, at tidal inlets contraction scour may result in a continual lowering of the bed (long-term degradation).

Live-bed contraction scour is typically cyclic; for example, the bed scours during the rising stage of a runoff event and fills on the falling stage. The contraction of flow due to a bridge can be caused by either a natural decrease in flow area of the stream channel or by abutments projecting into the channel and/or piers blocking a portion of the flow area. Contraction can also be caused by the approaches to a bridge cutting off floodplain flow. This can cause clear-water scour on a setback portion of a bridge section or a relief bridge because the floodplain flow does not normally transport significant concentrations of bed material sediments. This clear-water picks up additional sediment from the bed in the bridge opening. In addition, local scour at abutments may well be greater due to the clear-water floodplain flow returning to the main channel at the end of the abutment.

Other factors that can cause contraction scour are (1) natural stream constrictions, (2) long highway approaches to the bridge over the floodplain, (3) ice formations or jams, (4) natural berms along the banks due to sediment deposits, (5) debris, (6) vegetative growth in the channel or floodplain, and (7) pressure flow.

<u>Contraction Scour Equations</u>. There are two forms of contraction scour depending upon the competence of the uncontracted approach flow to transport bed material into the contraction.

Live-bed scour occurs when there is streambed sediment being transported into the contracted section from upstream. In this case, the scour hole reaches equilibrium when the transport of bed material out of the scour hole is equal to that transported into the scour hole from upstream.

Clear-water scour occurs when the bed material sediment transport in the uncontracted approach flow is negligible or the material being transported in the upstream reach is transported through the downstream reach at less than the capacity of the flow. In this case, the scour hole reaches equilibrium when the average bed shear stress is less than that required for incipient motion of the bed material.

Contraction scour equations are based on the principle of conservation of sediment transport (continuity). As scour develops, the shear stress in the contracted section decreases as a result of a larger flow area and decreasing average velocity. For **live-bed** scour, maximum scour occurs when the shear stress reduces to the point that sediment transported in equals the bed sediment transported out and the conditions for sediment continuity are in balance. For **clear-water** scour, the transport into the contracted section is essentially zero and maximum scour occurs when the shear stress reduces to the critical shear stress of the bed material in the bridge cross-section.

C.2 LIVE-BED CONTRACTION SCOUR EQUATION

Live-bed contraction scour occurs at a bridge when there is transport of bed material in the upstream reach into the bridge cross section. With live-bed contraction scour the area of the contracted section increases until, in the limit, the transport of sediment out of the contracted section equals the sediment transported in. Normally, the width of the contracted section is constrained and depth increases until the limiting conditions are reached.

Laursen derived the following live-bed contraction scour equation based on a simplified transport function, transport of sediment in uniform flow upstream and downstream of a long contraction, and other simplifying assumptions.[1]

$$\frac{y_2}{y_1} = \left(\frac{Q_2}{Q_1}\right)^{6/7} \left(\frac{W_1}{W_2}\right)^{k_1} \left(\frac{n_2}{n_1}\right)^{k_2} \qquad \text{(C.1)}$$

$$y_s = y_2 - y_o = \text{(Average scour depth, m)} \qquad \text{(C.2)}$$

where:

y_1	=	Average depth in the upstream main channel, m
y_2	=	Average depth in the contracted section, m
y_o	=	Existing depth in the contracted section before scour, m
Q_1	=	Flow in the upstream channel transporting sediment, m^3/s
Q_2	=	Flow in the contracted channel, m^3/s. Often this is equal to the total discharge unless the total flood flow is reduced by relief bridges, water overtopping the approach roadway, or in the setback area
W_1	=	Bottom width of the upstream main channel, m
W_2	=	Bottom width of main channel in the contracted section, m
n_1	=	Manning's n for upstream main channel
n_2	=	Manning's n for contracted section
k_1 & k_2	=	Exponents determined below depending on the mode of bed material transport

V_*/ω	k_1	k_2	Mode of Bed Material Transport
<0.50	0.59	0.066	Mostly contact bed material discharge
0.50 to 2.0	0.64	0.21	Some suspended bed material discharge
>2.0	0.69	0.37	Mostly suspended bed material discharge

V_* = $(gyS_1)^{1/2}$ shear velocity in the upstream section, m/s
ω = Median fall velocity of the bed material based on the D_{50}, m/s (see Figure 3 in Chapter 4)
g = Acceleration of gravity (9.81 m/s^2)
S_1 = Slope of energy grade line of main channel, m/m
D_{50} = Median diameter of the bed material, m

The location of the upstream section for y_1, Q_1, W_1, and n_1 needs to be located with engineering judgment. If WSPRO is used to obtain the values of the quantities, then the upstream channel section is located a distance equal to one bridge opening from the upstream face of the bridge.

C.3 CLEAR-WATER CONTRACTION SCOUR EQUATIONS

Clear-water contraction scour occurs in a bridge opening when (1) there is no bed material transport from the upstream reach into the downstream reach or (2) the material being transported in the upstream reach is transported through the downstream reach mostly in suspension and at less than capacity of the flow. With **clear-water** contraction scour the area of the contracted section increases until, in the limit, the velocity of the flow (V) or the shear stress (τ_o) on the bed is equal to the critical velocity (V_c) or the critical shear stress (τ_c) of a certain particle size (D) in the bed material. Normally, the width (W) of the contracted section is constrained and the depth (y) increases until the limiting conditions are reached.

Following a development given by Laursen[2] equations for determining the clear-water contraction scour in a long contraction were developed in metric units. For equilibrium in the contracted reach:

$$\tau_o = \tau_c \tag{C.3}$$

where:

τ_o = Average bed shear stress, contracted section, Pa (N/m^2)
τ_c = Critical bed shear stress at incipient motion, Pa (N/m^2)

The average bed shear stress using y for the hydraulic radius (R) and Manning's equation to determine the slope (S_f) can be expressed as follows:

$$\tau_o = \gamma\, y\, S_f = \frac{\rho\ g\ n^2\ V^2}{y^{1/3}} \tag{C.4}$$

For noncohesive bed materials and fully developed clear-water contraction scour, the critical shear stress can be determined using Shields relation[2, 3]

$$\tau_c = K_s\ (\rho_s - \rho)\ g\, D \tag{C.5}$$

The bed in a long contraction scours until $\tau_o = \tau_c$ resulting in

$$\frac{\rho\, g\, n^2\, V^2}{y^{1/3}} = K_s\, (\rho_s - \rho)\, g\, D \qquad \text{(C.6)}$$

Solving for the depth (y) in the contracted section gives

$$y = \left[\frac{n^2\, V^2}{K_s\, (S_s - 1)\, D}\right]^3 \qquad \text{(C.7)}$$

In terms of discharge (Q) the depth (y) is

$$y = \left[\frac{n^2\, Q^2}{K_s\, (S_s - 1)\, D\, W^2}\right]^{3/7} \qquad \text{(C.8)}$$

where:

y	=	Average equilibrium depth in the contracted section after contraction scour, m
S_f	=	Slope of the energy grade line, m/m
V	=	Average velocity in the contracted section, m/s
D	=	Diameter of smallest nontransportable particle in the bed material, m
Q	=	Discharge, m^3/s
W	=	Bottom width of contracted section, m
g	=	Acceleration of gravity (9.81 m/s^2)
n	=	Manning's roughness coefficient
K_s	=	Shield's coefficient
S_s	=	Specific gravity (2.65 for quartz)
γ	=	Unit weight of water (9800 N/m^3)
ρ	=	Density of water (1000 kg/m^3)
ρ_s	=	Density of sediment (quartz, 2647 kg/m^3)

Equations C.7 and C.8 are the basic equations for the **clear-water** scour depth (y) in a long contraction. Laursen, in English units used a value of 4 for K_s $(\rho_s-\rho)g$ in Equation C.5; D_{50} for the size (D) of the smallest nonmoving particle in the bed material and Strickler's approximation for Manning's n ($n = 0.034\ D_{50}^{1/6}$).[2] Laursen's assumption that $\tau_c = 4\ D_{50}$ with $S_s = 2.65$ is equivalent to assuming a Shields parameter $K_s = 0.039$.

From experiments in flumes and studies in natural rivers with bed material of sand, gravel cobbles, and boulders, Shield's coefficient (K_s) to initiate motion ranges from 0.01 to 0.25 and is a function of particle size, Froude Number, and size distribution.[4, 5, 6, 7, 8, 9] Some typical values for K_s for Fr. < 0.8 and as a function of bed material size are (1) K_s = 0.047 for sand (D_{50} from 0.065 to 2.0 mm); (2) K_s = 0.03 for median coarse-bed material (2 mm > D_{50} < 40 mm) and (3) K_s = 0.02 for coarse-bed material (D_{50} > 40 mm).

In metric units, Strickler's equation for n as given by Laursen is $0.041\ D_{50}^{1/6}$, where D_{50} is in meters. Research discussed in HDS 6[3] recommends the use of the effective mean bed material size (D_m) in place of the D_{50} size for the beginning of motion ($D_m = 1.25\ D_{50}$). Changing D_{50} to D_m in the Strickler's equation gives $n = 0.040\ D_m^{1/6}$. Substituting K_s = 0.039 into Equations C.7 and C.8 gives the following equations for y:

$$y = \left[\frac{V^2}{40\ D_m^{2/3}} \right]^3 \quad \text{(C.9)}$$

$$y = \left[\frac{Q^2}{40\ D_m^{2/3}\ W^2} \right]^{3/7} \quad \text{(C.10)}$$

$$y_s = y - y_o = \text{(average scour depth)} \quad \text{(C.11)}$$

where:

A	=	Discharge through contraction, m^3/s
D_m	=	Diameter of the bed material (1.25 D_{50}) in the contracted section, m
W	=	Bottom width in contraction, m
y_o	=	Average existing depth in the contracted section, m

The **clear-water** contraction scour equations assume homogeneous bed materials. However, with clear-water scour in stratified materials, using the layer with the finest D_{50} would result in the most conservative estimate of contraction scour. Alternatively, the clear-water contraction scour equations could be used sequentially for stratified bed materials.

Equations C.8 and C.10 do not give the distribution of the contraction scour in the cross section. In many cases, assuming a uniform contraction scour depth across the opening would not be in error (e.g., short bridges, relief bridges and bridges, with simple cross sections and on straight reaches). However, for wide bridges, bridges on bends, bridges with large overbank flow, or crossings with a large variation in bed material size distribution, the contraction scour depths will not be uniformly distributed across the bridge opening. In these cases, Equations C.7 or C.9 can be used if the distribution of the velocity and/or the bed material is known. The computer program WSPRO uses stream tubes to give the discharge and velocity distribution in the cross section.(10) Using this distribution, Equations C.7 or C.9 can be used to estimate the distribution of the contraction scour depths. Equations C.8 or C.10 are used to determine the average contraction scour depth in the section.

Both the **live-bed** and **clear-water** contraction scour equations are the best that are available and should be regarded as a first level of analysis. If a more detailed analysis is warranted, a sediment transport model like BRI-STARS could be used.(11)

C.4 CRITICAL VELOCITY OF THE BED MATERIAL

The velocity and depth given in Equation C.7 are associated with initiation of motion of the indicated particle size (D). Rearranging Equation C.7 to give the critical velocity (V_c) for beginning of motion of bed material of size D results in

$$V_c = \left[\frac{K_s^{1/2}\,(S_s - 1)^{1/2}\ D^{1/2}\ y^{1/6}}{n} \right] \quad \text{(C.11)}$$

Using K_s = 0.039, S_s = 2.65, and n = 0.041 $D^{1/6}$

$$V_c = 6.19\ y^{1/6}\ D^{1/3} \tag{C.12}$$

where:

V_c	=	Critical velocity above which bed material of size D and smaller will be transported, m/s
K_s	=	Shields parameter
S_s	=	Specific gravity of the bed material
D	=	Size of bed material, m
y	=	Depth of flow, m
n	=	Manning's roughness coefficient

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APPENDIX D

INTERIM PROCEDURE FOR ESTIMATING PIER SCOUR WITH DEBRIS

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APPENDIX D

Interim Procedure for Estimating Pier Scour with Debris

D.1 ASSUMPTIONS

1. Debris aligns with the flow direction and attaches to the upstream nose of a pier. The width of the accumulation, W, on each side of the pier is normal to the flow direction.

2. The trailing end of a long slender pier does not add significantly to pier scour for that portion of the length beyond 12 pier widths. This is consistent with the current guideline in HEC-18 to cut K_2 at L/a = 12.

3. The effect of the debris in increasing scour depths is taken into account by adding a width, W, to the sides and front of the pier. Engineering judgment and experience is used to determine the width, W.

D.2 SUGGESTED PROCEDURE

1. Use K_1 and K_2 = 1.0

2. Project the debris pile and up to twelve pier widths of the pier length normal to the flow direction as follows:

L' = L or 12(a) (whichever is less)

a_{proj} = 2W+a Cosθ or W+a Cosθ + L' Sin θ (whichever is greater)

3. Use K_1, K_2, K_3, K_4, and a_{proj} in the HEC-18 pier scour equation as follows:

$$\frac{y_s}{y_1} = 2.0\,(1.0)(1.0)K_3\,K_4\left(\frac{a_{proj}}{y_1}\right)^{0.65} Fr_1^{0.43}$$

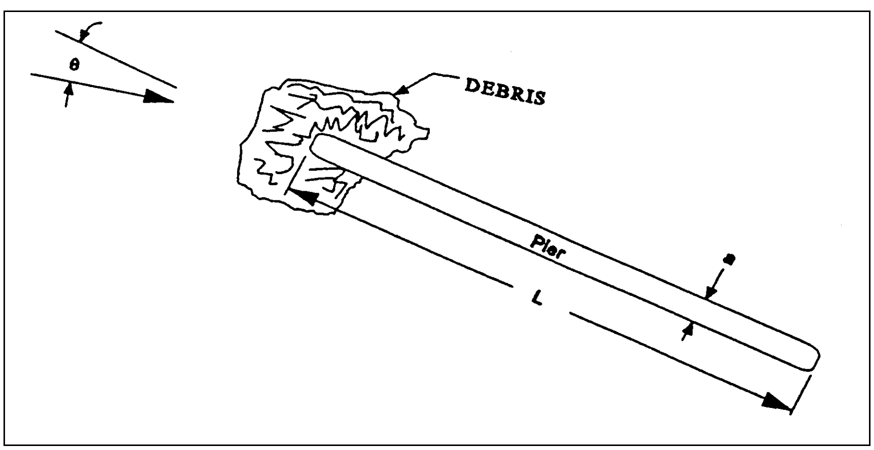

Figure D.1. Schematic for debris procedure.

D3. EXAMPLE PROBLEM (SI)

NVFAS 228 Bridge over the Humbolt River South Fork
Flow: depth, y_1 = 2.42 m; V_1 = 3.60 m/s; Fr_1 = 0.74
Pier: a = 0.46 m; L = 12.62 m; Skew to flow direction = 15 degrees
Debris: Local assumption for accumulation W = 0.61 m extended in front and on each side of pier

Computations:

L/a = 12.62/0.46=27.6>12: use L′ = 12 (0.46) = 5.52 m

a_{proj} = 1.22 + 0.46 (Cos 15°) = 1.66 m or
0.61 + 0.46 (Cos 15°) + 5.52 Sin 15° = 2.48 m

$$\frac{y_s}{2.42} = 2.0\ (1.0)\ (1.0)\ (1.1)\ (1.0) \left(\frac{2.48}{2.42}\right)^{0.65} (0.74)^{0.43}$$

$$y_s = 1.98\,(2.42) = 4.79\,\text{m}$$

D.4 EXAMPLE PROBLEM (English)

NVFAS 228 Bridge over the Humboldt River South Fork
Flow: depth, y_1 = 7.9 ft; V_1 = 11.81 ft/s; Fr_1= 0.74
Pier: a = 1.5 ft; L = 41.4 ft; Skew to flow direction = 15 degrees
Debris: Local assumption for accumulation W = 2.0 ft extended in front and on each side of pier

Computations:

L/a = 41.4/1.5 = 27.6>12: use L′ = 12 (1.5) = 18 ft

a_{proj} = 4.0 + 1.5 (Cos 15°) = 5.4 ft or
2.0 + 1.5 (Cos 15°) + 18 (Sin 15°) = 8.1 ft

use 8.1 ft

$$\frac{y_s}{7.9} = 2.0\ (1.0)\ (1.0)\ (1.1)\ (1.0) \left(\frac{8.1}{7.9}\right)^{0.65} (0.74)^{0.43}$$

$$y_s = 1.96\ (7.9) = 15.5\ \text{ft}$$

APPENDIX E

STURM ABUTMENT SCOUR EQUATIONS

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APPENDIX E

Sturm Abutment Scour Equations

E.1 INTRODUCTION

Sturm[1,2] utilized a flume with a compound channel to evaluate abutment scour. His research was funded by the National Transportation Board's National Cooperative Highway Research Program (NCHRP). He recognized that scour at abutments setback from the bankline or at the bankline depends on the interaction between main channel flow and the flow obstructed by the abutment. At the interface between the two flows is where vortices and momentum exchange occur which cause scour. Sturm determined that the use of a discharge distribution factor (M) is a better measure of the effect of flow redistribution, vortices and momentum exchange on scour at a bridge abutment than abutment length. From his flume experiments he developed equations and a method for determining scour in compound channels. The prediction method shows a strong correlation between predicted scour and measured scour (Figure E.1). The dashed lines of uncertainty represent a difference of +/- 30 percent from the measured value. No factor of safety was applied to the computed values in Figure E.1.

In the following sections the results of his research are given.

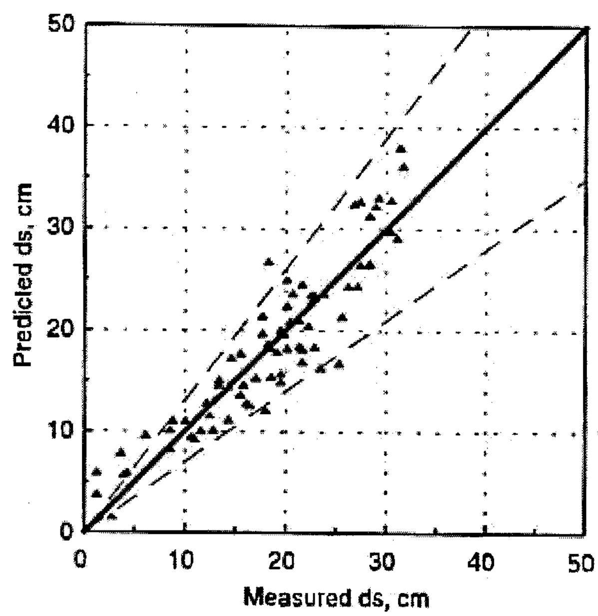

Figure E.1. Comparison of measured and predicted scour depths Sturm Method.[1,2]

E.2 STURM'S EQUATION FOR CLEAR-WATER ABUTMENT SCOUR

Sturm's scour prediction equation for clear-water scour around setback and bankline abutments is:

$$y_s / y_{f0} = 8.14\, K_{st}\, (q_{f1} / MV_{xc}\, y_{f0} - 0.4) + FS \qquad \text{(E.1)}$$

where:

y_s	=	Depth of scour at the abutment, m (ft)
y_{f0}	=	Average depth of flow on the floodplain at the approach section for existing conditions based on normal flow conditions in the river without backwater from the proposed bridge, m (ft)
K_{st}	=	Sturm's abutment shape factor
q_{f1}	=	Unit flow rate on the approach floodplain section that will be blocked by the embankment at Section 2. The conditions are based on the proposed structure in place and creating backwater effects at the approach section, m^3/s /m (cfs /ft)
M	=	Discharge distribution factor as defined below
V_{xc}	=	Critical velocity at the approach floodplain section for existing conditions based on normal flow conditions in the river without backwater from the proposed bridge, m/s (ft/sec)
FS	=	Factor of Safety with a recommended value of 1.0

E.3 STURM'S EQUATION FOR LIVE-BED SCOUR AT BANKLINE ABUTMENTS

$$y_s / y_{f0} = 2.0\, K_{st}\, [q_{m1} / (MV_{m0c}\, y_{f0}) - 0.47] + FS \qquad \text{(E.2)}$$

where:

y_s	=	Depth of scour at the abutment, m (ft)
y_{f0}	=	Average depth of flow on the floodplain (see E.4, Step 5), m (ft)
K_{st}	=	1.0
q_{m1}	=	Unit flow rate in the main channel at the approach Section 1 for the approach critical velocity, i.e., (V_{m1c} x y_{m1}), m^3/s/m (cfs/ft)
M	=	Discharge distribution factor (see E.4, Step 1)
V_{m0c}	=	Critical velocity in the main channel for unconstricted flow at depth y_{m0} (see E.4, Step 8), m/sec (ft/sec)
FS	=	Factor of Safety with a recommended value of 1.0

Note: Equation E.2 is based on experimental results for clear water scour around bankline abutments. Its extension to the live-bed case by assuming threshold live-bed scour is tentative at this time.

E.4 SOLVING STURM'S EQUATIONS

Sturm's equations are solved for through the application of the following steps:

1. Run WSPRO[(3)] or HEC-RAS[(4)] for the condition of the proposed bridge in place, creating a backwater at the approach Section 1 to the bridge. Compute the following for the left and right floodplains <u>in the approach Section 1</u> (Figure E.2) using the output from the water surface profile model to determine the overtopping flow and the flow distribution in the channel and on the floodplain:

M = discharge distribution factor

= $(Q_{1/2\ channel} + Q_{floodplain} - Q_{blocked\ flow})/(Q_{1/2\ channel} + Q_{floodplain})$

in which $Q_{1/2\ channel}$ is the discharge from the centerline to the bank of the main channel in the approach section; $Q_{floodplain}$ is the floodplain discharge in the approach section; and $Q_{blocked\ flow}$ is the floodplain discharge blocked by the embankment in the approach section.

The value of M needs to be determined separately for the right and left floodplains. For this purpose, it is assumed that the flow is divided down the centerline of the channel. The left half of the channel is used to calculate M for the left abutment, and the right half of the channel is used to calculate M for the right abutment. If there is overtopping flow, the denominator in the above equation should include only the flow going under the bridge. The overtopping flow will need to be distributed proportionally (according to the site conditions) between the flows for the left and right abutments.

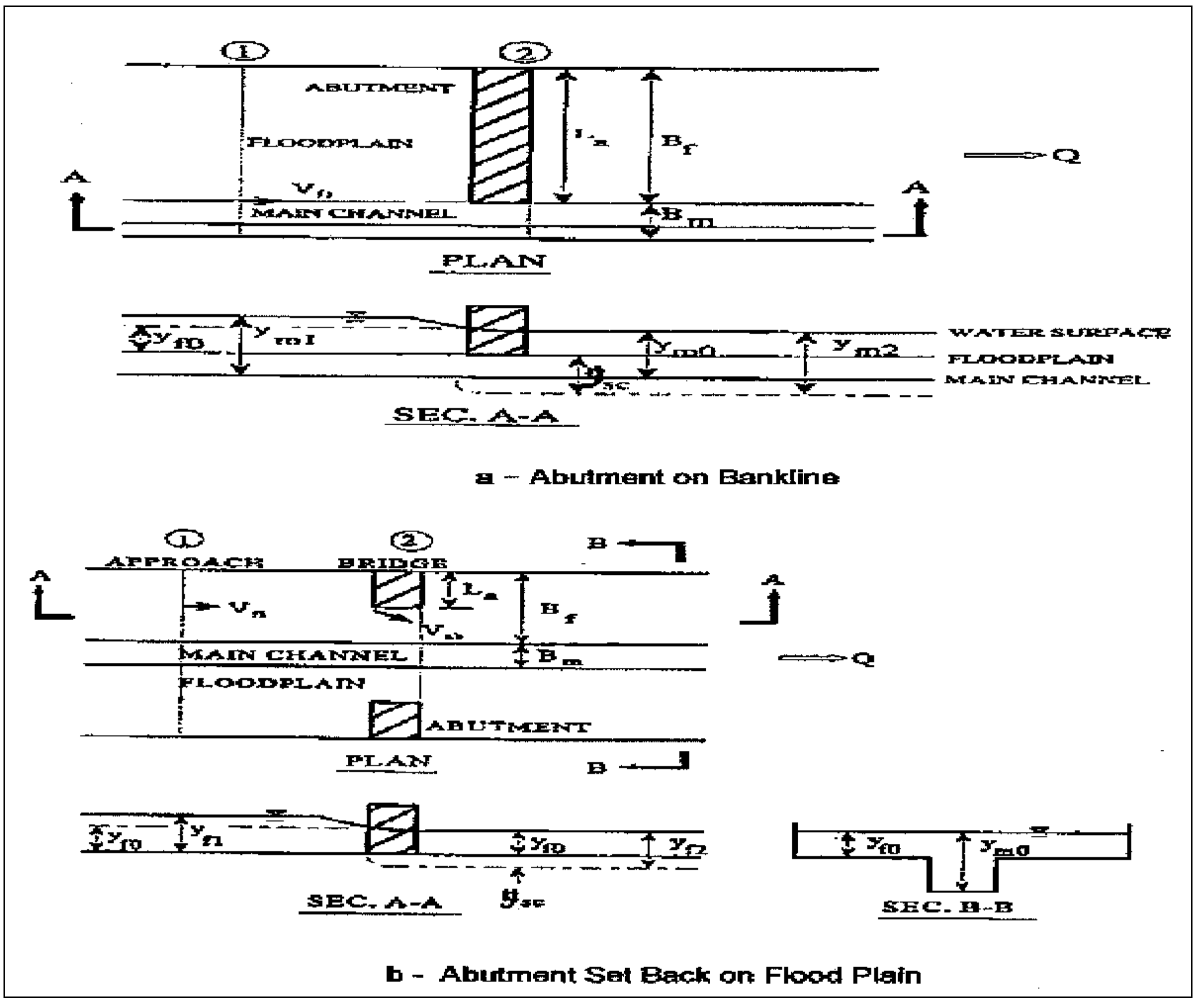

Figure E.2. Definition sketches for application of the Sturm method.

2. y_{f1} = average flow depth in the blocked section of flow in the approach section with a length approximately equal to the distance L_a, m (ft) as determined from the water surface profile model (Figure E.2). It is calculated as the blocked flow area divided by L_a.

3. V_{f1} = average flow velocity in the blocked section = $Q_{blocked\ flow}$ /(L_a x y_{f1}), m/s (ft/sec)

4. $q_{f1} = V_{f1} \times y_{f1}$ m^3/s /m (cfs/ft)

   Next, run WSPRO or HEC-RAS for the existing normal depth condition without the proposed bridge in place and determine the following parameters for the left and right floodplains in the approach Section 1:

5. Compute y_{f0} = average depth of flow on the floodplain, m (ft)

6. Compute the critical velocity of flow, $V_{xc,}$ m/s (ft/sec)

   a. For abutments set back from the channel banks, $V_{xc} = V_{f0c}$). Compute the critical velocity of flow (V_{f0c}) corresponding to the depth of flow, y_{f0} on the floodplain for unconstricted flow and the D_{50} grain size of the floodplain soils using Equation 5.1, Chapter 5.

   b. For abutments at or near the channel banks, $V_{xc} = V_{m0c}$ Compute the critical velocity of the flow (V_{m0c}) from the hydraulic radius of flow of the main channel for unconstricted flow and the D_{50} grain size of the channel bed material using Equation 5.1, Chapter 5.

   c. Compute the critical velocity in the approach Section 1, V_{f1c} or V_{m1c} , for the constricted flow in the same way as for the unconstricted flow except use the approach depth for the constricted flow and determine if the abutment scour will be clear water or live bed by comparing with V_{f1} or V_{m1}.

7. Select the appropriate scour equation:

   a. **Clear-water Scour**

      For clear-water scour, go to Step 8.

   b. **Live-bed Scour for Set Back Abutment**

      If the scour is live-bed scour and the abutment is set back, make the following adjustments: Set $V_{f1} = V_{f1c}$; recompute Step 4 as $q_{fl} = V_{f1c}$ (y_{f1}) and continue to Step 8. (Take into account the effect of floodplain vegetation in estimating V_{f1c}).

   c. **Live-bed Scour for Bankline Abutment**

      If the scour is live bed scour and the abutment is on or near the bankline, use the scour prediction equation for live bed scour at bankline abutments given in Section E.3.

8. Compute the abutment shape factor for the left and right abutments:

   a. Compute the abutment shape factor K_{st} for spill through slopes:

      Compute X_a : $X_a = q_{f1} / (M \; V_{xc} \; y_{f0})$
      q_{f1} from Step 4
      M from Step 1
      V_{xc} from Step 6a or 6b
      y_{f0} from Step 5

      Compute K_{st} :
      $K_{st} = 1.52 (X_a - 0.67)/ (X_a - 0.40)$

      where:

      $0.67 \leq X_a \leq .1.2$

      $K_{st} = 1.0$ where $X_a \geq 1.2$
      $K_{st} = 0.0$ where $X_a \leq 0.67$

   b. For vertical wall abutments, with or without wingwalls, abutment shape factor $K_{st} = 1.0$

9. Compute the value of y_s / y_{f0} and the abutment scour depth, y_s, from Equation E.1.

10. Evaluate the value of $y_s / y_{f0:}$

    Use a maximum value of 10 for y_s / y_{f0}, based on experimental data.

    If V_{f1} (Step 3) equals or exceeds the critical velocity V_{f1c} for setback abutments, then live bed scour occurs and V_{f1} is set equal to V_{f1c} .

    The datum for measuring y_s is the channel bottom. The bottom of the scour hole is a vertical distance of ($y_s + y_{f0}$) below the water surface for existing conditions.

    For bankline abutments, regardless of whether the scour is clear water or live bed, the calculated scour depth includes both abutment scour and contraction scour.

    For bankline abutments, check for the possibility of live bed scour by determining if $V_{m1} \geq V_{m1c.}$ V_{m1} = average velocity in the main channel at the approach section and V_{m1c} = critical velocity in the main channel at the approach section. Compute V_{m1c} by Equation 5.1 using the hydraulic radius of the main channel for constricted flow and the D_{50} particle size of the channel bed material. If $V_{m1} \geq V_{m1c}$, set $V_{m1} = V_{m1c}$ and use the live bed scour procedure equation presented in Section E.3.

E.5 REFERENCES

1. Sturm, T.W., 1999, "Abutment Scour in Compound Channels," ASCE Compendium, Stream Stability and Scour at Highway Bridges, Richardson and Lagasse (eds.), Reston, VA.

2. Sturm, T.W., 1999, "Abutment Scour Studies for Compound Channels," U.S. Department of Transportation, Federal Highway Administration, FHWA Report No. FHWA-RD-99-156, McLean, VA.

3. Arneson, L. and J.O. Shearman, 1998, "User's Manual for WSPRO - A Computer Model for Water Surface Profile Computations," Office of Technology Applications, Federal Highway Administration, FHWA Report No. FHWA-SA-98-080.

4. U.S. Army Corps of Engineers, 1998, "River Analysis System," HEC-RAS, User's Manual Version 2.2, Hydrologic Engineering Center, Davis, CA.

APPENDIX F

MARYLAND ABUTMENT SCOUR EVALUATION METHOD
ABSCOUR

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APPENDIX F

Maryland Abutment Scour Evaluation Method
ABSCOUR

F.1 MARYLAND SHA ABUTMENT SCOUR PROGRAM (ABSCOUR)

Maryland SHA developed a procedure for determining abutment scour based on coefficients applied to contraction scour. The equations and method are presented in this appendix for those states that might want to use the method to compare with the equations and advice given in Chapter 7.

The Maryland SHA abutment scour equations and methods are based on the research and development of Chang.[1, 2] Chang applied Laursen's long contraction theory to both clear-water and live-bed scour. He developed a "velocity adjustment factor" k_v to account for the non-uniform velocity distribution in the contracted section, and a "spiral-flow adjustment factor" k_f at the abutment toe that depends on the approach Froude number. The value of k_v was based on potential flow theory, and k_f was determined by Chang from the analysis of a collection of abutment scour experiments in laboratory flumes.[3]

F.1.2 Live-bed Abutment Scour

For live-bed abutment scour the equation is:

$$\frac{y_{2a}}{y_1} = K_f \left[\frac{k_v q_2}{q_1} \right]^{K_2} \qquad \text{(F.1)}$$

where:

- y_{2a} = Total flow depth in the abutment scour hole after scour has occurred, measured from the water surface to the bottom of the scour hole, m (ft)
- y_1 = Approach flow depth, m (ft)
- q_1 = Flow rate per unit width in the approach section, $m^3/s/m$ ($ft^3/s/ft$)
- q_2 = Flow rate per unit width in contracted section, $m^3/s/m$ ($ft^3/s/ft$) (Determination of q_1 and q_2 is explained in a section below)
- k_v = $0.8\,(q_1/q_2)^{1.5} + 1$
- k_f = $0.35 + 3.2\,F_1$ for live-bed scour

Equation F.1 applies to live-bed scour. It should be used for clear-water scour only for the condition where the shear stress in the approach section (Section 1) is at the critical value.

Values of k_v should range from 1.0 to 1.8. If the calculated value is smaller or larger than this range, use the limiting value.

Values of k_f should range from 1.0 to 3.3. If the calculated value is smaller or larger than this range, use the limiting value.

The Froude number in the approach Section 1 (F_1) = $V_1/(gy_1)^{0.5}$. where V_1 = average flow velocity in the approach floodplain or channel section (m/s or ft/s) and y_1 = average flow depth in the approach floodplain or channel section (m or ft).

K_2 = Laursen's sediment transport function = $0.11\,(\tau_c / \tau_1 + 0.4)^{2.2} + 0.623$ (F.2)

where:

τ_c = Critical shear stress of soil, N/m^2 (lb/ft^2)
τ_1 = Shear stress at approach section, N/m^2 (lb/ft^2), $\tau_1 \geq \tau_c$

The value of K_2 varies from 0.637 to 0.857. If $\tau_c \geq \tau_1$, select a value of K_2 equal to 0.857.

Unpublished studies by Chang have shown that, while K_2 is based on a concept that is similar to the K_1 coefficient in the table accompanying the live-bed contraction scour equation (Equation 5.2), the values of these coefficients are derived in different ways and cannot be mathematically correlated.

Figure F.1 illustrates the variables used in Equations F.1 and F.2. Both equations are non-dimensional and can be used either for English or SI units. The same symbols are used for flow depth in the main channel and floodplain, but the subscript is changed to denote the approach section and the bridge section.

F.1.3 Clear-Water Abutment Scour

Clear-water scour occurs If the shear stress in the approach Section 1 is less than critical, or if the approach section is armored. The clear-water abutment scour equation is as follows:

$$y_{2a} = k_f\,(k_v)^{0.857}\,y_{2c} \qquad \text{(F.3)}$$

where:

y_{2a} = Total depth of flow at the abutment, measured from the water surface down to the bottom of the abutment scour hole, m (ft)
y_{2c} = Clear water contraction scour depth in the channel or on the floodplain (beyond the abutment scour hole) at critical velocity $y_{2c} = q_2/V_c$, m (ft). Equation 5.1 or other similar equations can be used to compute V_c. Another approach would be to compute y_{2c} directly from Laursen's clear-water contraction scour Equation 5.4.
K_v = Dimensionless coefficients as defined above in live-bed scour
K_f = $0.1 + 4.5\,F_1$ for clear-water scour

Equation F.3 can be used either for English units or SI units.

When using Equations F.1 and F.3, the Engineer needs to take into account that the actual field conditions will most likely vary from the simple geometry depicted in Chapter 7 (Figure 7.6). Judgment is necessary in adjusting the theoretical scour to reflect actual field conditions.

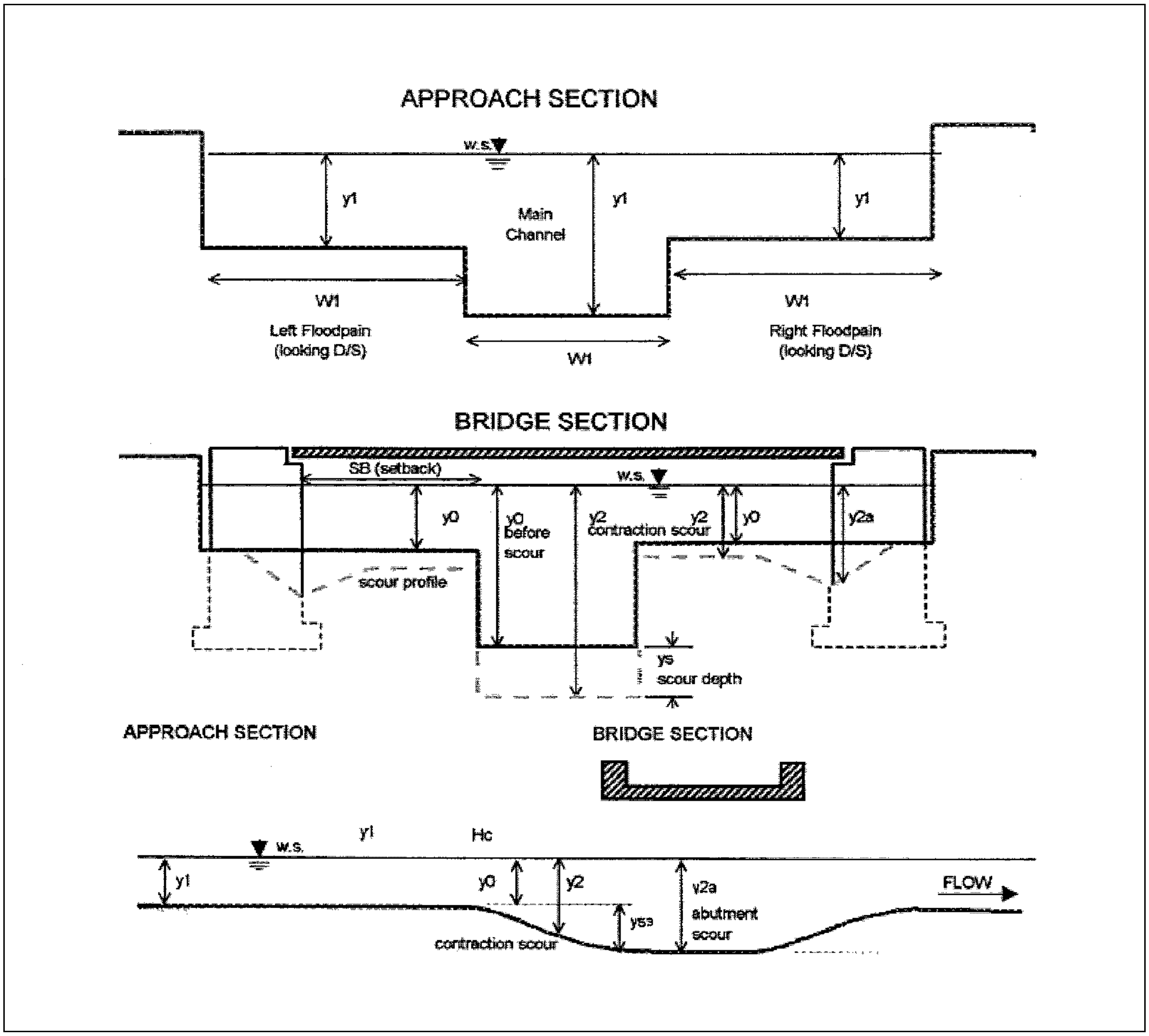

Figure F.1. Definition sketches for scour computations.

F.2 COMPUTATION OF UNIT DISCHARGE

Equations F.1 and F.3 were developed based on simple rectangular geometry for the channel and floodplains (Figure F.1). The method for computing unit discharges at Section 2 in the main channel and on the floodplain under the bridge (for setback abutments) is based on information obtained from the laboratory studies conducted by Sturm and others. The first step in this process is to determine in which category the abutment setback from the channel bank should be placed: **short setback, intermediate setback, or long setback**. The description below is based on the assumption that the left or right floodplain width is essentially the same at Section 1 as it is at Section 2 (Figure F.1). Where there is a significant difference in the floodplain width at Section 1 and Section 2, the Engineer will need to use judgment in selecting the most appropriate method for selecting the unit flow discharge.

F.2.1 Short Setback

If the setback from the main channel bank to the toe of the abutment is equal to or less than five times the depth of flow in the main channel at the bridge, the flow in the main channel and on the floodplain under the bridge is assumed to be mixed flow, having the same velocity. Note that this computation must be made separately for the left and the right floodplains. The average flow velocity through the bridge is computed as $V_{short} = Q_{bridge}/ A_{bridge}$. Q_{bridge} is equal to Q_{total} - $Q_{overtopping}$. A_{bridge} is equal to the total bridge waterway area below the water surface. The unit discharge at any point under the bridge, in the channel or the overbank area, is computed as:

$$q = V_{short}(y) \tag{F.4}$$

where:

q	=	Unit flow rate, m^3/s /m (cfs/ft)
V_{short}	=	Computed average velocity through the bridge determined by the above noted equation $V_{short} = Q_{bridge}/A_{bridge}$, m/s (ft /sec)
y	=	Depth of flow at the point of interest, m (ft)

F.2.2 Long Setback

If the abutment setback is greater than 75 percent of the total floodplain width at the approach section, the assumption is made that the channel flow, Q, at Section 2 under the bridge is the same as the channel flow, Q, at the approach Section 1. Similarly, the flow in the left or right floodplain in the approach Section 1 remains the same in the floodplain section under the bridge. (This is considered to be a conservative assumption.) The unit discharge on the left or right floodplain at Section 1 is computed as $q_1 = Q/W_1$ where Q is the floodplain flow and W_1 is the width of the floodplain. At the bridge Section 2, $q_2 = Q/W_2$ where W_2 is the setback distance to the abutment. It follows that:

$$q_2 = q_1(W_1 / W_2) \tag{F.5}$$

where:

q_z	=	Unit flow rate at setback abutment on floodplain, m^3/s /m (cfs/ft)
q_1	=	Unit flow rate at approach Section 1 on the floodplain, m^3/s /m (cfs/ft)
W_1	=	Width of floodplain at approach Section 1, m (ft)
W	=	Width of floodplain under bridge (abutment setback) at Section 2, m (ft)
V_{long}	=	q_2 / y_2 where y = the depth of flow at the point of interest, m (ft)

F.2.3 Intermediate Setback

In some cases, the abutment setback from the channel bank will be located at a point between the short setback and the long setback described in the forgoing sections. This location is defined as an intermediate setback. An interpolation scheme is used to compute the velocity ($V_{intermediate}$) and corresponding unit discharge ($q_{intermediate}$). This scheme provides for a smooth transition from the velocity associated with the short setback to the velocity associated with the long setback. $V_{intermediate}$ is determined by using the following three steps:

1. Calculate V_{short} at a setback distance equal to five times the channel depth at the bridge (Setback = 5 y_o = SB_{short})

2. Calculate V_{long} at a setback distance equal to 75 percent of the total floodplain width at the approach Section 1 (Setback = 0.75 W1 = SB_{long})

3. Calculate $V_{intermediate} = V_{short} - ((V_{short} - V_{long})/ (SB_{long} - SB_{short}))(SB - SB_{short})$ where SB = setback distance to abutment

The unit discharge, q, is then determined as $V_{intermediate}$ (y), where y is the depth of flow at the abutment.

Equations F.1 and F.3 compute the combined contraction scour and local abutment scour; therefore, contraction scour depths should not be added to the values obtained for scour at the abutment. Measurements of y_{2a} or y_{2c} are made from the water surface to the bottom of the abutment scour hole or to the contracted channel bed elevation, respectively.

The actual depth of abutment scour, y_{sa}, m (ft) is determined from Equation F.1 or Equation F.3 by subtracting the initial flow depth before scour, y_0 , from the flow depth to the bottom of the scour hole, y_{2a} :

$$y_{sa} = y_{2a} - y_0 \qquad \text{(F.6)}$$

F.3 ABUTMENT SHAPE FACTOR (K_t)

The scour depth, y_{sa}, determined in Equation F.6 must be modified by multiplying it by the abutment shape factor. The abutment shape factors given in Chapter 7, Table 7.1 apply only to short abutments in Maryland's abutment scour equations. As the length of the abutment and approach road in the floodplain increase, the effect of a spill through slope in reducing scour is decreased. For long approach road sections on the floodplain, this coefficient will approach a value of 1.0. Similarly, scour for vertical wall abutments with wingwalls on short abutment sections is reduced to 82 percent of the scour of vertical wall abutments without wingwalls. As the length of the abutment and approach road in the floodplain increase, the effect of the wingwall in reducing scour is decreased. For long approach road sections in the floodplain, this coefficient will approach a value of 1.0.

F.3.1 Maryland's Coefficient for Spill-Through Abutments

$$K_t = -.55 + 0.05((L / dL) - 1) \qquad \text{(F.7)}$$

where:

L	=	Total embankment encroachment length from the water's edge on the floodplain to the toe of the spill through slope, m (ft)
dL	=	Distance from the spill through toe to the point where the water surface intersects the spill through slope, m (ft)

If L/dL > 10, K_t = 1.0

F.3.2 Maryland's Coefficient for Vertical Wall with Wingwalls Abutments

$$K_t = 0.82 + 0.02((L / dL) - 1) \qquad \text{(F.8)}$$

where:

- L = Total embankment encroachment length from the water's edge on the floodplain to the face of the abutment, m (ft)
- dL = Distance measured parallel to the embankment from the end of the wingwall to the face of the abutment, m (ft)

If L/dL > 10, K_t = 1.0

F.3.3 Maryland's Coefficient for Vertical Wall without Wingwalls Abutments

For vertical wall abutments without wingwalls, K_t = 1.0

F.4 SKEW ANGLE FACTOR

The scour depth, y_{sa}, determined in Equation F.6 must be modified by multiplying it by the skew angle factor determined in Chapter 7, Section 7.2**.**

F.5 FACTOR OF SAFETY

Comparisons of computed vs. measured scour depths have been made using data from Sturm's tests[4] and other sources (Figure F.2). The lines of uncertainty represent a difference of +/-20 percent from the measured value. The Engineer may wish to apply a Factor of Safety of 20 to 40 percent of the computed scour value to account for this variation. (No Factor of Safety was applied to the computed values).

F.6 ABSCOUR PROGRAM

As noted in Chapter 5, the estimation of contraction scour at bridges involves consideration of a number of variables and becomes a complex process, particularly for Case 1c where the abutments are set back from the channel edge. For this reason, the Maryland SHA procedure for estimating abutment scour has been incorporated in a Windows-type software program entitled ABSCOUR to calculate contraction scour and abutment scour. The program facilitates rapid evaluation of the various factors affecting abutment scour and enables the Engineer to select the conditions and the scour analysis most appropriate for the site under evaluation. Various refinements have been incorporated in the program that would not be practical for use in a manual method. The ABSCOUR program is available from the Maryland SHA.

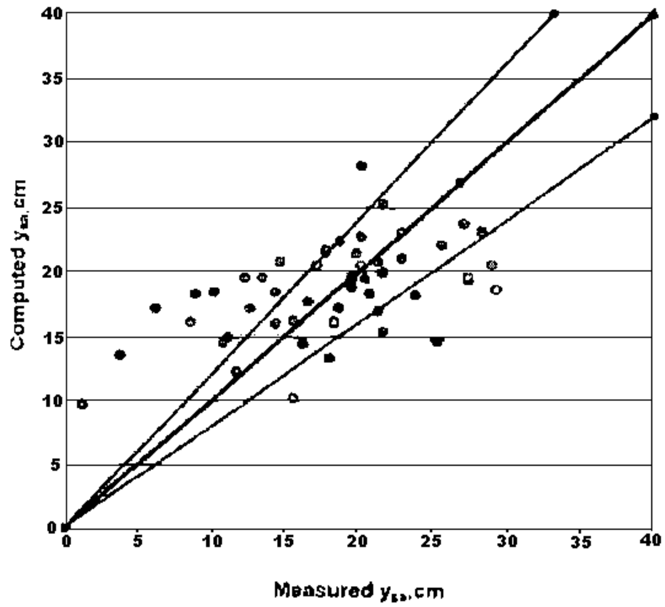

Figure F.2. Comparison of measured and predicted scour depths, Maryland SHA Equations.

F.7 REFERENCES

1. Chang, F. and S.R. Davis, 1999, "Maryland SHA Procedure for Estimating Scour at Bridge Abutments, Part I - Live Bed Scour," ASCE Compendium, Stream Stability and Scour at Highway Bridges, Richardson and Lagasse (eds.), Reston, VA.

2. Chang, F. and S.R. Davis, 1999, "The Maryland State Highway Administration ABSCOUR Program," Maryland SHA.

3. Palaviccini, M., 1993, "Scour Prediction Model at Bridge Abutments," Dissertation in partial fulfillment of the requirements of Doctor of Engineering Dissertation, The Catholic University of America, Washington, D.C.

4. Sturm, T.W., 1999, "Abutment Scour Studies for Compound Channels," U.S. Department of Transportation, Federal Highway Administration, September.

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APPENDIX G

WSPRO INPUT AND OUTPUT FOR EXAMPLE PROBLEMS IN CHAPTER 8 AND APPENDIX H

- **G1 (SI**
- **G2 (English)**

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APPENDIX G1

WSPRO Input and Output for Chapter 8 Example Problem (SI)

Line #	Input parameters

```
*f
T1        WORKSHOP PROBLEM - SCOUR CREEK - METRIC CONVERSION
T2        ESTIMATING SCOUR AT BRIDGES - COMPUTER SIMULATION
T3        CONTRACTION, PIER, AND ABUTMENT SCOUR CALCULATIONS
*
SI        1
*
Q         849.51
SK        0.002
*
XS   EXIT  228.6 * * * .002
GR        0,5.79  30.48,4.57  60.96,3.35  152.4,3.28  274.32,3.05  335.28,2.74
GR        370.33,1.68  381.00,1.49  396.24,0.93  411.48,1.48  422.15,1.55
GR        457.2,2.74  518.16,3.05  640.08,3.28  731.52,3.35  762.00,4.57
GR        792.48,5.79
N           0.042     0.032     0.042
SA               335.28    457.2
*
XS   FULLV 426.72
*
BR   BRDG  426.72
BL 1       198.12   335.28   457.2
BC         5.49
CD         3  15.24  2  6.71
AB         2
PD 0       1.72  9.14   6
N          0.042   0.032
SA            335.28
*
XS   APPR  640.08
*
HP 2 BRDG  4.23  1  4.23  849.51
HP 1 BRDG  4.15  1  4.15
HP 2 APPR  5.27  1  5.27  849.51
HP 1 APPR  5.27  1  5.27
*
EX
ER
```

OUTPUT DATA FOR CHAPTER EXAMPLE PROBLEM

Line #	Input parameters

```
 1         ***********************  W S P R O  ***************************
 2           Federal Highway Administration  -  U. S. Geological Survey
 3                  Model for Water-Surface Profile Computations.
 4             Run Date & Time: 10/26/94  1:55 pm     Version V081594
 5              Input File: scourcrm.dat   Output File: scourcrm.lst
 6        *----------------------------------------------------------------*
 7     *F
 8      ***                   Input Data In Free Format                   ***
 9
10     T1         WORKSHOP PROBLEM - SCOUR CREEK - METRIC CONVERSION
11     T2         ESTIMATING SCOUR AT BRIDGES - COMPUTER SIMULATION
12     T3         CONTRACTION, PIER, AND ABUTMENT SCOUR CALCULATIONS
13     SI         1
14                         Metric (SI) Units Used in WSPRO
15                    Quantity    SI Unit              Precision
16                  ------------ -------------------- -----------
17                   Length       meters                  0.001
18                   Depth        meters                  0.001
19                   Elevation    meters                  0.001
20                   Widths       meters                  0.001
21                   Velocity     meters/second           0.001
22                   Discharge    cubic meters/second     0.001
23                   Slope        meter/meter             0.001
24                   Angles       degrees                 0.01
25                  ------------ -------------------- -----------
26     Q          849.51
27      ***   Processing Flow Data; Placing Information into Sequence  1   ***
28     SK         0.002
29        ************************  W S P R O  ***************************
30
31              *--------------------------------------------------*
32              *      Starting To Process Header Record EXIT      *
33              *--------------------------------------------------*
34     XS   EXIT  228.6 * * * .002
35     GR         0,5.79  30.48,4.57  60.96,3.35  152.4,3.28  274.32,3.05  335.28,2.74
36     GR         370.33,1.68  381.00,1.49  396.24,0.93  411.48,1.48  422.15,1.55
37     GR         457.2,2.74  518.16,3.05  640.08,3.28  731.52,3.35  762.00,4.57
38     GR         792.48,5.79
39     N            0.042     0.032     0.042
40     SA                335.28    457.2
41
42      ***   Completed Reading Data Associated With Header Record EXIT    ***
43      ***   Storing Header Data In Temporary File As Record Number  1    ***
44
45      ***               Data Summary For Header Record EXIT              ***
46      SRD Location:      229.   Cross-Section Skew:    .0   Error Code   0
47      Valley Slope:   .00200    Averaging Conveyance By Geometric Mean.
48      Energy Loss Coefficients ->   Expansion:   .50   Contraction:   .00
49
50                          X,Y-coordinates (17 pairs)
51          X           Y            X          Y            X          Y
52     ----------  ----------   ----------  ----------   ----------  ----------
53          .000       5.790       30.480       4.570       60.960       3.350
54       152.400       3.280      274.320       3.050      335.280       2.740
55       370.330       1.680      381.000       1.490      396.240        .930
56       411.480       1.480      422.150       1.550      457.200       2.740
57       518.160       3.050      640.080       3.280      731.520       3.350
58       762.000       4.570      792.480       5.790
59     ----------  ----------   ----------  ----------   ----------  ----------
60                      Minimum and Maximum X,Y-coordinates
61       Minimum X-Station:        .000  ( associated Y-Elevation:    5.790 )
62       Maximum X-Station:     792.480  ( associated Y-Elevation:    5.790 )
63       Minimum Y-Elevation:      .930  ( associated X-Station:    396.240 )
64       Maximum Y-Elevation:     5.790  ( associated X-Station:    792.480 )
65
66   Subarea Breakpoints (NSA =   3):
67          335.    457.
68    Roughness Coefficients (NSA =  3):
```

```
          .042    .032    .042
             *---------------------------------------------------*
            *     Finished Processing Header Record EXIT       *
            *---------------------------------------------------*
      *************************  W S P R O  ***************************

            *---------------------------------------------------*
            *     Starting To Process Header Record FULLV      *
            *---------------------------------------------------*

   XS   FULLV 426.72

    ***   Completed Reading Data Associated With Header Record FULLV   ***
    ***   No Roughness Data Input, Propagating From Previous Section   ***
    ***   Storing Header Data In Temporary File As Record Number  2    ***

    ***              Data Summary For Header Record FULLV              ***

    SRD Location:      427.   Cross-Section Skew:    .0   Error Code   0
    Valley Slope:   .00200    Averaging Conveyance By Geometric Mean.
    Energy Loss Coefficients ->   Expansion:   .50   Contraction:   .00

                         X,Y-coordinates (17 pairs)
        X          Y            X          Y            X          Y
   ----------  ----------   ----------  ----------   ----------  ----------
         .000       6.186       30.480       4.966       60.960       3.746
      152.400       3.676      274.320       3.446      335.280       3.136
      370.330       2.076      381.000       1.886      396.240       1.326
      411.480       1.876      422.150       1.946      457.200       3.136
      518.160       3.446      640.080       3.676      731.520       3.746
      762.000       4.966      792.480       6.186
   ----------  ----------   ----------  ----------   ----------  ----------


                    Minimum and Maximum X,Y-coordinates
     Minimum X-Station:        .000  ( associated Y-Elevation:    6.186 )
     Maximum X-Station:     792.480  ( associated Y-Elevation:    6.186 )
     Minimum Y-Elevation:     1.326  ( associated X-Station:    396.240 )
     Maximum Y-Elevation:     6.186  ( associated X-Station:    792.480 )

 Subarea Breakpoints (NSA =   3):
         335.    457.

 Roughness Coefficients (NSA =  3):
          .042    .032    .042

            *---------------------------------------------------*
            *     Finished Processing Header Record FULLV      *
            *---------------------------------------------------*
      *************************  W S P R O  ***************************

            *---------------------------------------------------*
            *     Starting To Process Header Record BRDG       *
            *---------------------------------------------------*
   BR   BRDG  426.72
   BL  1       198.12   335.28   457.2
   BC          5.49
   CD          3  15.24  2  6.71
   AB          2
   PD  0       1.72  9.14   6
   N           0.042   0.032
   SA             335.28

    ***   Completed Reading Data Associated With Header Record BRDG    ***
    ***   Storing Header Data In Temporary File As Record Number  3    ***

    ***              Data Summary For Header Record BRDG               ***

    SRD Location:      427.   Cross-Section Skew:    .0   Error Code   0
    Valley Slope:   .00200    Averaging Conveyance By Geometric Mean.
    Energy Loss Coefficients ->   Expansion:   .50   Contraction:   .00
                          X,Y-coordinates (13 pairs)
        X          Y            X          Y            X          Y
   ----------  ----------   ----------  ----------   ----------  ----------
      263.788       5.490      267.852       3.458      274.319       3.446
```

```
       335.279       3.136      370.329       2.076      380.999       1.886
       396.239       1.326      411.479       1.875      422.149       1.946
       457.199       3.136      457.200       3.136      461.908       5.490
       263.788       5.490
    ----------  ----------   ----------  ----------   ----------  ----------
                    Minimum and Maximum X,Y-coordinates
      Minimum X-Station:     263.788  ( associated Y-Elevation:    5.490 )
      Maximum X-Station:     461.908  ( associated Y-Elevation:    5.490 )
      Minimum Y-Elevation:     1.326  ( associated X-Station:    396.239 )
      Maximum Y-Elevation:     5.490  ( associated X-Station:    263.788 )

  Subarea Breakpoints (NSA =   2):
 335
  Roughness Coefficients (NSA =  2):
          .042    .032

  Discharge coefficient parameters:
   BRTYPE   BRWDTH   EMBSS    EMBELV   USERCD
     3       15.2    2.00      6.71    ******

  Pressure flow elevations:  AVBCEL =     5.49   PFELEV =     5.49

  Abutment parameters:
    ABSLPL   ABSLPR   XTOELT   YTOELT   XTOERT   YTOERT
       2.0   ******    267.9      3.5    457.2      3.1

  Bridge Length and Bottom Chord component input data:
    BRLEN   LOCOPT   XCONLT   XCONRT   BCELEV    BCSLP   BCXSTA
    198.1       1.     335.     457.     5.49   ******  *******

  Pier Data:  Number  1 Pier/Pile Code: 0.
        ELEV  WDTH  #P/P       ELEV  WDTH  #P/P       ELEV  WDTH  #P/P
        1.72    9.1   6.00

             *---------------------------------------------------*
             *      Finished Processing Header Record BRDG       *
             *---------------------------------------------------*
       *************************  W S P R O  **************************

             *---------------------------------------------------*
             *      Starting To Process Header Record APPR       *
             *---------------------------------------------------*

    XS   APPR  640.08

     ***   Completed Reading Data Associated With Header Record APPR    ***
     ***   No Roughness Data Input, Propagating From Previous Section   ***
     ***   Storing Header Data In Temporary File As Record Number  4    ***

     ***              Data Summary For Header Record APPR               ***

     SRD Location:      640.   Cross-Section Skew:    .0   Error Code   0
     Valley Slope:   .00200    Averaging Conveyance By Geometric Mean.
     Energy Loss Coefficients ->   Expansion:   .50   Contraction:   .00

                       X,Y-coordinates (17 pairs)
         X          Y            X          Y            X          Y
    ----------  ----------   ----------  ----------   ----------  ----------
          .000       6.613       30.479       5.393       60.959       4.173
       152.399       4.103      274.319       3.873      335.279       3.563
       370.329       2.503      380.999       2.313      396.239       1.753
       411.479       2.302      422.149       2.373      457.199       3.563
       518.159       3.873      640.079       4.103      731.519       4.173
       761.999       5.393      792.479       6.613
    ----------  ----------   ----------  ----------   ----------  ----------
                     Minimum and Maximum X,Y-coordinates
      Minimum X-Station:        .000  ( associated Y-Elevation:    6.613 )
      Maximum X-Station:     792.479  ( associated Y-Elevation:    6.613 )
      Minimum Y-Elevation:     1.753  ( associated X-Station:    396.239 )
      Maximum Y-Elevation:     6.613  ( associated X-Station:    792.479 )

  Subarea Breakpoints (NSA =   3):
         335.    457.
```

```
Roughness Coefficients (NSA =  3):
         .042    .032    .042

Bridge datum projection(s):  XREFLT  XREFRT  FDSTLT  FDSTRT
                            ******* ******* ******* *******

          *----------------------------------------------------*
          *      Finished Processing Header Record APPR        *
          *----------------------------------------------------*
     ************************  W S P R O  **************************

  HP  2 BRDG  4.23  1  4.23  849.51
  HP  1 BRDG  4.15  1  4.15
  HP  2 APPR  5.27  1  5.27  849.51
  HP  1 APPR  5.27  1  5.27
  EX

         *===================================================*
         *     Summary of Boundary Condition Information     *
         *===================================================*

           Reach      Water Surface   Friction
      #   Discharge      Elevation     Slope        Flow Regime
     --   ---------    -------------   --------   --------------------
      1      849.51      ********        .0020        Sub-Critical
     --   ---------    -------------   --------   --------------------

         *===================================================*
         *        Beginning  1 Profile Calculation(s)        *
         *===================================================*

     ************************  W S P R O  **************************

                    WSEL    VHD       Q         AREA      SRDL      LEW
                    EGEL     HF       V          K        FLEN      REW
                    CRWS     HO      FR #        SF       ALPHA     ERR
                 --------- ------ ---------- ---------- --------- ---------
 Section: EXIT       3.832   .173    849.509    622.871      .000    48.894
 Header Type: XS     4.006   .000      1.364   18992.99      .000   743.584
 SRD:    228.600     3.615   .000       .622      .0000     1.830      .000

 Section: FULLV      4.231   .172    849.509    624.430   198.119    48.837
 Header Type: FV     4.404   .395      1.360   19053.13   198.119   743.642
 SRD:    426.719     4.011   .000       .620      .0020     1.828      .002

        <<< The Preceding Data Reflect The "Unconstricted" Profile >>>

 Section: APPR       4.658   .172    849.509    624.574   213.360    48.829
 Header Type: AS     4.830   .424      1.360   19059.62   213.360   743.648
 SRD:    640.080     4.438   .000       .620      .0020     1.828      .002

        <<< The Preceding Data Reflect The "Unconstricted" Profile >>>

         <<< The Following Data Reflect The "Constricted" Profile >>>
               <<< Beginning Bridge Hydraulics Computations >>>

 Section: BRDG       4.151   .769    849.509    294.018   198.119   266.464
 Header Type: BR     4.921   .620      2.889   12559.79   198.119   459.231
 SRD:    426.719     3.990   .293      1.004      .0020     1.806      .000

 Specific Bridge Information   C     P/A   PFELEV    BLEN     XLAB     XRAB
 Bridge Type 3   Flow Type 1 ------ ----- -------- -------- -------- --------
 Pier/Pile Code  0           .7441  .034    5.489  198.120  267.851  457.197
 -------------------------- ------ ----- -------- -------- -------- --------

 Section: APPR       5.268   .050    849.509   1058.158   198.120    33.581
 Header Type: AS     5.318   .323       .802   39088.53   213.359   758.896
 SRD:    640.080     4.438   .074       .263      .0020     1.534     -.003
               Approach Section APPR  Flow Contraction Information
             M( G )   M( K )     KQ      XLKQ     XRKQ     OTEL
            -------- -------- --------- -------- -------- --------
                .722     .426   22535.5  271.518  463.594    5.175
            -------- -------- --------- -------- -------- --------
```

```
                  <<< End of Bridge Hydraulics Computations >>>

     *************************  W S P R O  ***************************

   ***    Beginning Velocity Distribution For Header Record BRDG     ***
          SRD Location:    426.720       Header Record Number  3

       Water Surface Elevation:      4.230              Element # 1
       Flow:     849.510   Velocity:  2.75  Hydraulic Depth:   1.600
       Cross-Section Area:    309.17         Conveyance:    13531.24
          Bank Stations ->  Left:    266.307   Right:    459.388

X STA.       266.3      305.8      332.9      348.6      358.2      366.0
  A( I )           32.8       27.5       19.8       15.8       14.9
  V( I )           1.29       1.54       2.15       2.68       2.86
  D( I )            .83       1.01       1.26       1.64       1.91

X STA.       366.0      372.2      378.1      383.5      388.3      392.6
  A( I )           13.2       13.2       12.5       12.1       11.8
  V( I )           3.22       3.21       3.39       3.50       3.61
  D( I )           2.11       2.24       2.35       2.52       2.69

X STA.       392.6      396.8      400.8      405.2      410.1      415.4
  A( I )           11.6       11.4       11.7       12.2       12.6
  V( I )           3.65       3.73       3.62       3.47       3.38
  D( I )           2.82       2.84       2.66       2.49       2.35

X STA.       415.4      421.0      427.2      434.3      443.5      459.4
  A( I )           12.8       13.8       14.3       15.7       19.4
  V( I )           3.32       3.09       2.97       2.71       2.19
  D( I )           2.31       2.22       1.99       1.71       1.22


     *************************  W S P R O  ***************************

   ***    Compute Cross-Section Properties For Header Record BRDG    ***
           SRD Location:    426.720       Header Record Number  3

Water     S  Cross      Cross                  Bank Station
Surface   A  Section    Section  Top   Wetted  ------------  Hydrlic  Critical
Elevation #  Conveyance Area(s) Width   Pmtr    Left  Right   Depth     Flow
--------- -- ---------- ------- ------ ------- ------ ------ ------- ---------
          1    1208.24     57.   68.8   68.98                  .834    164.12
          2   11333.03    236.  123.9  124.25                 1.906   1021.92
    4.150     12541.26    294.  192.8  193.22  266.5  459.2   1.523   1052.71
--------- -- ---------- ------- ------ ------- ------ ------ ------- ---------
     *************************  W S P R O  ***************************

   ***    Beginning Velocity Distribution For Header Record APPR     ***
          SRD Location:    640.080       Header Record Number  4

       Water Surface Elevation:      5.270              Element # 1
       Flow:     849.510   Velocity:   .80  Hydraulic Depth:   1.460
       Cross-Section Area:   1059.34         Conveyance:    39151.16
          Bank Stations ->  Left:     33.541   Right:    758.937

X STA.        33.5      124.4      186.1      242.1      290.5      330.4
  A( I )           86.2       72.9       71.9       67.3       63.1
  V( I )            .49        .58        .59        .63        .67
  D( I )            .95       1.18       1.28       1.39       1.58

X STA.       330.4      352.8      366.9      378.4      388.4      396.9
  A( I )           42.7       34.7       32.2       30.5       28.9
   V( I )            .99       1.22       1.32       1.39       1.47
  D( I )           1.91       2.45       2.80       3.05       3.38

X STA.       396.9      405.5      415.4      426.5      440.0      462.5
  A( I )           28.5       30.2       32.0       33.9       43.5
  V( I )           1.49       1.41       1.33       1.25        .98
  D( I )           3.34       3.03       2.88       2.52       1.93

X STA.       462.5      501.9      549.6      604.8      668.4      758.9
  A( I )           62.2       66.4       71.0       75.2       85.8
  V( I )            .68        .64        .60        .57        .49
```

D(I)   1.58   1.39   1.29   1.18   .95

************************* W S P R O ***************************

*** Compute Cross-Section Properties For Header Record APPR ***

SRD Location: 640.080   Header Record Number 4

Water Surface Elevation	S A #	Cross Section Conveyance	Cross Section Area(s)	Top Width	Wetted Pmtr	Bank Station Left	Bank Station Right	Hydrlic Depth	Critical Flow
	1	10075.62	370.	301.7	301.76			1.225	1281.66
	2	18999.92	320.	121.9	121.98			2.622	1622.42
	3	10075.62	370.	301.7	301.76			1.225	1281.66
5.270		39151.16	1059.	725.4	725.50	33.5	758.9	1.460	3237.29

ER

****************** Normal end of WSPRO execution. *****************

*************** Elasped Time: 0 Minutes 0 Seconds **************

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APPENDIX G2

WSPRO Input and Output for Appendix H Example Problem (English)

INPUT DATA FOR CHAPTER 4 EXAMPLE PROBLEM

```
T1       SCOUR EXAMPLE #2 - HYPOTHETICAL EXAMPLE
T2       CONTRACTION, PIER, AND ABUTMENT SCOUR CALCULATIONS
T3       HEC-18 - EVALUATING SCOUR AT BRIDGES
*
Q        30000
SK        0.002
*
XS   EXIT  750 * * * .002
GR       0,19   100,15   200,11   500,10.75   900,10   1100,9.0   1215,5.5
GR        1250,4.9   1300,3.05   1350,4.85   1385,5.1   1500,9.0   1700,10
GR        2100,10.75   2400,11   2500,15   2600,19
N          0.042    0.032    0.042
SA              1100      1500
*
XS   FULLV 1400
*
BR   BRDG  1400
BL 1       650   1100   1500
BD         4  22
CD         3  50  2  22
AB         2
PW         5.65  30
N         0.042   0.032
SA            1100
*
AS   APPR  2100
*
HP 2 BRDG  13.82 * * 30000
*
HP 1 BRDG  13.54 1 13.54
*
HP 2 APPR  17.36 * * 30000
*
HP 1 APPR  17.36 1 17.36
*
EX
ER
```

OUTPUT

```
1
WSPRO        FEDERAL HIGHWAY ADMINISTRATION - U. S. GEOLOGICAL SURVEY
P060188          MODEL  FOR  WATER-SURFACE  PROFILE  COMPUTATIONS

         *** RUN DATE & TIME: 09-10-92  10:08

 T1       SCOUR EXAMPLE #2 - HYPOTHETICAL EXAMPLE
 T2       CONTRACTION, PIER, AND ABUTMENT SCOUR CALCULATIONS
 T3       HEC-18 - EVALUATING SCOUR AT BRIDGES
 *
 Q        30000
*** Q-DATA FOR SEC-ID, ISEQ =           1
 SK       0.002
 *
1
WSPRO        FEDERAL HIGHWAY ADMINISTRATION - U. S. GEOLOGICAL SURVEY
P060188          MODEL  FOR  WATER-SURFACE  PROFILE  COMPUTATIONS

       SCOUR EXAMPLE #2 - HYPOTHETICAL EXAMPLE
       CONTRACTION, PIER, AND ABUTMENT SCOUR CALCULATIONS
       HEC-18 - EVALUATING SCOUR AT BRIDGES
         *** RUN DATE & TIME: 09-10-92  10:08

*** START PROCESSING CROSS SECTION - "EXIT "
 XS   EXIT  750 * * * .002
 GR       0,19   100,15   200,11   500,10.75   900,10   1100,9.0   1215,5.5
 GR       1250,4.9   1300,3.05   1350,4.85   1385,5.1   1500,9.0   1700,10
 GR       2100,10.75   2400,11   2500,15   2600,19
 N          0.042    0.032    0.042
 SA             1100      1500
 *

*** FINISH PROCESSING CROSS SECTION - "EXIT "
*** CROSS SECTION "EXIT " WRITTEN TO DISK, RECORD NO. =  1

--- DATA SUMMARY FOR SECID "EXIT " AT SRD =    750.  ERR-CODE =      0

    SKEW    IHFNO    VSLOPE       EK       CK
      .0       0.    .0020      .50      .00

X-Y COORDINATE PAIRS  (NGP =  17):
      X     Y        X     Y        X     Y        X     Y
      .0   19.00     100.0   15.00     200.0   11.00     500.0   10.75
   900.0   10.00    1100.0    9.00    1215.0    5.50    1250.0    4.90
  1300.0    3.05    1350.0    4.85    1385.0    5.10    1500.0    9.00
  1700.0   10.00    2100.0   10.75    2400.0   11.00    2500.0   15.00
  2600.0   19.00

 X-Y MAX-MIN POINTS:
    XMIN      Y        X    YMIN     XMAX      Y        X    YMAX
      .0   19.00    1300.0    3.05    2600.0   19.00       .0   19.00

SUBAREA BREAKPOINTS (NSA =  3):
      1100.   1500.

ROUGHNESS COEFFICIENTS (NSA =  3):
        .042    .032    .042
1
WSPRO        FEDERAL HIGHWAY ADMINISTRATION - U. S. GEOLOGICAL SURVEY
P060188          MODEL  FOR  WATER-SURFACE  PROFILE  COMPUTATIONS

       SCOUR EXAMPLE #2 - HYPOTHETICAL EXAMPLE
       CONTRACTION, PIER, AND ABUTMENT SCOUR CALCULATIONS
       HEC-18 - EVALUATING SCOUR AT BRIDGES
         *** RUN DATE & TIME: 09-10-92  10:08

*** START PROCESSING CROSS SECTION - "FULLV"
 XS   FULLV 1400
```

```
 *

*** FINISH PROCESSING CROSS SECTION - "FULLV"
*** NO ROUGHNESS DATA INPUT, WILL PROPAGATE FROM PREVIOUS CROSS SECTION.
*** CROSS SECTION "FULLV" WRITTEN TO DISK, RECORD NO. =  2

--- DATA SUMMARY FOR SECID "FULLV" AT SRD =   1400.  ERR-CODE =     0

    SKEW    IHFNO    VSLOPE       EK       CK
      .0       0.    .0020       .50      .00

X-Y COORDINATE PAIRS  (NGP =  17):
      X      Y          X      Y          X      Y          X      Y
      .0   20.30    100.0   16.30    200.0   12.30    500.0   12.05
   900.0   11.30   1100.0   10.30   1215.0    6.80   1250.0    6.20
  1300.0    4.35   1350.0    6.15   1385.0    6.40   1500.0   10.30
  1700.0   11.30   2100.0   12.05   2400.0   12.30   2500.0   16.30
  2600.0   20.30

 X-Y MAX-MIN POINTS:
    XMIN      Y         X    YMIN      XMAX      Y         X    YMAX
      .0   20.30    1300.0    4.35    2600.0   20.30       .0   20.30

SUBAREA BREAKPOINTS (NSA =  3):
      1100.   1500.

ROUGHNESS COEFFICIENTS (NSA =  3):
        .042    .032    .042
1
WSPRO        FEDERAL HIGHWAY ADMINISTRATION - U. S. GEOLOGICAL SURVEY
P060188          MODEL  FOR  WATER-SURFACE  PROFILE  COMPUTATIONS

        SCOUR EXAMPLE #2 - HYPOTHETICAL EXAMPLE
        CONTRACTION, PIER, AND ABUTMENT SCOUR CALCULATIONS
        HEC-18 - EVALUATING SCOUR AT BRIDGES
           *** RUN DATE & TIME: 09-10-92  10:08

*** START PROCESSING CROSS SECTION - "BRDG "
  BR   BRDG  1400
  BL 1      650   1100   1500
  BD        4  22
  CD        3  50  2  22
  AB        2
  PW        5.65  30
  N       0.042   0.032
  SA          1100
  *

*** FINISH PROCESSING CROSS SECTION - "BRDG "
*** CROSS SECTION "BRDG " WRITTEN TO DISK, RECORD NO. =  3

--- DATA SUMMARY FOR SECID "BRDG " AT SRD =   1400.  ERR-CODE =     0

    SKEW    IHFNO    VSLOPE       EK       CK
      .0       0.    .0020       .50      .00

X-Y COORDINATE PAIRS  (NGP =  13):
      X      Y          X      Y          X      Y          X      Y
   865.4   18.00    878.7   11.34    900.0   11.30   1100.0   10.30
  1215.0    6.80   1250.0    6.20   1300.0    4.35   1350.0    6.15
  1385.0    6.40   1500.0   10.30   1500.0   10.30   1515.4   18.00
```

```
   865.4   18.00

 X-Y MAX-MIN POINTS:
    XMIN      Y        X    YMIN     XMAX      Y        X    YMAX
   865.4   18.00    1300.0   4.35    1515.4   18.00     865.4   18.00

SUBAREA BREAKPOINTS (NSA =  2):
      1100.

ROUGHNESS COEFFICIENTS (NSA =  2):
        .042    .032

BRIDGE PARAMETERS:
 BRTYPE  BRWDTH    LSEL  USERCD  EMBSS   EMBELV  ABSLPL  ABSLPR
   3      50.0    18.00 *******   2.00    22.00    2.00 *******

DESIGN DATA:    BRLEN  LOCOPT  XCONLT  XCONRT
              650.0      1.   1100.   1500.

             GIRDEP  BDELEV   BDSLP   BDSTA
               4.00   22.00 ******* *******

PIER DATA:  NPW =  1    PPCD = 0.
    PELV  PWDTH     PELV  PWDTH     PELV  PWDTH     PELV  PWDTH
     5.65  30.0
1
WSPRO        FEDERAL HIGHWAY ADMINISTRATION - U. S. GEOLOGICAL SURVEY
P060188          MODEL  FOR  WATER-SURFACE  PROFILE  COMPUTATIONS

       SCOUR EXAMPLE #2 - HYPOTHETICAL EXAMPLE
       CONTRACTION, PIER, AND ABUTMENT SCOUR CALCULATIONS
       HEC-18 - EVALUATING SCOUR AT BRIDGES
          *** RUN DATE & TIME: 09-10-92  10:08

*** START PROCESSING CROSS SECTION - "APPR "
  AS   APPR  2100
  *
  HP 2 BRDG  13.82 * * 30000

*** FINISH PROCESSING CROSS SECTION - "APPR "
*** NO ROUGHNESS DATA INPUT, WILL PROPAGATE FROM PREVIOUS CROSS SECTION.
*** CROSS SECTION "APPR " WRITTEN TO DISK, RECORD NO. =  4

--- DATA SUMMARY FOR SECID "APPR " AT SRD =   2100.  ERR-CODE =      0

    SKEW     IHFNO    VSLOPE       EK       CK
      .0       0.     .0020      .50      .00

X-Y COORDINATE PAIRS  (NGP =  17):
      X      Y         X      Y         X      Y         X      Y
      .0   21.70     100.0   17.70     200.0   13.70     500.0   13.45
   900.0   12.70    1100.0   11.70    1215.0    8.20    1250.0    7.60
  1300.0    5.75    1350.0    7.55    1385.0    7.80    1500.0   11.70
  1700.0   12.70    2100.0   13.45    2400.0   13.70    2500.0   17.70
  2600.0   21.70

 X-Y MAX-MIN POINTS:
    XMIN      Y        X    YMIN     XMAX      Y        X    YMAX
      .0   21.70    1300.0   5.75    2600.0   21.70       .0   21.70

SUBAREA BREAKPOINTS (NSA =  3):
```

```
      1100.   1500.

ROUGHNESS COEFFICIENTS (NSA =  3):
       .042    .032    .042

BRIDGE PROJECTION DATA:  XREFLT  XREFRT  FDSTLT  FDSTRT
                   ******* ******* ******* *******
1
WSPRO        FEDERAL HIGHWAY ADMINISTRATION - U. S. GEOLOGICAL SURVEY
P060188          MODEL  FOR  WATER-SURFACE  PROFILE  COMPUTATIONS

      SCOUR EXAMPLE #2 - HYPOTHETICAL EXAMPLE
      CONTRACTION, PIER, AND ABUTMENT SCOUR CALCULATIONS
      HEC-18 - EVALUATING SCOUR AT BRIDGES
        *** RUN DATE & TIME: 09-10-92  10:08



   VELOCITY DISTRIBUTION:  ISEQ =  3;  SECID = BRDG ;  SRD =    1400.

       WSEL     LEW     REW    AREA       K       Q    VEL
      13.82   873.8  1507.0  3286.9  470494.   30000.   9.13

X STA.      873.8     1003.3     1096.9     1150.0     1180.3     1203.9
  A(I)          346.5      305.9      225.0      166.6      149.6
  V(I)           4.33       4.90       6.67       9.00      10.03

X STA.     1203.9     1223.7     1241.9     1259.0     1274.4     1288.4
  A(I)          137.8      133.3      131.0      126.9      123.1
  V(I)          10.89      11.26      11.45      11.82      12.18

X STA.     1288.4     1301.6     1314.7     1329.0     1344.3     1361.3
  A(I)          122.0      120.7      123.8      124.5      131.2
  V(I)          12.29      12.43      12.11      12.05      11.43

X STA.     1361.3     1379.0     1397.3     1418.7     1447.3     1507.0
  A(I)          133.2      133.3      141.9      165.3      245.2
  V(I)          11.26      11.25      10.57       9.07       6.12
1
  *
  HP 1 BRDG  13.54 1 13.54
1
WSPRO        FEDERAL HIGHWAY ADMINISTRATION - U. S. GEOLOGICAL SURVEY
P060188          MODEL  FOR  WATER-SURFACE  PROFILE  COMPUTATIONS

      SCOUR EXAMPLE #2 - HYPOTHETICAL EXAMPLE
      CONTRACTION, PIER, AND ABUTMENT SCOUR CALCULATIONS
      HEC-18 - EVALUATING SCOUR AT BRIDGES
        *** RUN DATE & TIME: 09-10-92  10:08
   CROSS-SECTION PROPERTIES:  ISEQ =  3;  SECID = BRDG ;  SRD =    1400.

    WSEL  SA#    AREA       K   TOPW   WETP  ALPH    LEW     REW     QCR
           1     600.   40797.   226.   226.                  5553.
           2    2510.  392654.   406.   407.                  35385.
   13.54        3110.  433451.   632.   634.  1.16   874.  1506.  36279.
1
  *
  HP 2 APPR  17.36 * * 30000
1
WSPRO        FEDERAL HIGHWAY ADMINISTRATION - U. S. GEOLOGICAL SURVEY
P060188          MODEL  FOR  WATER-SURFACE  PROFILE  COMPUTATIONS
```

```
      SCOUR EXAMPLE #2 - HYPOTHETICAL EXAMPLE
      CONTRACTION, PIER, AND ABUTMENT SCOUR CALCULATIONS
      HEC-18 - EVALUATING SCOUR AT BRIDGES
        *** RUN DATE & TIME: 09-10-92  10:08



   VELOCITY DISTRIBUTION:  ISEQ =  4;  SECID = APPR ;  SRD =    2100.

       WSEL     LEW     REW    AREA       K       Q    VEL
      17.36   108.5  2491.5 11565.0 1414915.   30000.   2.59

 X STA.      108.5      416.1      623.7      798.5      951.8     1077.6
   A(I)         978.0      823.0      752.7      711.6      658.1
   V(I)          1.53       1.82       1.99       2.11       2.28

 X STA.     1077.6     1158.1     1204.1     1241.5     1274.0     1301.7
   A(I)         506.1      373.9      346.5      327.0      309.8
   V(I)          2.96       4.01       4.33       4.59       4.84

 X STA.     1301.7     1330.6     1363.3     1399.1     1443.3     1522.7
   A(I)         318.4      327.1      340.0      368.6      502.7
   V(I)          4.71       4.59       4.41       4.07       2.98

 X STA.     1522.7     1646.7     1803.5     1977.8     2184.8     2491.5
   A(I)         649.2      727.8      749.9      820.2      974.5
   V(I)          2.31       2.06       2.00       1.83       1.54
1
  *
  HP 1 APPR  17.36 1 17.36
1
 WSPRO        FEDERAL HIGHWAY ADMINISTRATION - U. S. GEOLOGICAL SURVEY
 P060188          MODEL  FOR  WATER-SURFACE  PROFILE  COMPUTATIONS

      SCOUR EXAMPLE #2 - HYPOTHETICAL EXAMPLE
      CONTRACTION, PIER, AND ABUTMENT SCOUR CALCULATIONS
      HEC-18 - EVALUATING SCOUR AT BRIDGES
        *** RUN DATE & TIME: 09-10-92  10:08
   CROSS-SECTION PROPERTIES:  ISEQ =  4;  SECID = APPR ;  SRD =    2100.

     WSEL  SA#    AREA       K   TOPW   WETP  ALPH    LEW    REW     QCR
            1    4049.  366963.   992.   992.                   46430.
            2    3467.  680989.   400.   400.                   57923.
            3    4049.  366963.   992.   992.                   46430.
    17.36        11565. 1414915.  2383.  2383.  1.53   108.  2492. 117067.
1
  *
  EX

 +++ BEGINNING PROFILE CALCULATIONS --   1
1
 WSPRO        FEDERAL HIGHWAY ADMINISTRATION - U. S. GEOLOGICAL SURVEY
 P060188          MODEL  FOR  WATER-SURFACE  PROFILE  COMPUTATIONS

      SCOUR EXAMPLE #2 - HYPOTHETICAL EXAMPLE
      CONTRACTION, PIER, AND ABUTMENT SCOUR CALCULATIONS
      HEC-18 - EVALUATING SCOUR AT BRIDGES
        *** RUN DATE & TIME: 09-10-92  10:08

  XSID:CODE   SRDL    LEW     AREA   VHD    HF    EGL    CRWS      Q    WSEL
```

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     SRD   FLEN    REW       K  ALPH    HO     ERR     FR#     VEL
 EXIT :XS   ******    161.    6692.   .57 *****   13.14   11.86  30000.   12.57
      750. ******  2439.  670723.  1.83 ***** *******     .62    4.48
 FULLV:FV     650.   161.    6706.   .57  1.30   14.44 *******  30000.   13.88
     1400.   650.  2439.  672489.  1.83   .00     .01     .62    4.47
        <<<<<THE ABOVE RESULTS REFLECT "NORMAL" (UNCONSTRICTED) FLOW>>>>>
 APPR :AS     700.   161.    6700.   .57  1.39   15.84 *******  30000.   15.27
     2100.   700.  2439.  671817.  1.83   .00     .00     .62    4.48
        <<<<<THE ABOVE RESULTS REFLECT "NORMAL" (UNCONSTRICTED) FLOW>>>>>
          <<<<<RESULTS REFLECTING THE CONSTRICTED FLOW FOLLOW>>>>>
  XSID:CODE   SRDL    LEW     AREA   VHD    HF     EGL    CRWS      Q    WSEL
     SRD   FLEN    REW       K  ALPH    HO     ERR     FR#     VEL
 BRDG :BR     650.   874.    3107.  2.69  2.01   16.23   13.27  30000.   13.54
     1400.   650.  1506.  432822.  1.86  1.07     .00    1.05    9.66
     TYPE PPCD FLOW      C    P/A    LSEL   BLEN   XLAB   XRAB
       3.   0.   1.   .734   .076   18.00   650.   879.  1500.
  XSID:CODE   SRDL    LEW     AREA   VHD    HF     EGL    CRWS      Q    WSEL
     SRD   FLEN    REW       K  ALPH    HO     ERR     FR#     VEL
 APPR :AS     650.   108.   11574.   .16  1.02   17.52   14.56  30000.   17.36
     2100.   697.  2492. 1416461.  1.52   .28    -.02     .26    2.59
       M(G)   M(K)      KQ   XLKQ   XRKQ    OTEL
       .722   .430  811434.   891.  1521.    17.08
               <<<<<END OF BRIDGE COMPUTATIONS>>>>>
  ER
1  NORMAL  END  OF  WSPRO  EXECUTION.
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APPENDIX H

COMPREHENSIVE EXAMPLE SCOUR PROBLEM (ENGLISH UNITS)

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APPENDIX H

Comprehensive Example Scour Problem (English Units)

H.1 GENERAL DESCRIPTION OF PROBLEM

This example problem is taken from a paper by Arneson et al.[77] FHWA's WSPRO computer program was used to obtain the hydraulic variables. The program uses 20 stream tubes to give a quasi 2-dimensional analysis. Each stream tube has the same discharge (1/20 of the total discharge). The stream tubes provide the velocity distribution across the flow and the program has excellent bridge routines. The problem presented here is an English version of the comprehensive scour problem in Chapter 8, which is worked in metric (SI) units. The solution follows Steps 1-7 of the specific design approach of Chapter 2 (Section 2.4).

A 650-foot long bridge (Figure H.1) is to be constructed over a channel with spill-through abutments (slope of 1V:2H). The left abutment is set approximately 200 ft back from the channel bank. The right abutment is set at the channel bank. The bridge deck is set at elevation 22 ft and has a girder depth of 4 ft. Six round-nose piers are evenly spaced in the bridge opening. The piers are 5 ft thick, 40 ft long, and are aligned with the flow. The 100-year design discharge is 30,000 cfs. The 500-year flow of 51,000 cfs was estimated by multiplying the Q_{100} by 1.7 since no hydrologic records were available to predict the 500-year flow.

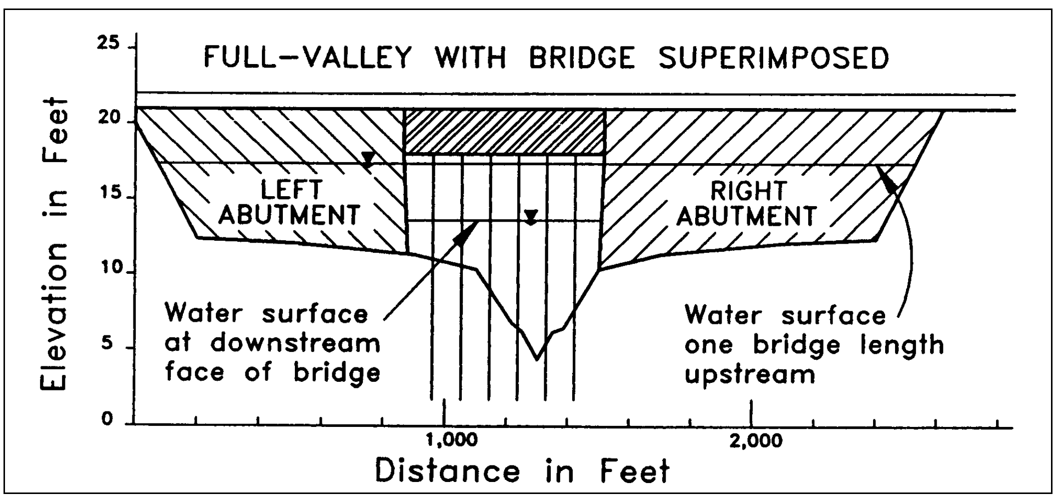

Figure H.1. Cross section of proposed bridge.

H.2 STEP 1: DETERMINE SCOUR ANALYSIS VARIABLES

From Level 1 and Level 2 analysis: a site investigation of the crossing was conducted to identify potential stream stability problems at this crossing. Evaluation of the site indicates that the river has a relatively wide floodplain. The floodplain is well vegetated with grass and trees; however, the presence of remnant channels indicates that there is a potential for lateral shifting of the channel.

The bridge crossing is located on a relatively straight reach of channel. The channel geometry is relatively the same for approximately 1,000 ft up- and downstream of the bridge crossing. The D_{50} of the bed material and overbank material is approximately 2 mm. The maximum grain size of the bed material is approximately 8 mm. The specific gravity of the bed material was determined to be equal to 2.65.

The river and crossing are located in a rural area with the primary land use consisting of agriculture and forest.

Review of bridge inspection reports for bridges located upstream and downstream of the proposed crossing indicates no long-term aggradation or degradation in this reach. At the bridge site, bedrock is approximately 150 ft below the channel bed.

Since this is a sand-bed channel, no armoring potential is expected. Furthermore, the bed for this channel at low flow consists of dunes which are approximately 1 to 1.5 ft high. At higher flows, above the Q_5, the bed will be either plane bed or antidunes.

The left and right banks are relatively well vegetated and stable; however, there are isolated portions of the bank which appear to have been undercut and are eroding. Brush and trees grow to the edge of the banks. Banks will require riprap protection if disturbed. Riprap will be required upstream of the bridge and extend downstream of the bridge.

H.2.1 Hydraulic Characteristics

Hydraulic characteristics at the bridge were determined using WSPRO.[15] Three cross sections were used for this analysis and are denoted as "EXIT" for the section downstream of the bridge, "FULLV" for the full-valley section at the bridge, and "APPR" for the approach section located one bridge length upstream of the bridge. The bridge geometry was superimposed on the full-valley section and is denoted "BRDG." Values used for this example problem are based on the output from the WSPRO model which is presented in Appendix G. Specific values for scour analysis variables are given for each computation separately and cross referenced to the line numbers of the WSPRO output.

The HP2 option was used to provide hydraulic characteristics at both the bridge and approach sections. This WSPRO option subdivides the cross section into 20 equal conveyance tubes. Figures H.2 and H.3 illustrate the location of these conveyance tubes for the approach and bridge cross section, respectively. Figure H.4 illustrates the average velocities in each conveyance tube and the contraction of the flow from the approach section through the bridge. Figure H.4 also identifies the equal conveyance tubes of the approach section which are cut off by the abutments.

Hydraulic variables for performing the various scour computations were determined from the WSPRO output (Appendix G) and from Figures H.2, H.3, and H.4. These variables, which will be used to compute contraction scour and local scour, are presented in Tables H.1 through H.6.

Contraction scour will occur both in the main channel and on the left overbank of the bridge opening. For the main channel, contraction scour could be either clear-water or live-bed depending on the magnitude of the channel velocity and the critical velocity for sediment movement. A computation will be performed to determine the sediment transport characteristics of the main channel and the appropriate contraction scour equation.

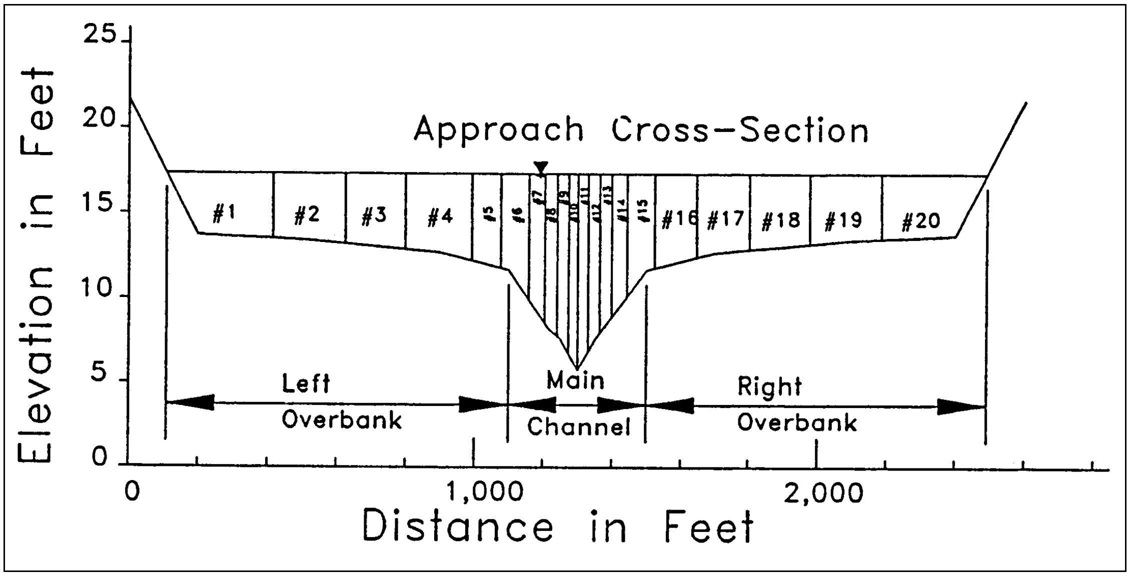

Figure H.2. Equal conveyance tubes of approach section.

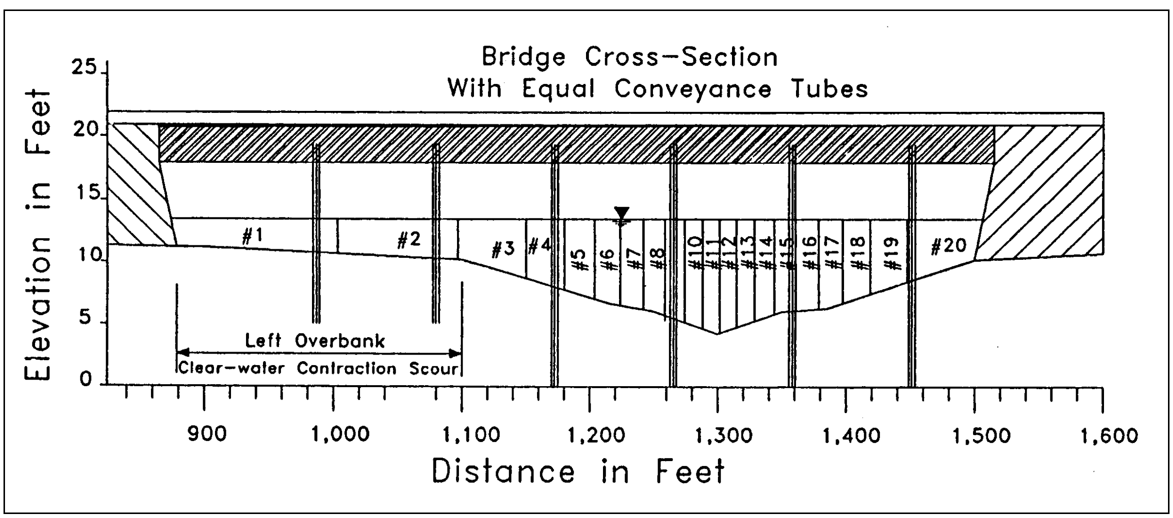

Figure H.3. Equal conveyance tubes of bridge section.

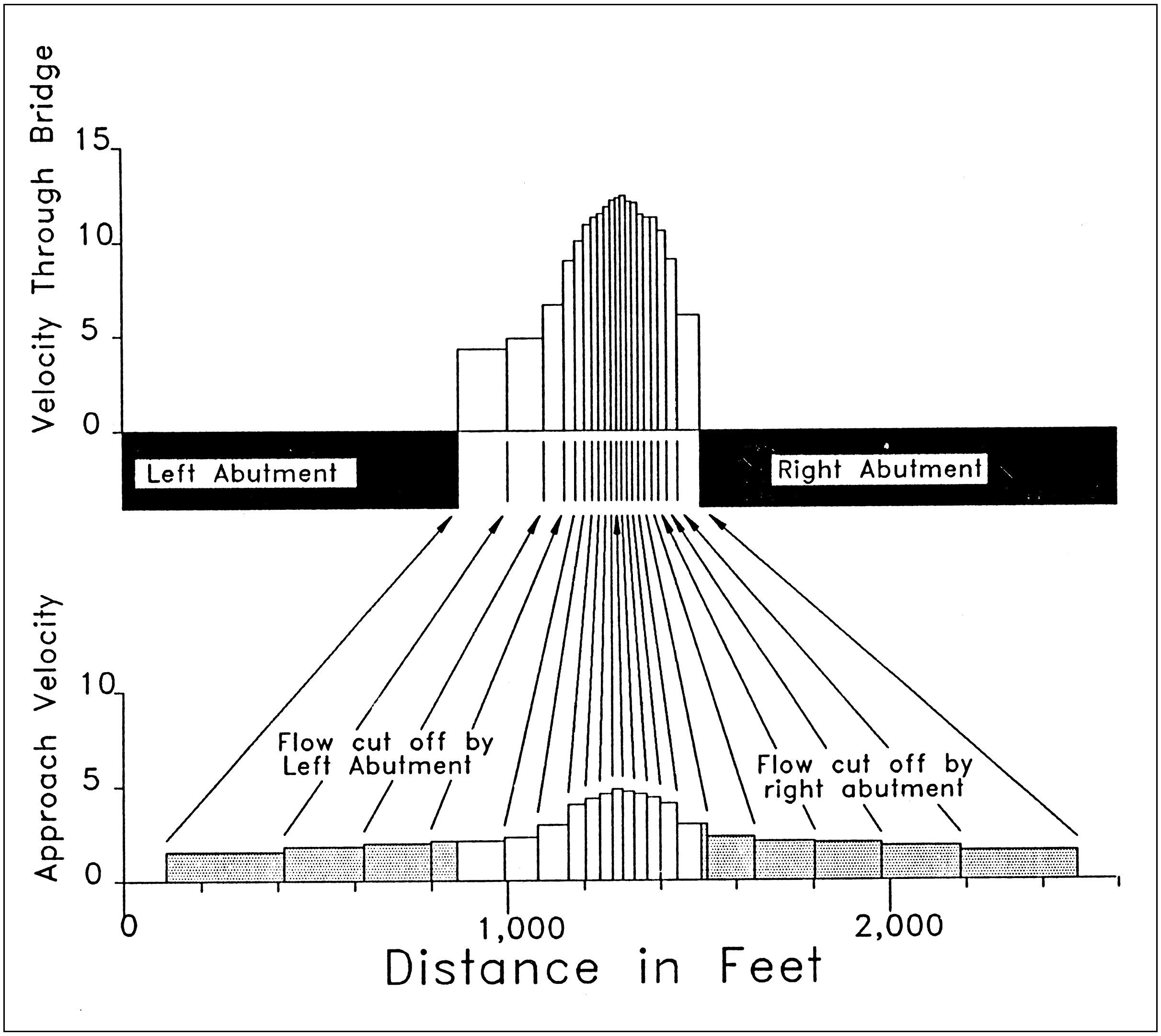

Figure H.4. Plan view of equal conveyance tubes showing velocity distribution at approach and bridge sections.

Table H.1. Hydraulic Variables from WSPRO for Estimation of Live-bed Contraction Scour.		
		Remarks
Q (cfs)	30,000	Total discharge, line 5 of WSPRO input or Line 11 of WSPRO output.
K_1 (Approach)	680,989	Conveyance of main channel of approach. Line 295 of WSPRO output, SA#2.
K_{total} (Approach)	1,414,915	Total conveyance of approach section. Line 297 of WSPRO output.
W_1 or TOPW (Approach) (ft)	400	Topwidth of flow (TOPW). Assumed to represent active live bed width of approach. Line 295 of WSPRO output, SA#2.
A_c (Approach) (ft^2)	3,467	Area of main channel approach section. Line 295, SA#2.
WETP (Approach) (ft)	400	Wetted perimeter of main channel approach section. Line 295 of WSPRO output, SA#2.
K_c (Bridge)	392,654	Conveyance of main channel through bridge. Line 244 of WSPRO output, SA#2.
K_{total} (Bridge)	433,451	Total conveyance through bridge. Line 245 of WSPRO output.
A_c (Bridge) (ft^2)	2,510	Area of the main channel, bridge section. Line 244 of WSPRO output, SA #2.
W_c (Bridge) (ft^2)	400	Channel width at the bridge. Difference between subarea break-points defining banks at bridge, line 93 of WSPRO output.
W_2 (Bridge) (ft)	380	Channel width at bridge, less 4 channel pier widths (6.08 m).
S_f (ft/ft)	0.002	Average unconstricted energy slope. Defined as the headloss listed on line 318 or 322 of the WSPRO output divided by the distance between cross sections listed on lines 316, 319, and 323.

Table H.2. Hydraulic Variables from WSPRO for Estimation of Clear-water Contraction Scour on Left Overbank.		
		Remarks
Q (cfs)	30,000	Total discharge, (see Table H.1).
Q_{chan} (Bridge) (cfs)	27,176.4	Flow in main channel at bridge. Determined in live-bed computation of Step 3A.
Q_2 (Bridge) (cfs)	2,823.6	Flow in left overbank through bridge. Determined by subtracting Q_{chan} (listed above) from total discharge through bridge.
D_m (Bridge Overbank) (ft)	0.00825	Grain size of left overbank area. $D_m = 1.25\ D_{50}$.
$W_{setback}$ (Bridge)(ft)	226	Topwidth of left overbank area (SA #1) at bridge. Line 243, of WSPRO output.
$W_{contracted}$ (Bridge) (ft)	216	Set back width less two pier widths (10 ft)
A_{left} (Bridge) (ft^2)	600	Area of left overbank at the bridge. Line 243 of WSPRO output, SA #1.

Table H.3. Hydraulic Variables from WSPRO for Estimation of Pier Scour (Conveyance Tube Number 12).		
		Remarks
V_1 (ft/s)	12.43	Velocity in conveyance tube #12. Line 224 of WSPRO output.
Y_1 (ft)	9.21	Mean depth of tube #12. Computed as area divided by topwidth of conveyance tube.

Table H.4. Hydraulic Variables from WSPRO for Estimation of Abutment Scour Using Froehlich's Equation for Left Abutment.		
		Remarks
Q (cfs)	30,000	Total discharge (Table H.1)
q_{tube} (cfs)	1,500	Discharge per equal conveyance tube, defined as total discharge divided by 20.
#Tubes	3.5	Number of approach section conveyance tubes which are obstructed by left abutment. Determined by super-imposing abutment geometry onto the approach section (Figure H.4)
Q_e (cfs)	5,250	Flow in left overbank obstructed by left abutment and approach embankment. Determined by multiplying # Tubes and q_{tube}.
A_e (left abut.) (ft^2)	2,910	Area of approach section conveyance tubes number 1, 2, 3, and half of tube 4. Line 266 of WSPRO output.
L (ft)	766.65	Length of abutment projected into flow, determined by adding top widths of approach section conveyance tubes number 1, 2, 3, and half of tube 4. Line 265 of WSPRO output.
L′ (ft)	536.6	Length of active flow obstructed by embankment. Width of approach section conveyance tube directly upstream of abutment times the number of conveyance tubes blocked by the embankment (951.8-798.5) x 3.5 = 536.6. Note: Conveyance tube widths from line 265 of WSPRO output.

Table H.5. Hydraulic Variables from WSPRO for Estimation of Abutment Scour Using HIRE Equation for Left Abutment.		
		Remarks
V_{tube} (ft/s) (Bridge x-Section)	4.33	Mean velocity of conveyance tube #1, adjacent to left abutment. Line 216 of WSPRO output.
y_1 (ft) (Bridge x-Section)	2.68	Average depth of conveyance tube #1. Computed as area divided by topwidth of conveyance tube

Table H.6. Hydraulic Variables from WSPRO for Estimation of Abutment Scour Using HIRE Equation for Right Abutment.		
		Remarks
V_{tube} (ft/s)	6.12	Mean velocity of conveyance tube 20, adjacent to right abutment. Line 228 of WSPRO output.
y_1 (ft)	4.11	Average depth of conveyance tube 20. Computed as area divided by topwidth of conveyance tube.

In the overbank area adjacent to the left abutment, clear-water scour will occur. This is because the overbank areas upstream of the bridge are vegetated, and because the velocities in these areas will be low. Thus, returning overbank flow which will pass under the bridge adjacent to the left abutment will not be transporting significant amounts of material to replenish the scour on the left overbank adjacent to the left abutment.

Because of this, two computations for contraction scour will be required. The first computation, which will be illustrated in Step 3A will determine the magnitude of the contraction scour in the main channel. The second computation, which is illustrated in Step 3B will utilize the clear-water equation for the left overbank area. Hydraulic data for these two computations are presented in Tables H.1 and H.2 for the channel and left overbank contraction scour computations, respectively.

Table H.3 lists the hydraulic variables which will be used to estimate the local scour at the piers (Step 5). These hydraulic variables were determined from a plot of the velocity distribution derived from the WSPRO output (Figure H.5). For this example the highest velocities and flow depths in the bridge cross section will be used (at conveyance tube number 12). Only one pier scour computation will be completed because the possibility of thalweg shifting and lateral migration will require that all of the piers be set assuming that any pier could be subjected to the maximum scour producing variables.

Local scour at the left abutment and right abutment will be illustrated in Steps 6A and B using the HIRE equation. Scour variables derived from the WSPRO output for these computations are presented in Tables H.4 and H.5.

H.3 STEP 2: ANALYZE LONG-TERM BED ELEVATION CHANGE

Evaluation of stage discharge relationships and cross sectional data obtained from other agencies do not indicate progressive aggradation or degradation. Also, long-term aggradation or degradation are not evident at neighboring bridges. Based on these observations, the channel is relatively stable vertically, at present. Furthermore, there are no plans to change the local land use in the watershed. The forested areas of the watershed are government-owned and regulated to prevent wide spread fire damage, and instream gravel mining is prohibited. These observations indicate that future aggradation or degradation of the channel, due to changes in sediment delivery from the watershed, are minimal.

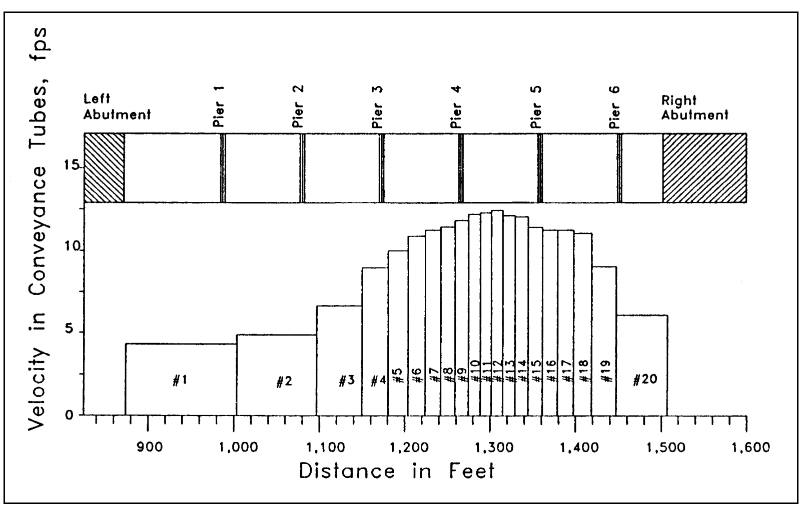

Figure H.5. Velocity distribution at bridge crossing.

Based on these observations, and due to the lack of other possible impacts to the river reach, it is determined that the channel will be relatively stable vertically at the bridge crossing and long-term aggradation or degradation potential is considered to be minimal. However, there is evidence that the channel is unstable laterally. This will need to be considered when assessing the total scour at the bridge.

H.4 STEP 3A: COMPUTE THE MAGNITUDE OF THE GENERAL (CONTRACTION) SCOUR IN MAIN CHANNEL

As a precursor to the computation of contraction scour in the main channel under the bridge, it is first necessary to determine whether the flow condition in the main channel is either live-bed or clear-water. This is determined by comparing the critical velocity for sediment movement at the approach section to the average channel velocity of the flow at the approach section as computed using the WSPRO output. This comparison is conducted using the average velocity in the main channel of the approach section to the bridge. If the average computed channel velocity is greater than the critical velocity, the live-bed equation should be used. Conversely, if the average channel velocity is less than the critical velocity, the clear-water equation is applicable. The following computations are based on the quantities tabulated in Table H.1.

The discharge in the main channel of the approach section is determined from the ratio of the conveyance in the main channel to the total conveyance of the approach section. By multiplying this ratio by the total discharge, the discharge in the main channel at the approach section (Q_1) is computed.

$$Q_1 = Q\ (K_1 / K_{total}) = 30{,}000\ \text{cfs} \left(\frac{680{,}989}{1{,}414{,}915}\right)$$

$$Q_1 = 14{,}439\ \text{cfs}$$

The average velocity in the main channel of the approach section is determined by dividing the discharge computed in Equation H.1 by the cross-sectional area of the main channel.

$$V_1 = (Q_1 / A_c) = \left(\frac{14{,}439}{3{,}467}\right) = 4.16\ \text{ft/s}$$

The average flow depth in the approach section is determined by dividing the flow area by the topwidth of the channel.

$$y_1 = \left(A_1 / \text{TOPW}\right) = \left(\frac{3{,}467}{400}\right) = 8.7\ \text{ft}$$

The channel velocity is compared to the critical velocity of the D_{50} size for sediment movement (V_c) to determine whether the flow condition is either clear-water or live-bed.

$$V_c = 11.2\ y_1^{1/6}\ D_{50}^{1/3}$$

$$V_c = 11.2\ (8.7\ \text{ft})^{1/6}\ (0.0066\ \text{ft})^{1/3}$$

$V_c = 3.0 \text{ ft/s}$

Since the average velocity in the main channel is greater than the critical velocity ($V_1 > V_c$), the flow condition will be live-bed. The following computations illustrate the computation of the contraction scour using the live-bed equation.

The following computation determines the mode of bed material transport and the factor k_1. All hydraulic parameters which are needed for this computation are listed in Table H.1.

The hydraulic radius of the approach channel is:

$$R = \frac{A_c}{WETP} = \frac{3{,}467 \text{ ft}^2}{400 \text{ ft}} = 8.7 \text{ ft}$$

Notice that the hydraulic radius of the approach is equal to the average flow depth computed earlier (Equation H.3). This condition indicates that the channel is wide with its width greater than 10 times the flow depth. **If the width was less than 10 times the average flow depth, the channel could not be assumed to be wide and the hydraulic radius would deviate from the average flow depth**.

The average shear stress on the channel bed is:

$$\tau_o = \gamma R S$$

$$\tau_o = (62.4 \text{ lb/ft}^3)(8.7 \text{ ft})(0.002 \text{ ft/ft}) = 1.08 \text{ lb/ft}^2$$

The shear velocity in the approach channel is:

$$V_* = (\tau_o / \rho)^{0.5} = (1.08 / 1.94)^{0.5} = 0.75 \text{ ft/s}$$

Bed material is sand with $D_{50} = 0.0066$ ft

Fall velocity (ω) = 0.9 ft/s from Figure 5.8 at 20°C and D_s = 2 mm

Therefore

$$\frac{V_*}{\omega} = \frac{0.75}{0.9} = 0.83$$

From the above, the coefficient k_1 is determined (from the discussion for Equation 5.2) to be equal to 0.64 which indicates that the mode of bed material transport is a mixture of suspended and contact bed material discharge.

The discharge in the main channel at the bridge (Q_2) is determined from the ratio of conveyances for the bridge section. This procedure for obtaining the discharge is similar to the procedure used to obtain the discharge in the main channel of the approach which was previously illustrated in Equation H.1.

$$Q_2 = Q(K_2 / K_{total}) = 30{,}000 \text{ cfs}\left(\frac{392{,}654}{433{,}451}\right)$$

$$Q_2 = 27{,}176 \text{ cfs}$$

The channel widths at the approach and bridge section are given in Table H.1. Therefore all parameters to determine live-bed contraction scour have been determined and Equation 5.2 can be employed.

$$\frac{y_2}{y_1} = \left(\frac{Q_2}{Q_1}\right)^{6/7}\left(\frac{W_1}{W_2}\right)^{k_1}$$

$$\frac{y_2}{8.7} = \left(\frac{27{,}176}{14{,}439}\right)^{6/7}\left(\frac{400}{380}\right)^{0.64} = 1.78$$

$$y_2 = (8.7)(1.78) = 15.5 \text{ ft}$$

Live-bed contraction scour is calculated by subtracting the flow depth in the bridge (y_0) from y_2. The bridge channel flow depth (y_0) is the area divided by the topwidth, y_0 = 2510 ft^2/400 ft = 6.3 ft. Therefore, the depth of contraction scour in the main channel is:

$$y_s = y_2 - y_0 = 15.5 \text{ ft} - 6.3 \text{ ft} = 9.2 \text{ ft}$$

This amount of contraction scour is large and could be minimized by increasing the bridge opening, providing for relief bridges in the overbank, or in some cases, providing for highway approach overtopping.

If this were the design of a new bridge, the excessive backwater (2 ft) would require a change in the design to meet FEMA backwater requirements. The increase in backwater is obtained by subtracting the elevation given in line 322 from the elevation given in line 340 in Appendix G. However, in the evaluation of an existing bridge for safety from scour, this amount of contraction scour could occur and the scour analysis should proceed.

H.5 STEP 3B: COMPUTE GENERAL (CONTRACTION) SCOUR FOR LEFT OVERBANK

Clear-water contraction scour will occur in the overbank area between the left abutment and the left bank of bridge opening. Although the bed material in the overbank area is soil, it is protected by vegetation. Therefore, there would be no bed-material transport into the set-back bridge opening (clear-water conditions). The subsequent computations are based on the discharge and depth of flow passing under the bridge in the left overbank. These hydraulic variables were determined from the WSPRO output and are tabulated in Table H.2.

Computation of clear-water contraction scour (Equation 5.4)

$$y_2 = \left[\frac{0.0077\,Q^2}{(D_m^{2/3}\ W_{contracted}{}^2)}\right]^{3/7}$$

Computation of contraction scour flow depth in left overbank area under the bridge, y_2:

$$Y_2 = \left[\frac{0.0077\,(2{,}823.6\,CFS)^2}{(0.0083\,FT)^{2/3}\,(216\,FT)^2} \right]^{3/7} = 4.4\,FT$$

Computation of average flow depth in left overbank bridge section, y_0:

$$y_0 = \frac{A}{TOPW} = \frac{(600\ ft^2)}{(226\ m)} = 2.7\,ft$$

Therefore, the clear-water contraction scour in the left overbank of the bridge opening is:

$$y_s = y_2 - y_0 = 4.4\,ft - 2.7\,ft = 1.7\,ft$$

H.6 STEP 4: COMPUTE THE MAGNITUDE OF OTHER GENERAL SCOUR COMPONENTS

The crossing is on a relatively straight reach with no channel braiding, and there are no downstream controls of water surface elevations. Thus, the other general scour components (bend scour, confluence scour, etc) will not be a factor.

H.7 STEP 5: COMPUTE THE MAGNITUDE OF LOCAL SCOUR AT PIERS

It is anticipated that any pier under the bridge could potentially be subject to the maximum flow depths and velocities derived from the WSPRO hydraulic model (Table H.3). Therefore, only one computation for pier scour is conducted and assumed to apply to each of the six piers for the bridge. This assumption is appropriate based on the fact that the thalweg is prone to shifting and because there is a possibility of lateral channel migration.

H.7.1 Computation of Pier Scour

The Froude Number for the pier scour computation is based on the hydraulic characteristics of conveyance tube number 12. Therefore:

$$Fr_1 = \frac{V}{(g\ y_1)^{0.5}} = \frac{12.43\ ft/s}{[(32.2\ ft/s^2)\ (9.21 ft)]^{0.5}}$$

$$Fr_1 = 0.72$$

For a round-nose pier, aligned with the flow and sand-bed material:

$$K_1 = K_2 = K_4 = 1.0$$

For plane-bed condition:

$$K_3 = 1.1$$

Using Equation 6.3:

$$\frac{y_s}{y_1} = 2.0\ K_1\ K_2\ K_3\ K_4 \left(\frac{a}{y_1}\right)^{0.65} Fr_1^{0.43}$$

$$\frac{y_s}{9.21\,ft} = 2\ (1)\ (1)\ (1.1)\ (1) \left(\frac{5.0\,ft}{9.21\,ft}\right)^{0.65} (0.72)^{0.43}$$

$$\frac{y_s}{9.21} = 1.28$$

$$y_s = 11.8\ ft$$

From the above computation the maximum local pier scour depth will be 11.8 ft.

H.7.2 Correction for Angle of Attack

The above computation assumes that the piers are aligned with the flow (skew angles are less than 5°). However, if the piers were skewed to the flow by more than 5°, the value of y_s/y_1, as computed above, would need to be adjusted by K_2. The following computations illustrate the adjustment for piers skewed 10°.

$$\frac{L}{a} = \frac{40\,ft}{5\,ft} = 8$$

K_2 can then be obtained by using Equation 6.4 for an L/a of 8 and a 10° angle of attack. For this example, K_2=1.67. Applying this correction:

$$\frac{y_s}{9.21} = 1.67\ (1.28) = 2.1$$

$$y_s = 19.3\,ft$$

Therefore, the maximum local pier scour depth for a pier angled 10° to the flow is 19.3 ft.

H.7.3 Discussion of Pier Scour Computation

Although the estimated local pier scour would probably not occur at each pier, the possibility of thalweg shifting, which was identified in the Level 1 analysis, precludes setting the piers at different depths even if there were a substantial savings in cost. This is because any of the piers could be subjected to the worst-case scour conditions.

It is also important to assess the possibility of lateral migration of the channel. This possibility can lead to directing the flow at an angle to the piers, thus increasing local scour. Countermeasures to minimize this problem could include riprap for the channel banks both up- and downstream of the bridge, and installation of guide banks to align flow through the bridge opening.

The possibility of lateral migration precludes setting the foundations for the overbank piers at a higher elevation. Therefore, in this example the foundations for the overbank piers should be set at the same elevation as the main channel piers.

H.8 STEP 6A: COMPUTE THE MAGNITUDE OF LOCAL SCOUR AT LEFT ABUTMENT

H.8.1 Computation of Abutment Scour Depth Using Froehlich's Equation

For spill-through abutments, K_1 = 0.55. For this example, the abutments are set perpendicular to the flow; therefore, K_2 = 1.0. Abutment scour can be estimated using Froehlich's equation with data derived from the WSPRO output (Table H.4).

$$y_a = \frac{A_e}{L} = \frac{2{,}910\ \text{ft}^2}{766.6\ \text{ft}} = 3.80\ \text{ft}$$

The y_a value at the abutment is assumed to be the average flow depth in the overbank area. It is computed as the cross-sectional area of the left overbank cut off by the left abutment divided by the distance the left abutment protrudes into the overbank flow.

The average velocity of the flow in the left overbank (Figure H.4) which is cut off by the left abutment is computed as the discharge cutoff by the abutment divided by the area of the left overbank cut off by the left abutment.

$$V_e = \frac{Q_e}{A_e} = \frac{5{,}250\ \text{cfs}}{2{,}910\ \text{ft}^2} = 1.8\ \text{ft / s}$$

Using these parameters, the Froude Number of the overbank flow is:

$$Fr = \frac{V_e}{(g\ y_a)^{1/2}} = \frac{1.8\ \text{ft / s}}{[(32.2\ \text{ft / s}^2)\ (3.8\ \text{ft})]^{0.5}}$$

Fr = 0.16

Using Froehlich's equation (Equation 7.1):

$$\frac{y_s}{y_a} = 2.27\ K_1\ K_2 \left(\frac{L'}{y_a}\right)^{0.43} Fr^{0.61} + 1$$

$$\frac{y_s}{3.8} = 2.27\,(0.55)\ (1.0) \left(\frac{536.6}{3.8}\right)^{0.43} (0.17)^{0.61} + 1$$

$$\frac{y_s}{3.8\,ft} = 4.56$$

$$y_s = 17.3\,ft$$

Using Froehlich's equation, the abutment scour at the left abutment is computed to be 17.3 ft.

H.8.2 Computation of Abutment Scour Depth Using the HIRE Equation

The HIRE equation for abutment is applicable for this situation because L/y_1 is greater than 25.

The HIRE equation is based on the velocity and depth of the flow passing through the bridge opening adjacent to the abutment end which is listed in Table H.5. Therefore, the Froude Number of this flow is:

$$Fr_1 = \frac{4.33\,ft/s}{[(32.2\ ft/s^2)\ (2.68\,ft)]^{0.5}} = 0.47$$

Using the HIRE equation with K_1 = 0.55 and K_2 = 1.0 (Equation 7.2):

$$\frac{y_s}{2.68\,ft} = 4\ Fr_1^{0.33} = 4\ (0.47)^{0.33} = 3.12$$

$$y_s = 8.4\ ft$$

From the above computation, the depth of scour at the left abutment as computed using the HIRE equation, is 8.4 ft.

H.9 STEP 6B: COMPUTE THE MAGNITUDE OF LOCAL SCOUR AT RIGHT ABUTMENT

The HIRE equation for abutment is also applicable for the right abutment since L/y_1 is greater than 25.

The HIRE equation is based on the velocity and depth of the flow passing through the bridge opening adjacent to the end of the right abutment and listed in Table H.6. The Froude Number of this flow is:

$$Fr_1 = \frac{6.12\,ft/s}{[(32.2\,ft/s^2)\ (4.11ft)]^{0.5}} = 0.53$$

Using the HIRE equation with K_1 = 0.55 and K_2 = 1.0:

$$\frac{y_s}{4.11ft} = 4\,Fr_1^{0.33} = 4\,(0.53)^{0.33} = 3.24$$

$y_s = 13.3\,\text{ft}$

From the above computation, the depth of scour at the right abutment, as computed using the HIRE equation is 13.3 ft.

H.10 DISCUSSION OF ABUTMENT SCOUR COMPUTATIONS

Abutment scour as computed using the Froehlich equation[70] will generally result in deeper scour predictions than will be experienced in the field. These scour depths could occur if the abutments protruded into the main channel flow, or when a uniform velocity field is cut off by the abutment in a manner that most of the returning overbank flow is forced to return to the main channel at the abutment end. For most cases, however, when the overbank area, channel banks and area adjacent to the abutment are well vegetated, scour depths as predicted with the Froehlich equation will probably not occur.

All of the abutment scour computations (left and right abutments) assumed that the abutments were set perpendicular to the flow. If the abutments were angled to the flow, a correction utilizing K_2 would be applied to Froehlich's equation and to the equation from HDS 6.[22] However the adjustment for skewed abutments is minor when compared to the magnitude of the computed scour depths. For example, if the abutments for this example problem were angled 30° upstream ($\theta = 90° + 30° = 120°$), the correction for skew would increase the computed depth of abutment scour by no more than 3 to 4 percent for the Froehlich and HIRE equation, respectively.

H.11 STEP 7: PLOT TOTAL SCOUR DEPTH AND EVALUATE DESIGN

As a final step, the results of the scour computations are plotted on the bridge cross section and carefully evaluated (Figure H.6). For this example, only the computations for pier scour with piers aligned with the flow were plotted and the abutment scour computations reflect the results from the HIRE equation. The topwidth of the local scour holes is suggested as 2.0 y_s.

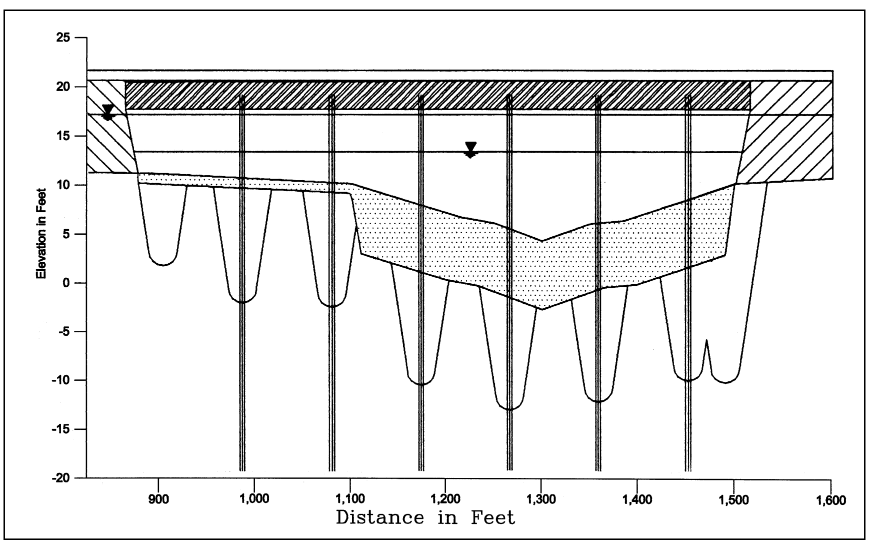

Figure H.6. Plot of total scour for example problem.

It is important to evaluate carefully the results of the scour computations. For example, although the total scour plot indicates that the total scour at the overbank piers is less than for the channel piers, this does not indicate that the foundations for the overbank piers can be set at a higher elevation. Due to the possibility of channel and thalweg shifting, all of the piers should be set to account for the maximum total scour. Also, the computed contraction scour is distributed uniformly across the channel in Figure H.6. However, in reality this may not be what would happen. With the flow from the overbank area returning to the channel, the contraction scour could be deeper at both abutments. The use of guide banks would distribute the contraction scour more uniformly across the channel. This would make a strong case for guide banks in addition to the protection they would provide to the abutments. The stream tube velocities could be used to distribute the scour depths across this section.

The plot of the total scour also indicates that there is a possibility of overlapping scour holes between the sixth pier and right abutment, and it is not clear from where the right abutment scour should be measured, since the abutment is located at the channel bank. Both of these uncertainties should be avoided for replacement and new bridges whenever possible. Consequently, it would be advisable to set the right abutment back from the main channel. This would also tend to reduce the magnitude of contraction scour in the main channel.

The possibility of lateral migration of the channel will have an adverse effect on the magnitude of the pier scour. This is because lateral migration will most likely skew the flow to the piers. This problem can be minimized by using circular piers. An alternative approach would be to install guide banks to align the flow through the bridge opening.

A final concern relates to the location and depth of contraction scour in the main channel near the second pier and toe of the right abutment. At these locations, contraction scour in the main channel could increase the bank height to a point where bank failure and sloughing would occur. It is recommended that the existing bank lines be protected with revetment (i.e., riprap, gabions, etc.). Since the river has a history of channel migration, the bridge inspection and maintenance crews should be briefed on the nature of this problem so that any lateral migration can be identified.

The plot of the scour prism in Figure H.6 should be reploted to show the potential for the scour to occur at any location in the bridge opening. This is shown in Figure H.7

H.12 COMPLETE THE GENERAL DESIGN PROCEDURE

This design problem uses Steps 1 through 7 of the specific design approach (Chapter 2) and completes Steps 1 through 6 of the general design procedure in Chapter 2. The design must now proceed to Steps 7 and 8, which include bridge foundation analysis and consideration of the check for superflood. This is not done for this example problem.

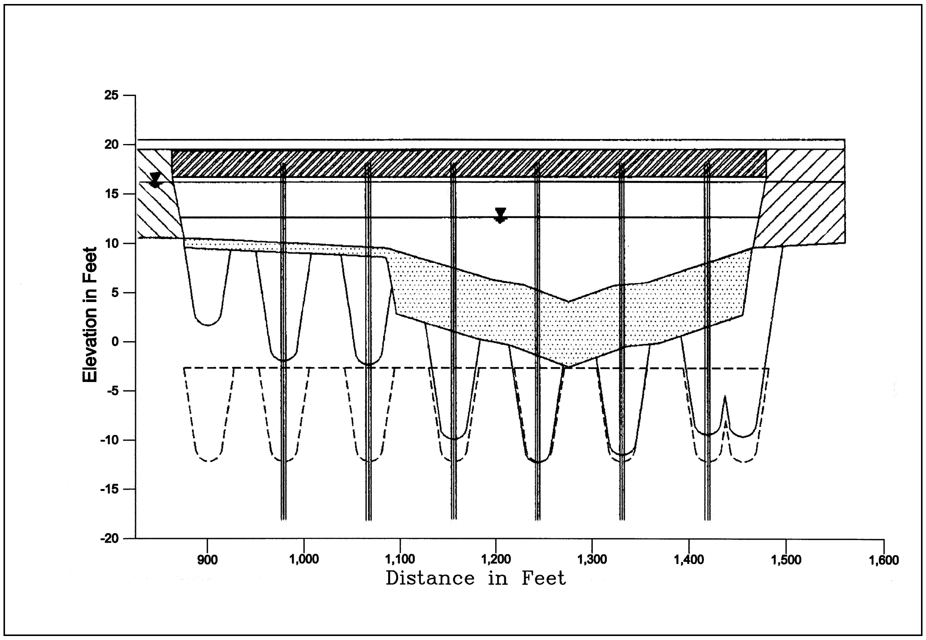

Figure H.7. Revised plot of total scour for example problem.

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APPENDIX I

FHWA TECHNICAL ADVISORY T 5140.23

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EVALUATING SCOUR AT BRIDGES

T 5140.23 October 28, 1991

1. PURPOSE. To provide guidance on developing and implementing a scour evaluation program for:

   a. designing new bridges to resist damage resulting from scour;

   b. evaluating existing bridges for vulnerability to scour;

   c. using scour countermeasures; and

   d. improving the state-of-practice of estimating scour at bridges.

2. CANCELLATION. Technical Advisory T 5140.20, Scour at Bridges, dated September 16, 1988, is cancelled.

3. BACKGROUND.

   a. The need to minimize future flood damage to the Nation's bridges requires that additional attention be devoted to developing and implementing improved procedures for designing, protecting and inspecting bridges for scour. (See National Bridge Inspection Standards, 23 CFR 650 Subpart C.) Current information on this subject has been assembled in the Federal Highway Administration (FHWA) design publication Hydraulic Engineering Circular (HEC) 18, "Evaluating Scour at Bridges," FHWA-IP-90-017 (*FHWA NHI 01-001, fourth edition*).

   b. Paragraph 4 contains the FHWA recommendations for developing and implementing a scour evaluation program. The recommendations have been developed based on the review and evaluation of the existing policies and guidance pertaining to bridge scour set forth in paragraph 5. The procedures in HEC 18 provide approaches for implementing these recommendations.

4. RECOMMENDATIONS FOR DEVELOPING AND IMPLEMENTING A SCOUR EVALUATION PROGRAM. Every bridge over a waterway, whether existing or under design, should be evaluated as to its vulnerability to scour in order to determine the prudent measures to be taken for its protection. Most waterways can be expected to experience scour over a bridge's service life (which could approach 100 years). Exceptions might include waterways in massive, competent rock formations where scour and erosion occur on a scale that is measured in centuries. [See HEC 18, Chapter 2 (*Chapter 3 in the fourth edition*)]. The added cost of making a bridge less vulnerable to scour is small when compared to the total cost of a failure which can easily be two or three times the original cost of the bridge. Moreover, the need to ensure public safety and to minimize the adverse effects stemming from bridge closures requires the best effort to improve the state-of-practice of designing and maintaining bridge foundations to resist the effects of scour. The recommendations listed below summarize the essential elements which should be addressed in developing a program for evaluating bridges and providing countermeasures for scour. Detailed guidance regarding approaches for implementing the recommendations is included in HEC 18.

a. Interdisciplinary Team. Scour evaluations of new and existing bridges should be conducted by an interdisciplinary team comprised of hydraulic, geotechnical and structural engineers. [See HEC 18, Chapters 3 and 5 (*Chapters 2 and 10 in the fourth edition*)].

b. New Bridges. Bridges over tidal and non-tidal waterways with scourable beds should withstand the effects of scour from a superflood (a flood exceeding the 100-year flood) without failing; i.e., experiencing foundation movement of a magnitude that requires corrective action.

(1) Hydraulic studies should be prepared for bridges over waterways in accordance with Article 1.3.2 of the Standard Specifications for Highway Bridges of the American Association of State Highway and Transportation Officials (AASHTO) and the floodplain regulation of the FHWA as set forth in 23 CFR 650, Subpart A.

(2) Hydraulic studies should include estimates of scour at bridge piers and evaluation of abutment stability. Bridge foundations should be designed to withstand the effects of scour without failing for the worst conditions resulting from floods equal to or less than the 100-year flood. [See HEC 18, Chapters 3 and 4 (*Chapter 2 in the fourth edition*)]. Bridge foundations should be checked to ensure that they will not fail due to scour resulting from the

occurrence of a superflood on the order of magnitude of a 500-year flood. [See HEC 18,Chapter 3, (*Chapter 2 in the fourth edition*)].

(3) The geotechnical analysis of bridge foundations should be performed on the basis that all stream bed material in the scour prism above the total scour line for the design flood (for scour) has been removed and is not available for bearing or lateral support. In addition, the ratio of ultimate to applied loads should be greater than 1.0 for conditions of scour for the superflood. [See HEC 18, Chapter 3 (*Chapter 2 in the fourth edition*)].

(4) Data on scour at bridge piers and abutments should be collected and analyzed in order to improve existing procedures for estimating scour. (See HEC 18, Chapter 1.)

c. Existing Bridges. All existing bridges over tidal and non-tidal waterways should be evaluated for the risk of failure from scour during the occurrence of a superflood on the order of magnitude of a 500-year flood. [See HEC 18, Chapter 5 (*Chapter 10 in the fourth edition*)].

(1) An initial screening process should identify bridges susceptible to scour and establish a priority list for evaluation. [See HEC 18, Chapter 5 (*Chapter 10 in the fourth edition*)].

(2) Bridge scour evaluations should be conducted for each bridge to determine whether it is scour critical. A scour critical bridge is one with abutment or pier foundations which are rated as unstable due to:

(a) observed scour at the bridge site or

(b) a scour potential as determined from a scour evaluation study. [See HEC 18, Chapter 5 (*Chapter 10 in the fourth edition*)] .

(3) The procedures in Chapter 5 of HEC 18 (*Chapter 10 of the fourth edition*) should be followed in conducting and documenting the results of scour evaluation studies

d. Scour Critical Existing Bridges. A plan of action should be developed for each existing bridge determined to be scour critical. [See HEC 18, Chapter 5 (*Chapters 2 and 10 of the fourth edition*)].

(1) The plan of action should include instructions regarding the type and frequency of inspections to be

made at the bridge, particularly in regard to monitoring the performance and closing of the bridge, if necessary, during and after flood events. [See HEC 18, Chapter 7 (*Chapter 12 in the fourth edition*)].

(2) The plan of action should include a schedule for the timely design and construction of scour countermeasures determined to be needed for the protection of the bridge. [See HEC 18, Chapter 7 (*Chapter 12 in the fourth edition*)].

e. Bridge Inspectors. Bridge inspectors should receive appropriate training and instruction in inspecting bridges for scour. [See HEC 18, Chapter 6 (*Chapters 11 and 12 in the fourth edition*)].

(1) The bridge inspector should accurately record the present condition of the bridge and the stream. At least one cross section at each bridge should be documented and compared with previously recorded cross section(s) at the site. Pier locations and footing elevations should be included.

(2) The bridge inspector should identify conditions that are indicative of potential problems with scour and stream stability.

(3) Effective notification procedures should be available to permit the inspector to promptly communicate findings of actual or potential scour problems to others for further review and evaluation.

(4) Special attention should be focused on the routine inspection of scour critical bridges and on the monitoring and closing as necessary of scour critical and other bridges during and after floods.

5. EXISTING POLICY AND GUIDANCE. The following existing policy and guidance serve as the basis for the recommendations set forth in paragraph 4.

a. AASHTO Standard Specifications for Highway Bridges. The FHWA has accepted these specifications for the design of highway bridges. The 1991 Interim Specifications contain requirements for designing bridges to resist scour. Particular attention is directed to Article 1.3.2, Hydraulic Studies, which advises that, "Hydraulic studies . . . should include applicable parts of the following outline:" Included in this outline is item 1.3.2.3 (b), Estimated scour depth at piers and abutments of proposed structures.

b. AASHTO Manual for Bridge Maintenance. The FHWA endorses the guidance contained in this 1987 Manual for Bridge Maintenance. Particular attention is directed to the following two statements which support the recommendations contained in this Technical Advisory:

(1) "The primary function of the bridge maintenance program is to maintain the bridges in a condition that will provide for safe and uninterrupted traffic flows. The protection of the investment in the structure facility through well programmed repairs is second only to the safety of traffic and to the structure itself." (p. 25.)

(2) "Determining an effective solution to a stream bed or river problem is difficult. Settlement of foundations, local scour, bank erosion, and channel degradation are complex problems and cannot be solved by one or two prescribed methods. Hydraulic, geotechnical, and structural engineers are all needed for consultation prior to undertaking the solution of a serious maintenance problem. In some cases, certain remedial work could actually be detrimental to the structure." (p. 155.)

c. AASHTO Manual for Maintenance Inspection of Bridges. The FHWA endorses the guidance provided in the current version of this manual which serves as a standard and provides uniformity in the procedures and policies in determining the physical condition and maintenance needs of bridges. The manual emphasizes the importance of documenting and comparing cross sections taken upstream of bridges over time to discern potential scour problems.

d. Code of Federal Regulations, 23 CFR 650, Subpart C. The 1989 revision of this FHWA regulation on the National Bridge Inspection Standards requires that bridge owners maintain a bridge inspection program that includes procedures for underwater inspection. This Technical Advisory and HEC 18 provide guidance on the development and implementation of procedures for evaluating bridge scour to meet the requirements of the regulation.

e. Memorandum From the Director, Office of Engineering, to Regional Federal Highway Administrators and Direct Federal Program Administrator Dated April 17, 1987. This memorandum stated in part, "Each State should evaluate the risk of its bridges being subjected to scour damage during floods on the order of a 100 to 500 year return period or more."

f. <u>FY 1991 High Priority Research Program of the FHWA</u>. The FHWA recognizes the subject of scour at bridges as a long range high priority national program area for research and recommends that appropriate studies be carried out to improve the state-of-practice of designing new bridges and evaluating existing bridges for scour.

Thomas O. Willett, Director
Office of Engineering

APPENDIX J

FHWA 1995 RECORDING AND CODING GUIDE FOR THE STRUCTURE INVENTORY AND APPRAISAL OF THE NATION'S BRIDGES

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APPENDIX J

FHWA 1995 Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges

J.1 CODING GUIDE

This appendix contains relevant material for recording and coding the results of the evaluation of scour at bridges (Items 60, 61, 71, 92, 93, 113). The material is excerpted from the Federal Highway Administration document "Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges," dated 1995.(1) Recently implemented revisions are included on Items 60 and 113 as shown in the enclosed extracts from the Coding Guide (see Attachment 1, Appendix J).

J.2 COUNTERMEASURES

If a bridge is scour critical (Item 113 code of 3 or less), a countermeasure should be considered to decrease the risk of failure of the foundation. If a countermeasure is installed using the criteria listed below, the bridge owner has the following Item 113 coding options: (A) use a code of 8 if the bridge foundation can be determined to be stable by assessment or by installation of properly designed countermeasures, or (B) use a code of 7 to indicate a countermeasure has been installed to mitigate an existing problem with scour and to reduce the risk of failure during a flood event.

In general, the riprap must be designed to withstand the appropriate bridge structure design frequency. The criteria apply to existing bridges. All new bridge designs must have stable foundations designed for the estimated hydraulics and scour. The criteria that must be met are:

1. The countermeasure must be designed to provide the same level of stability as the bridge structure. For example, if the bridge structure was designed using a 100-year event then the countermeasure must be stable and withstand a 100-year event.

2. The design must be supported by appropriate hydraulics and scour computations. These may include the incipient roadway overtopping event, design event, 100-year flood and the 500-year flood. If the bridge design was not supported by appropriate hydraulics and scour computations, then these computations should be made to determine the actual level of service the bridge provides.

3. A geotextile filter, geotextile bags, or fascine mat must be used (see HEC-23,(2) the FHWA publication, "Geosynthetic Design and Construction Guidelines,"(3) or HEC-11.(4)

4. For example, if riprap is used as a pier scour countermeasure, it should be sized according to the HEC-23(2) pier riprap sizing equation or other appropriate approach. If a class of riprap is used, then the median size of the riprap class must equal or exceed the design median size (D_{50}). Figures J.1 and J.2 show preliminary recommendations for pier riprap design.

- The top of the riprap should be located at the channel bed elevation or, if a complete channel riprap armor is installed, flush with the riprap armor at the pier or abutment. Riprap mounded around the pier is not acceptable.

- The required thickness of riprap is dependent on the amount of contraction scour expected during the design event. The thickness will be a minimum of three times the median riprap size ($3xD_{50}$) unless the computed contraction scour amount is greater. If the contraction scour exceeds $3xD_{50}$ then the bottom of the riprap must extend down to the contraction scour elevation and the top of the riprap remains at the channel bed.

- The riprap will extend at least twice the pier width or 1.2 times the computed pier scour depth, which ever is greater, but may also be controlled by contraction scour. The riprap will launch away from the pier due to contraction scour. The post-event riprap configuration must be estimated using a 1V:1.5H slope to ensure that the riprap surface extends at least the pier width after the design event. Figures J.1 and J.2 show two methods for constructing pier riprap. In Figure J.1, the vertical riprap edge is achieved by using temporary sheet pile. Figure J.2 shows riprap placement using excavation only.

- The riprap must be inspected at a minimum interval of two years and, as a minimum, after any flood equaling or exceeding the 25-year recurrence interval.

J.3 REFERENCES

1. Federal Highway Administration, 1995, "Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges, Report No. FHWA-PD-96-001, U.S. Department of Transportation, Washington, D.C.

2. Lagasse, P.F., L.W. Zevenbergen, J.D. Schall, and P.E. Clopper, 2001, "Bridge Scour and Stream Instability - Countermeasures - Experience, Selection, and Design Guidelines, Hydraulic Engineering Circular No. 23, Second Edition, FHWA NHI 01-003, Federal Highway Administration, Washington, D.C.

3. Holz, D.H., B.R. Christopher, and R.R. Berg, 1995, "Geosynthetic Design and Construction Guidelines," National Highway Institute, Publication No. FHWA HI-95-038, Federal Highway Administration, Washington, D.C., May.

4. Brown, S.A. and E.S. Clyde, 1989, "Design of Riprap Revetment," Hydraulic Engineering Circular No. 11, FHWA-IP-016, prepared for FHWA, Washington, D.C.

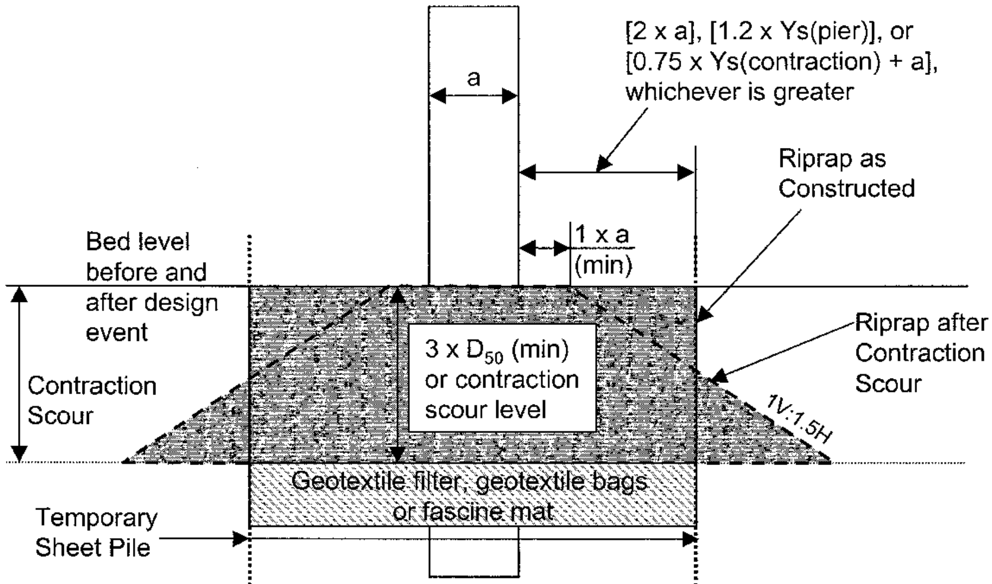

Figure J.1. Riprap design using temporary sheet pile.

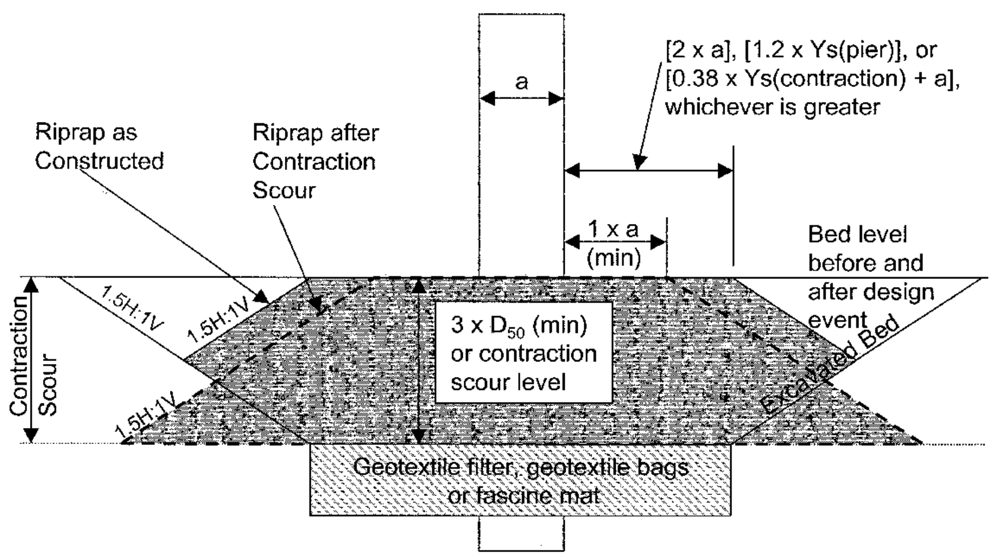

Figure J.2. Riprap design using excavation only.

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ATTACHMENT B

EXTRACTS FROM THE CODING GUIDE

Items 58 through 62 - Indicate the Condition Ratings

In order to promote uniformity between bridge inspectors, these guidelines will be used to rate and code Items 58, 59, 60, 61, and 62. The use of the AASHTO Guide for Commonly Recognized (CoRe) Structural Elements is an acceptable alternative to using these rating guidelines for Items 58, 59, 60, and 62, provided the FHWA translator computer program is used to convert the inspection data to NBI condition ratings for NBI data submittal.

Condition ratings are used to describe the existing, in-place bridge as compared to the as-built condition. Evaluation is for the materials related, physical condition of the deck, superstructure, and substructure components of a bridge. The condition evaluation of channels and channel protection and culverts is also included. Condition codes are properly used when they provide an overall characterization of the general condition of the entire component being rated. Conversely, they are improperly used if they attempt to describe localized or nominally occurring instances of deterioration or disrepair. Correct assignment of a condition code must, therefore, consider both the severity of the deterioration or disrepair and the extent to which it is widespread throughout the component being rated.

The load-carrying capacity will not be used in evaluating condition items. The fact that a bridge was designed for less than current legal loads and may be posted shall have no influence upon condition ratings.

Portions of bridges that are being supported or strengthened by temporary members will be rated based on their actual condition; that is, the temporary members are not considered in the rating of the item. (See Item 103 - Temporary Structure Designation for the definition of a temporary bridge.)

Completed bridges not yet opened to traffic, if rated, shall be coded as if open to traffic

Item 60 - Substructure 1 digit

This item describes the physical condition of piers, abutments, piles, fenders, footings, or other components. Rate and code the condition in accordance with the previously described general condition ratings. Code N for all culverts.

All substructure elements should be inspected for visible signs of distress including evidence of cracking, section loss, settlement, misalignment, scour, collision damage, and corrosion. The rating factor given to Item 60 should be consistent with the one given to Item 113 whenever a rating factor of 2 or below is determined for Item 113 - Scour Critical Bridges.

The substructure condition rating shall be made independent of the deck and superstructure.

Integral-abutment wingwalls to the first construction or expansion joint shall be included in the evaluation. For non-integral superstructure and substructure units, the substructure shall be considered as the portion below the bearings. For structures where the substructure and superstructure are integral, the substructure shall be considered as the portion below the superstructure.

The following general condition ratings shall be used as a guide in evaluating Items 58, 59, and 60:

<u>Code</u> <u>Description</u>

N NOT APPLICABLE
9 EXCELLENT CONDITION
8 VERY GOOD CONDITION - no problems noted.
7 GOOD CONDITION - some minor problems.
6 SATISFACTORY CONDITION - structural elements show some minor deterioration.
5 FAIR CONDITION - all primary structural elements are sound but may have minor section loss, cracking, spalling or scour.
4 POOR CONDITION - advanced section loss, deterioration, spalling or scour.
3 SERIOUS CONDITION - loss of section, deterioration, spalling or scour have seriously affected primary structural components. Local failures are possible. Fatigue cracks in steel or shear cracks in concrete may be present.
2 CRITICAL CONDITION - advanced deterioration of primary structural elements. Fatigue cracks in steel or shear cracks in concrete may be present or scour may have removed substructure support. Unless closely monitored it may be necessary to close the bridge until corrective action is taken.
1 "IMMINENT" FAILURE CONDITION - major deterioration or section loss present in critical structural components or obvious vertical or horizontal movement affecting structure stability. Bridge is closed to traffic but corrective action may put back in light service.

0 FAILED CONDITION - out of service - beyond corrective action.

Item 61 - Channel and Channel Protection 1 digit

This item describes the physical conditions associated with the flow of water through the bridge such as stream stability and the condition of the channel, riprap, slope protection, or stream control devices including spur dikes. The inspector should be particularly concerned with visible signs of excessive water velocity which may affect undermining of slope protection, erosion of banks, and realignment of the stream which may result in immediate or potential problems. Accumulation of drift and debris on the superstructure and substructure should be noted on the inspection form but not included in the condition rating.

Rate and code the condition in accordance with the previously described general condition ratings and the following descriptive codes:

Code	Description
N	Not applicable. Use when bridge is not over a waterway channel).
9	There are no noticeable or noteworthy deficiencies which affect the condition of the channel.
8	Banks are protected or well vegetated. River control devices such as spur dikes and embankment protection are not required or are in a stable condition.
7	Bank protection is in need of minor repairs. River control devices and embankment protection have a little minor damage. Banks and/or channel have minor amounts of drift.
6	Bank is beginning to slump. River control devices and embankment protection have widespread minor damage. There is minor stream bed movement evident. Debris is restricting the channel slightly.
5	Bank protection is being eroded. River control devices and/or embankment have major damage. Trees and brush restrict the channel.
4	Bank and embankment protection is severely undermined. River control devices have severe damage. Large deposits of debris are in the channel.
3	Bank protection has failed. River control devices have been destroyed. Stream bed aggradation, degradation or lateral movement has changed the channel to now threaten the bridge and/or approach roadway.
2	The channel has changed to the extent the bridge is near a state of collapse.
1	Bridge closed because of channel failure. Corrective action may put back in light service.
0	Bridge closed because of channel failure. Replacement necessary.

Item 71 - Waterway Adequacy   1 digit

This item appraises the waterway opening with respect to passage of flow through the bridge. The following codes shall be used in evaluating waterway adequacy (interpolate where appropriate). Site conditions may warrant somewhat higher or lower ratings than indicated by the table (e.g., flooding of an urban area due to a restricted bridge opening).

Where overtopping frequency information is available, the descriptions given in the table for chance of overtopping mean the following:

- Remote - greater than 100 years
- Slight - 11 to 100 years
- Occasional - 3 to 10 years
- Frequent - less than 3 years

Adjectives describing traffic delays mean the following:

- Insignificant - Minor inconvenience. Highway passable in a matter of hours.
- Significant - Traffic delays of up to several days.
- Severe - Long term delays to traffic with resulting hardship.

Functional Classification			
Principal Arterials – Interstates, Freeways, or Expressways	Other Principal and Minor Arterials and Major Collectors	Minor Collectors, Locals	Description
Code			
N	N	N	Bridge not over a waterway.
9	9	9	Bridge deck and roadway approaches above flood water elevations (high water). Chance of overtopping is remote.
8	8	8	Bridge deck above roadway approaches. Slight chance of overtopping roadway approaches.
6	6	7	Slight chance of overtopping bridge deck and roadway approaches.
4	5	6	Bridge deck above roadway approaches. Occasional overtopping of roadway approaches with insignificant traffic delays.

Functional Classification			Description
Principal Arterials - Interstates, Freeways, or Expressways	Other Principal and Minor Arterials and Major Collectors	Minor Collectors, Locals	
Code			
3	4	5	Bridge deck above roadway ap proaches. Occasional overtopping of roadway approaches with significant traffic delays.
2	3	4	Occasional overtopping of bridge deck and roadway approaches with significant traffic delays.
2	2	3	Frequent overtopping of bridge deck and roadway approaches with significant traffic delays.
2	2	2	Occasional or frequent overtopping of bridge deck and roadway approaches with severe traffic delays.
0	0	0	Bridge closed.

Item 92 - Critical Feature Inspection — 9 digits

Using a series of 3-digit code segments, denote critical features that need special inspections or special emphasis during inspections and the designated inspection interval in months as determined by the individual in charge of the inspection program. The designated inspection interval could vary from inspection to inspection depending on the condition of the bridge at the time of inspection.

Segment	Description	Length
92A	Fracture Critical Details	3 digits
92B	Underwater Inspection	3 digits
92C	Other Special Inspection	3 digits

For each segment of Item 92A, B, and C, code the first digit Y for special inspection or emphasis needed and code N for not needed. The first digit of Item 92A, B, and C must be coded for all structures to designate either a yes or no answer. Those bridges coded with a Y in Item 92A or B should be the same bridges contained in the Master Lists of fracture critical and special underwater inspection bridges. In the second and third digits of each segment, code a 2-digit number to indicate the number of months between inspections only if the first digit is coded Y. If the first digit is coded N, the second and third digits are left blank.

Current guidelines for the maximum allowable interval between inspections can be summarized as follows:

Fracture Critical Details	24 months
Underwater Inspection	60 months
Other Special Inspections	60 months

EXAMPLES:	Item	Code
A 2-girder system structure which is being inspected yearly and no other special inspections are required.	92A 92B 92C	Y12 N__ N__
A structure where both fracture critical and underwater inspection are being performed on a 1-year interval. Other special inspections are not required.	92A 92B 92C	Y12 Y12 N__
A structure has been temporarily shored and is being inspected on a 6-month interval. Other special inspections are not required.	92A 92B 92C	N__ N__ Y06

Item 93 - Critical Feature Inspection Date 12 digits

Code only if the first digit of Item 92A, B, or C is coded Y for yes. Record as a series of 4-digit code segments, the month and year that the last inspection of the denoted critical feature was performed.

Segment	Description	Length
93A	Fracture Critical Details	4 digits
93B	Underwater Inspection	4 digits
93C	Other Special Inspection	4 digits

For each segment of this item, when applicable, code a 4-digit number to represent the month and year. The number of the month should be coded in the first 2 digits with a leading zero as required and the last 2 digits of the year coded as the third and fourth digits of the field. If the first digit of any part of Item 92 is coded N, then the corresponding part of this item shall be blank.

EXAMPLES:	Item	Code
A structure has fracture critical members which were last inspected in March 1986. It does not require underwater or other special feature inspections.	93A 93B 93C	0386 (blank) (blank)
A structure has no fracture critical details, but requires underwater inspection and has other special features (for example, a temporary support) for whichthe State requires special inspection. The last underwater inspection was done in April 1986 and the last special feature inspection was done in November 1985.	93A 93B 93C	(blank) 0486 1185

Item 94 - Bridge Improvement Cost 6 digits

Code a 6-digit number to represent the estimated cost of the proposed bridge or major structure improvements in thousands of dollars. This cost shall include only bridge construction costs, excluding roadway, right of way, detour, demolition, preliminary engineering, etc. Code the base year for the cost in Item 97 - Year of Improvement Cost Estimate. Do not use this item for estimating maintenance costs.

This item must be coded for bridges eligible for the Highway Bridge Replacement and Rehabilitation Program. It may be coded for other bridges at the option of the highway agency.

EXAMPLES:		Code
Bridge Improvement Cost	$ 55,850	000056
	250,000	000250
	7,451,233	007451

Item 113 - Scour Critical Bridges 1 digit

Use a single-digit code as indicated below to identify the current status of the bridge regarding its vulnerability to scour. Evaluations shall be made by hydraulic/geotechnical/structural engineers. Guidance on conducting a scour evaluation is included in the FHWA Technical Advisory T 5140.23 titled, "Evaluating Scour at Bridges."[1] Detailed engineering guidance is provided in the Hydraulic Engineering Circular 18 titled "Evaluating Scour at Bridges."[2] Whenever a rating factor of 2 or below is determined for this item, the rating factor for Item 60 -- Substructure and other affected items (i.e., load ratings, superstructure rating) should be revised to be consistent with the severity of observed scour and resultant damage to the bridge. A plan of action should be developed for each scour critical bridge (see FHWA Technical Advisory T 5140.23, HEC 18 and HEC 23[3]). A scour critical bridge is one with abutment or pier foundation rated as unstable due to (1) observed scour at the bridge site (rating factor of 2, 1, or 0) or (2) a scour potential as determined from a scour evaluation study (rating factor of 3). It is assumed that the coding of this item has been based on an engineering evaluation, which includes consultation of the NBIS field inspection findings.

Code Description

N Bridge not over waterway.

U Bridge with "unknown" foundation that has not been evaluated for scour. Until risk can be determined, a plan of action should be developed and implemented to reduce the risk to users from a bridge failure during and immediately after a flood event (see HEC 23).

T Bridge over "tidal" waters that has not been evaluated for scour, but considered low risk. Bridge will be monitored with regular inspection cycle and with appropriate underwater inspections until an evaluation is performed ("Unknown" foundations in "tidal" waters should be coded U.)

9 Bridge foundations (including piles) on dry land well above flood water elevations.

8 Bridge foundations determined to be stable for the assessed or calculated scour condition. Scour is determined to be above top of footing (Example A) by assessment (i.e., bridge foundations are on rock formations that have been determined to resist scour within the service life of the bridge[4]), by calculation or by installation of properly designed countermeasures (see HEC 23).

7 Countermeasures have been installed to mitigate an existing problem with scour and to reduce the risk of bridge failure during a flood event. Instructions contained in a plan of action

have been implemented to reduce the risk to users from a bridge failure during or immediately after a flood event.

6 Scour calculation/evaluation has not been made. (Use only to describe case where bridge has not yet been evaluated for scour potential.)

5 Bridge foundations determined to be stable for assessed or calculated scour condition. Scour is determined to be within the limits of footing or piles (Example B) by assessment (i.e., bridge foundations are on rock formations that have been determined to resist scour within the service life of the bridge), by calculations or by installation of properly designed countermeasures (see HEC 23).

4 Bridge foundations determined to be stable for assessed or calculated scour conditions; field review indicates action is required to protect exposed foundations (see HEC 23).

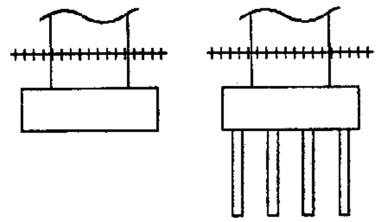
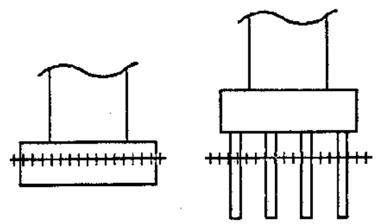
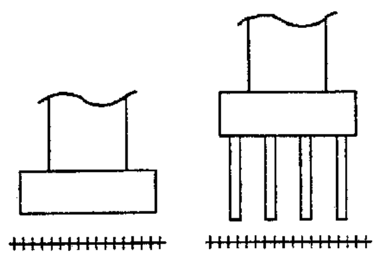
3 Bridge is scour critical; bridge foundations determined to be unstable for assessed or calculated scour conditions:
- Scour within limits of footing or piles. (Example B)
- Scour below spread-footing base or pile tips. (Example C)

2 Bridge is scour critical; field review indicates that extensive scour has occurred at bridge foundations, which are determined to be unstable by:
- a comparison of calculated scour and observed scour during the bridge inspection, or
- an engineering evaluation of the observed scour condition reported by the bridge inspector in Item 60.

1 Bridge is scour critical; field review indicates that failure of piers/abutments is imminent. Bridge is closed to traffic. Failure is imminent based on:
- a comparison of calculated and observed scour during the bridge inspection, or
- an engineering evaluation of the observed scour condition reported by the bridge inspector in Item 60.

0 Bridge is scour critical. Bridge has failed and is closed to traffic.

[1] FHWA Technical Advisory T 5140.23, Evaluating Scour at Bridges, dated October 28, 1991.
[2] HEC 18, Evaluating Scour at Bridges, Fourth Edition.
[3] HEC 23, Bridge Scour and Stream Instability Countermeasures, Second Edition.
[4] FHWA Memorandum "Scourability of Rock Formations," dated July 19, 1991.

EXAMPLES:	CALCULATED SCOUR DEPTH	ACTION NEEDED
A. Above top of footing		None - indicate rating of 8 for this item
B. Within limits of footing or piles		Conduct foundation structural analysis
C. Below pile tips or spread-footing base		Provide for monitoring and scour countermeasures as necessary
	SPREAD FOOTING (NOT FOUNDED IN ROCK) PILE FOOTING	

= Calculated scour depth

APPENDIX K

UNKNOWN FOUNDATIONS

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APPENDIX K

Unknown Foundations

K.1 INTRODUCTION

Bridges are classified as having unknown foundations when the type (spread footing, piles, columns), dimensions (length, width, or thickness), reinforcing, and/or elevation are unknown. They are classified as U in Item 113 of the coding guide (Appendix I). The screening program in the National Evaluation program has identified 90,000 bridges with unknown foundations. Research under the National Cooperative Highway Research Program (NCHRP) has investigated nondestructive testing methods which in many cases can determine pile length. This appendix provides a status report and guidance for protecting bridges with unknown foundations from scour.

K.2 PLAN OF ACTION

For bridges with unknown foundations a Plan of Action should be developed (see Chapter 12). The plan of action to take into consideration the service life of the bridge, the volume and type of traffic, and important of the highway (interstate, primary or rural farm to market). The Plan of Action includes:

- Describing the foundation and scour condition
- Timely installation of countermeasures to reduce the risk from scour (e.g., riprap.)
- Development and implementation of a scour monitoring and/or inspection program
- Development of a plan for closure of the bridge, if needed
- Determining if nondestructive test is economical and feasible to determine foundation characteristics
- Schedule timely design and construction of a new bridge or countermeasures to make the bridge safe from scour and stream instability

K.3 NONDESTRUCTIVE TESTING (NTD) RESEARCH

NCHRP Project 21-5 initiated in 1996, identified and tested the following NTD methods:[1,2]

- Sonic echo/impulse response
- Bending wave method
- Ultraseismic test method
- SASW method
- Dynamic foundation response method
- Borehole parallel seismic test method
- Borehole sonic method
- Borehole radar method
- Induction field method

As a result of the above research, a second phase of this project (NCHRP 21-5 (2)) was initiated to research and develop equipment, field techniques, and analysis methods for the most promising methods. The methods selected were:

- Ultraseismic (including sonic echo/impulse response and bending wave methods)
- Borehole of parallel seismic and induction field

In general the results of testing NTD methods have not been as satisfactory as the initial research indicated. The results of NCHRP Project 21-5 indicate that of all the surface and borehold methods, the Parallel Seismic test was found to have the broadest applications for determining the bottom depth of substructures. Of the surface tests (no boring required), the Ultraseismic test has the broadest application to the determination of the depths of unknown bridge foundations but will provide no information on piles below larger substructure (pilecaps). The Sonic Echo/Impulse Response, Bending Wave, Spectral Analysis of Surface Wave, and Borehold Radar methods all had more specific applications.[3] It is recommended that at this time a Plan of Action and appropriate countermeasures continue to be used as the primary measures to protect bridges with unknown foundations from failure from scour.

K.4 OTHER TEST PROCEDURES

K.4.1 Core Drilling

A simple method used by one State Highway Agency (SHA) to explore unknown foundations is to use a drilling rig to core the bridge deck and to continue down through the pier or abutment footing into the supporting soil or rock under the foundation. This procedure has been used successfully to determine the foundations of some 40 structures and to reclassify the structures as known foundations for purposes of rating them for Item 113, Scour Critical Bridges.

K.4.2 Forensic Engineering

There may be a considerable amount of information in the files of the bridge owner that can be reviewed for information pertaining to the bridge foundations even though as-built plans are no longer available:

- Inspection records may indicate channel bed elevations taken over a period of time. In one state, a concerted effort was made to record channel bed elevations at many bridges immediately after a major flood occurred in 1973. This information now serves as a benchmark for assessing current conditions. If the channel bed is now four or five feet higher than it was in 1973, and the bridge was not damaged in the 1973 flood, this information becomes very useful in assessing the risk posed to the structure by the river.

- Inspectors may have documented exposed foundations in the aftermath of previous floods. While the foundation may no longer be visible, this knowledge of the elevation of the top or bottom of a footing will help the engineer to determine necessary information about the bridge foundation.

- Channel bed under bridges is subject to scour and subsequent infilling of material back into the scour hole. The infill material is likely to be soft fine material that can be easily probed with a reinforcing rod. Careful probing will reveal the elevation of the tops of footings located several feet below the channel bed. Inspections records will often contain basic information about the bridge foundation and whether it is a spread footing or on piles. This information can be used to estimate the footing dimensions within a reasonable degree of accuracy so that an assessment can be made as to whether worst-case scour conditions are likely to exceed the bottom of the footing.

K.5 REFERENCES

1. Transportation Research Board, 1996, "Nondestructive Destructive of Unknown Subsurface bridge Foundations- Results of NCHRP Project 21-5." Research Results Digest No. 213, Transportation Research Board, Washington, D.C.

2. Olson, L.D., F. Jalinoos, and M.F. Aouad, 1995. "Determination of Unknown Subsurface bridge Foundations-Final Report," NCHRP Project 21-5, Transportation Research Board, National Research Council, Washington, D.C.

3. Olson, L.D. and M.F. Aouad, 1998, "NCHRP 21-5 Research Results on Determination of Unknown Bridge Foundation Depths," ASCE Proceedings of International Water Resources Engineering Conference, August 3-7, 1998, Memphis, TN.

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APPENDIX L

SCOUR IN COHESIVE SOILS

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APPENDIX L

Scour In Cohesive Soils

L.1 INTRODUCTION

The maximum depth of local scour at piers in cohesive soils is the same as in non-cohesive soils.[1,2,3] Time is the difference. Maximum scour depth is reached in hours or one runoff event in non-cohesive sand, but may take days and many runoff events in cohesive clays. Local pier scour in cohesive clays may be 1,000 times slower than non-cohesive sand.[1] In addition, by inference, contraction scour and local scour at abutments in cohesive soils do not reach maximum depth as rapidly; but the ultimate scour depth will be the same as for non-cohesive soil.

The equations and methodologies presented in this manual, which predict the maximum scour depth in non-cohesive soil, may, in some circumstance be too conservative. The pier scour equation represents an envelope curve of the deepest scour observed during the various laboratory studies and field data. There is much merit in using a conservative approach, taking into consideration the wide range of soil characteristics, the intricate interactions between soil and water, and the uncertainties inherent in predicting flood flows and their flow patterns through the bridge over its service life. When applied with engineering judgment, this conservative approach is usually reasonable and cost-effective.

On the other hand, there are site conditions and bridges where an alternative method for scour evaluation would be appropriate. Examples include bridges founded on highly scour-resistant cohesive soils where the useful life of the bridge is short in relation to the expected number of scouring floods and rate of scour in cohesive soils, bridges scheduled to be replaced in a couple of years, or bridges on low traffic volume roads which are monitored. Significant savings can be achieved for bridges under these conditions, when the characteristics of the cohesive soils to resist scour are taken into account in the design of the foundation. It is not good engineering judgment to design foundations for scour less than the maximum for bridges in cohesive soils that have a long or undetermined design life, have a very large traffic volume, are not monitored, or serve hospitals or schools. However, it is always good engineering practice to use several methods to determine scour depths and use engineering judgment in the design of bridge foundations.

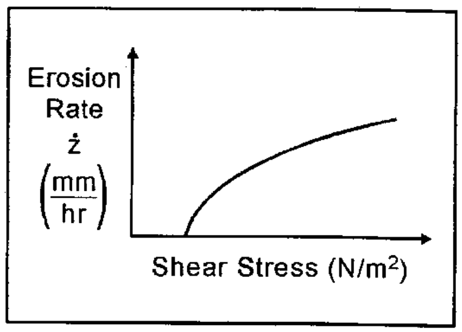
Cohesive soils include silts and clays. According to the unified soil classification system, silts and clays are soils which have more than 50% by weight of particles passing the 0.075mm sieve opening. Silt size particles are between 0.075mm and 0.002mm and clay size particles are smaller than 0.002mm. Cohesive soils are not classified by grain size, but instead by their degree of plasticity which is measured by the Atterberg limits.

Because cohesive soils can scour much slower than non-cohesive soils, it is reasonable to include the scour rate in the calculations. Indeed, while one flood may be sufficient to create the maximum scour depth (z_{max}) in cohesionless soils, the scour depth after many years of flood history at a bridge in an erosion resistant cohesive soil may only be a fraction of z_{max}. The scour rate effect in cohesive soils can be measured by an erosion rate versus shear stress relation. This relation can be used to calculate the scour depth in the case of cohesive soils. This calculated scour depth along with the calculated maximum scour depth, bridge site conditions, type of highway, life cycle of the bridge, traffic volume and comfort level of the DOT can be used in the design of the foundations.

Briaud et al.[1,2] developed a device to measure the scour rate in cohesive soils and equations and methods to use this rate to determine the scour depth at bridges in cohesive soils. The method is called SRICOS for scour rate in cohesive soils. The SRICOS method was developed on the basis of flume tests, numerical testing, and erosion testing of the soil. The device to measure the erosion rate is called EFA (Erosion Function Apparatus). In the following sections the SRICOS method will be described.

L.2 SRICOS METHOD

The first step in the SRICOS method is to develop a plan for testing of the subsurface soils at the bridge site. Representative soils samples are obtained with Shelby tubes and shipped to the laboratory for testing. At the laboratory the EFA is used to determine the erosion rate versus shear stress curve, Figure L.1. The erosion rate dz/dt is defined as the vertical distance scoured per unit of time and is reported in mm/hr. The shear stress, τ, is the shear stress imposed at the water soil interface and is given in N/m^2.



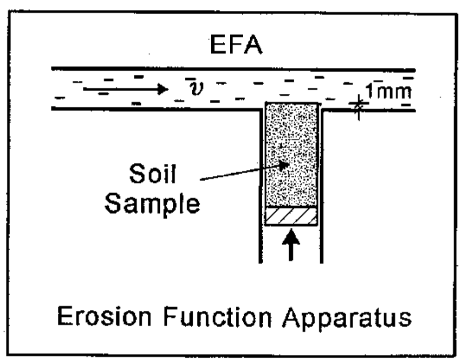


Figure L.1. Erosion rate vs. shear stress and the Erosion Function Apparatus.

The dz/dt versus τ curve is a measure of the erodibility of the soil. Typically the erosion rate dz/dt is zero until the critical shear stress, τ_c, is reached and then dz/dt increases as τ increases. The dz/dt versus τ curve can be measured with the EFA (Erosion Function Apparatus).[1] Once the dz/dt versus τ curve is obtained the method to predict the pier scour depth as a function of time proceeds as follows. First, the maximum shear stress τ_{max} around the bridge pier is calculated:[1]

$$\tau_{max} = 0.0094\ \rho\ V^2 \left[\frac{1}{\log Re} - \frac{1}{10} \right] \quad \text{(L.1)}$$

where:

ρ	=	Density of water
V	=	Mean approach velocity
Re	=	Pier Reynolds number

Second, the initial scour rate dz/dt_i corresponding to τ_{max} is read on the dz/dt vs. τ curve. Third, the maximum depth of scour z_{max} is calculated using the pier scour equations and methods given in Chapter 6.

Note that Briaud[(1)] determined that z_{max} in cohesive soils is very close to that for cohesionless soils. It was found that the maximum depths of scour in clays and in sands were approximately the same in flume experiments. In those same experiments, however, it was found that the scour hole in clay developed to the side and in the back of the pier and not in the front of the pier. This indicates that for scour in clay the front of the pier may not be the best place to install monitoring equipment.

It is then possible to make scour predictions by applying a detailed velocity (shear stress) history over the design life of the bridge and summing the erosion rates for the cumulative amount of time that the shear stress exceeds the critical shear stress. This requires the use of a computer program which can also consider the case of a layered soil system.[(2)] The limitation of this method is that it is for circular bridge piers and for water depth over pier diameter ratios larger than 2. Existing correction factors are recommended for other cases.

To apply this approach to contraction scour, the computed hydraulic shear stress would be used directly rather than Equation L.1, which is specific to circular piers. For abutment scour, a relationship would need to be developed to determine local shear stress, or a detailed 2-dimensional model would need to be used to compute shear stresses in the vicinity of the abutment toe.

L.3 REFERENCES

1. Briaud, J.-L., F.C.K. Ting, H.C. Chen, R. Gudavaiii, S. Perugu, G. Wei, 1999a, "SRICOS: Prediction of Scour Rate in Cohesive Soils at Bridge Piers," ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 125, No. 4, pp 237-246, Reston, VA.

2. Briaud, J.-L., F.C.K. Ting, H.C. Chen, R. Gudavaiii, K. Kwak, B. Philogene, S.W. Han., S. Perugu, G. Wei, P. Nurtjahyo, Y. Cao, Y. Li, 1999b, "SRICOS Prediction of Scour Rate at Bridge Piers," Report 2937-F, Texas Depart. of Transportation, Texas A&M University, Civil Engineering, College Station, TX 77843-3136.

3. Richardson, E.V., L.J. Harrison, and S.R. Davis, 1991, “Evaluating Scour at Bridges,” HEC-18, Report FHWA-IP-90-017, Washington, D.C.

APPENDIX M

SCOUR COMPETENCE OF ROCK

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APPENDIX M

Scour Competence of Rock

M.1 INTRODUCTION

The equations and methods given in this manual are for determining scour depths for the design of bridge foundations in granular soils (silts, sands, gravels, cobbles, and boulders). In Chapter 2 recommendations are given for the design of bridge foundations on rock highly resistant to scour. **The problem is determining if rock is resistance to scour.** There are examples in the literature of bridge foundation failure form scour in what was supposed to be rock**.** An example is the failure of the I-90 bridge over Schoharie Creek in upstate New York (see Chapter 11, Section 11.4). The rock foundation material was massive, extensive, and very hard with a blow count on the order of 80 to 100. However, when subjected to water flow in a flume test it started to erode at a velocity of 1.22 m/s (4 ft/s) and would erode rapidly at a velocity of 2.44 m/s (8 ft/s).[1,2]

The determination if the bridge foundations are founded in scour resistance rock and the design of foundations in rock require the expertise of geologist and geotechnical engineers. In addition to standard geologic and geotechnical tests, core or block samples can be taken and subjected to flume studies. The Erosion Function Apparatus (EFA) described in Appendix L or a simply constructed or available flume can be used to determine the scourability of the rock material. In the following sections four recommendations are given for determining if rock foundations are scour resistance. However, additional research is needed in this area. The four recommendations are:

- Geologic, geomorphologic, and geotechnical analyses
- July, 1991 memorandum from the FHWA titled "Scourability of Rock Formations, (see Attachment 1, Appendix M)
- Flume tests to determine the resistance of rock to scour
- Erodibility Index procedure

M.2 GEOLOGIC, GEOMORPHOLOGY, STREAM STABILITY AND GEOTECHNICAL ANALYSIS

The geology, geomorphology and stream stability of the bridge crossing, and geotechnical analysis of the foundation material are extremely important. Coring of the site must be extensive to determine the location and extent (depth and area) of the rock. The cores to be subjected to the standard field classification and soil mechanics tests. In addition, where the scourability of the rock type is unknown, erosion tests as described later should be made. The geologic formation on which the bridge foundations are to be constructed needs to be determined and mapped (depth, areal extent and massiveness). The scour resistance of the geologic formation needs to be known or determined. The geomorphology of the site needs to be determined and related to the erodibility of the foundation material (alluvial fan, karst topography, desert, mountain or plain stream, etc). The long-term stability of the stream should be estimated.

Some questions to be answered are:

- Is the competent rock only a relatively thin layer 0.6 to 1 m (2 to 3 ft) that can be undermined?
- Is there the potential for a headcut or nickpint from downstream to undermine the rock?
- What is the geologic formation for the foundation (granite, sandstone, glacial till, etc)?
- What is the scour experience of bridges in the area or in similar geologic formations?
- Is the foundation material subjected to freezing and thawing?
- Is the foundation material susceptible to leaching by flowing water (limestone)?
- What is the planform of the stream at the bridge crossing (meandering, braided or straight)?
- Is the stream aggaiding or degrading?
- Are the foundation material subject to abrasion by the sediment discharge of the stream? If so, how resistance to abrasion is the rock material?

M.3 FLUME TESTS

Samples (standard core or other square or round samples) of the foundation material that is thought to be resistant to the erosion action of water can be tested in flumes. Any flume that is used for hydraulic research can be used if it has a sufficiently large range of velocities at a depth of 0.15 m (0.5 ft) or more. At modest cost a flume can be built to determine the resistance of a rock sample to erosion. The Erosion Function Apparatus (EFA) used to determine the erodibility of cohesive soils can and has been used to determine the erodibility of rock samples.[3,4] The EFA determines the scour rate in mm/hr vs. shear stress in N/m^2 or velocity in m/s. The apparatus and method are described in Appendix L. The samples should be subjected to velocities as large as are to be expected at the bridge crossing and placed in the flume flush with the floor or only slightly projecting into the flow. Projections of 1mm (.03 in) to 3 or 4 mm (0.16 in) are acceptable. If standard cores are not taken a square approximately 0.3 m (1 ft) should be sufficient to test.

Flume tests can determine if the rock material will not erode for the expected velocity or shear stress, or if the material will erode. In some cases a time rate of erosion (mm/hr, inches/hr) can be obtained. In the latter case, methods proposed in Appendix L can be used to determine if the maximum calculated scour can be reduced.

In obtaining samples of the foundation material care must be exercised to not destroy the integrity of the foundation material at the bridge site. Often, with the help of a geologists, samples can be taken and tested of similar material from another location.

M.3.1 Examples of Flume Erosion Tests

Ice Compacted Glacial Till Erodibility tests of the ice compacted glacial till that was the foundation material for the I-90 bridge over Schoharie Bridge were made in flume tests at Cornell University.[2,5,6] Although the foundation material was extremely dense, difficult to penetrate with piles or to excavate, erosion would start at a velocity of 1.22 m/s (4 ft/s) and would be large at 2.4 m/s (8.0 ft/s).

Caliche Soil Layers Erodibility test were made of caliche layers that are found in the bed of dry arroyos in the desert soils of Arizona. Caliche soil layers are soils composed of silt, sand, gravel or cobbles cemented by secondary calcium carbonate precipitate. The layers may be a few centimeter (inches) to several meters (ft) thick and erodibility may range form easily to very hard. Tests were made using the EFA on a 3 inch (0.76 m) core [7] and using a specially constructed flume on three 1 ft. (0.3 m) roughly cubic samples.[8] In the EFA tests the core was subjected to velocities ranging from O.21 m/s to 4.7 m/s (0.7 to 15.4 ft/s). Both the bottom and top of the 3 inch core was tested. Erosion rates for the top of the core ranged from 0.15 mm/h (0.006 inch/hr) at a velocity of 0.21 m/s (0.70 ft/s) to 219.8 mm/hr (8.7 inch/hr) at 1.46 m/s (4.79 ft/s). The erosion rates for the bottom layer ranged from 0 mm/hr at 0.53 m/s (1.73 ft/s) to 22.05 mm/hr (0.87 inch/hr) at 2.43 m/s (7.97 ft/s). The core as tested was approximately 70 mm (2.76 inch) in length. Similar results were obtained using a specially constructed flume on the three 1 ft. (0.3 m) roughly cubic samples. For the sample that had the smallest erosion rate, the rate ranged from 0.60 mm/hr (0.24 inch/hr) at 0.75 m/s (2.46 ft/s) to 2.10 mm/hr (0.83 inch/hr) at 3.14 m/s (10.30 ft/s). The sample with the largest erosion rate the rate ranged from 4.12 mm/hr (0.16 inch/hr) at a velocity of 0.64 m/s (2.10 ft/s) to 177.6 mm/hr (6.99 inch/hr) at a velocity of 2.96 m/s (9.71 ft/s) . The results of the tests showed the variability in the erodibility of the caliche layers, and the comparability and usefulness of the two testing methods.

M.4 ERODIBILITY INDEX METHOD

Annandale[9,10] developed an Erodibility Index, which is identical to Kirsten's Excavatability Index,[11] to quantify the relative ability of non-uniform earth material to resist erosion. He proposed a relation between the Erodibility Index and stream power for use in determining pier scour. Measurements of pier scour collected at FHWA's Turner-Fairbank Highway Research Center Hydraulics Laboratory were used to study a relationship between scour depth and stream power in order to develop a practical application of the Erodibility Index to scour prediction.[12] The Erodibility Index to quantify the ability of rock material to resist erosion and the development of a relation between the index and stream power for contraction and local scour at piers and abutments appears feasible, but needs further research.

The Index is defined as:

$$K = M_s \, . \, K_b \, . \, K_d \, . \, J_s$$

where:

K	=	Erodibility Index
M_s	=	Intact mass strength number
K_b	=	Block size number
K_d	=	Discontinuity or inter-particle bond shear strength number
J_s	=	Orientation and shape number

The values of these parameters are determined by making use of field and/or laboratory observations, and tables published in Annandale,[9] Kirsten,[11] and the National Engineering Handbook.[13] The mass strength number M_s represents the strength on an intact representative sample of the earth material without regard to geologic heterogeneity within the mass. K_b is a function of the Rock Quality Designation (RQD) in the case of rock and is a function of an effective particle diameter in the case of granular material. K_d represents the shear strength at the interface of failure planes, such as fissures or slickensides in clay, or joints and fractures in rock. This value can be estimated from the properties of joint and fracture planes in the case of rock, or from tri-axial tests in the case of granular materials. The orientation and shape number is a function of the dip and strike of rock, and of the relative shape of individual rock blocks. J_s accounts for the structure of the ground with respect to stream flow. It is a complex function that considers orientation and shape of individual blocks with respect to stream flow. Additional description of the variables is given in Annandale,[9] and Annandale and Kirsten.[14]

For this application stream power is defines as:

$$P = \gamma q S$$

where:

P	=	Stream power, kg/s, (lb/s)
γ	=	Unit weight of water, kg/m^3, (lb/ft$^{3)}$
q	=	Unite discharge of water, m^3/s, (ft^3/s)
S	=	Slope of the energy grade line, m/m (ft/ft)

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ATTACHMENT 1

FHWA 1991 MEMORANDUM "SCOURABILITY OF ROCK FORMATIONS"

Federal Highway Administration
Date: July 19, 1991
Subject: Scourability of Rock Formations

From: Chief, Bridge Division
Office of Engineering

To: Regional Federal Highway Administrators
Federal Lands Highway Program Administrator

Usually rock is regarded as the best bearing material for structural foundations, however, there are conditions, such as sinkholes in limestone, weathering and scourability which can present problems. Bridge foundation failures have occurred due to scour of rock or rock-like materials. This memorandum presents interim guidance on empirical methods and testing procedures to assess rock scourability until results of ongoing research permit more accurate evaluation procedures. These empirical methods are commonly used by geotechnical engineers and geologists to determine rock mass. engineering properties such as, allowable bearing pressures for shallow and deep foundations. Footing elevations on rock should be conservatively selected based on experience and the indirect qualitative interpretation of the methods discussed below. While safety of the traveling public is the primary design consideration, bridge designers should recognize that scour assumptions have a significant impact on the cost and constructibility of foundations and overly conservative assumptions should be avoided.

Academic geologic studies have shown that even the hardest of rocks can scour when exposed to moving water. However, the time for a finite depth of scour, is not possible to predict at this time. Empirical methods can be used to approximate rock scourability within the lifetime of a structure. Several properties contribute to the quality, bearing capacity and soundness of rock. Hence, no single -index property will correctly assess the potential for scour. Designers are encouraged to utilize a combination of the following methods to assess rock scourability until a more quantitative procedure becomes available.

1. <u>Subsurface Investigation</u>

The objective of a subsurface investigation for shallow foundations on rock should permit an identification of rock type, determination of discontinuity frequency and recovery of high quality rock core samples for testing and evaluation. The number of drill holes par substructure unit should be based on the footing size, structure criticality and variability of subsurface conditions. A minimum of one boring per substructure unit and a 3.3 meter (10 foot) minimum core length below the bottom of footing are recommended.

Rock core sample quality is greatly influence by drilling equipment and technique. Poor drilling techniques will penalize rock quality) assessments by lowering core recovery and rock quality designation (RQD). Rock cores should be obtained with NX diameter size core barrels 5.4 centimeters, (2 1/8 inch) or larger. Double or triple tube core barrels should be used for all structural foundation projects.

2. Geologic Formation/Discontinuities

Rock type and frequency of discontinuities have a significant impact on engineering properties. The three classes of rock based on geologic origin are igneous, sedimentary and metamorphic. Igneous rocks are formed by solidification of molten material from deep beneath the earth's surface. They are generally uniform in structure and lack stratification and cleavage planes. Examples of igneous rock are granite, diorite, gabbro, basalt and diabase.

Sedimentary rocks are products of disintegration and decomposition by weathering of preexisting rock. These rocks are formed by mechanical cementation, chemical precipitant and pressure. Examples of sedimentary rock are sandstone, limestone, dolomite, shale and chert. Some common - features of sedimentary rock are rounded grains, stratifications, inclination of bedding planes and abrupt color changes between layers.

Metamorphic rock is formed from igneous or sedimentary rocks which have been altered physically or chemically by intense heat and pressure. Examples are quartzite, marble. slate and schist. Some features include the ease with which parallel layers break into slabs. In general, harder and more sound rock is less susceptible to scour.

If rocks were free of defects, then the allowable bearing pressure could be taken conservatively as the average compression strength of unconfined rock core samples. However, rock masses are seldom free of imperfections and fractures which have a significant influence on rock behavior. The spacing of discontinuities is an indication of overall rock quality. Spacing is measured-d as the perpendicular distance between parallel discontinuities. Measurement is easily accomplished for rock outcrops, but is difficult from vertical drill holes. Drill cores with one fracture or less per foot would indicate a good quality rock mass. High fracture frequency (five or six fractures per foot) would indicate a poorer quality rock which would be considerably weaker and more scourable.

3. Rock Quality Designation (RQD)

The RQD value is a modified computation of percent rock core recovery that reflects the relative frequency of discontinuities, the compressibility of the rock mass & may say indirectly be utilized as a measure of scourability. The RQD is determined by measuring and summing all the pieces of sound rock 10.2 centimeters (4 inches) and longer in length in a core run, and dividing this by the total core run length. The RQD should be computed using NX diameter cores or larger and on samples from double tube core barrels. Figure I provides an example of RQD computation and a relationship between RQD and rock quality. Table I provides a relationship between RQD, rock type and allowable bearing pressures. Scourability potential will increase as the quality of the rock becomes poorer. Rock with an RQD value less than 50 percent should be assumed to be soil-like with regard to scour potential.

4. Unconfined Compressive Strength (qu, ASTM D29361)

The primary intact rock property of interest for foundation design is unconfined compressive strength. Although it is known that strength of jointed rocks is generally less than individual units of the rock mass, the unconfined compressive strength provides an upper limit of the rock mass bearing capacity and an index value for rock classification. In general, samples with unconfined strengths below 1724 Kpa (250 psi) are not considered to behave as rock. As unconfined compressive strength increases, bearing capacity generally increases and scourability decreases. There is only a generalized correlation between unconfined compression strength and scourability.

5. Slake Durability Index (SDI, International Society of Rock Mechanics)

The SDI is a test used on metamorphic and sedimentary rocks such as slate and shale. An SDI value of less than 90 indicates a poor rock quality. The lower value of SDI, the more scourable and less durable the rock.

6. Soundness (AASHTO T104)

The laboratory test for soundness of rock uses a soaking procedure in magnesium or sodium sulfate solution. Generally, the less sound the rock, the more scourable it will be. Threshold loss rates of 12 (sodium) and 18 (magnesium) percent can be used as an indirect measure of scour potential.

7. Abrasion (AASHTO T96)

The Los Angeles Abrasion Test is an empirical test to assess abrasion of aggregates. In general, the less a material abrades during this test, the less it will scour. Materials with loss percentages greater than 40 should be considered scourable.

The above procedures can be effectively utilized to produce a rational screening process to assess rock scourability until more quantitative, methods become available.

Stanley Gordon

ROCK QUALITY DESIGNATION (RQD) EXAMPLE

An example is given below from a core run of 152.4 cm (60 inches). For this particular case the total core recovery is 127 cm (50 inches) yielding a core recovery of 83 percent. On the-modified basis, only 99 cm (34 inches) are counted and the RQD is 65 percent.

CORE RECOVERY, in	MODIFIED CORE RECOVERY, in
10	10
2	
2	
3	
4	4
5	5
3	
4	4
6	6
4	4
2	
5	5
50	39

% Core Recovery = 50/60 - 83%; RQD= 39/60 = 65%

A general description of the rock quality can be made from the RQD value.

RQD (ROCK QUALITY DESIGNATION)	DESCRIPTION OF ROCK QUALITY
0- 25	very poor
25-50	poor
50-75	fair
75- 90	good
90 -100	excellent

FIGURE 1

TABLE I

RECOMMENDED ALLOWABLE BEARING PRESSURE FOR FOOTINGS ON ROCK

MATERIAL	ALLOWABLE CONTACT PRESSURE (Kpa)
Such igneous and sedimentary rock as crystalline bedrock, including granite, diorite, gneiss, traprock; and hard limestone, and dolomite, in sound condition:	
RQD = 75 to 100 percent	11491 (120 tsf)
RQD = 50 to 75 percent	6224 (65 tsf)
RQD = 25 to 50 percent	2873 (30 tfs)
RQD - 0 to 25 percent	958 (10 tfs)
Such metamorphic rock as foliated rocks, such as schist or slate; and bedded limestone, in sound condition:	
RQD > 50 percent	3830 (40 tfs)
RQD < 50 percent	958 (10 tfs)
Sedimentary rocks, including hard shales and sandstones, in sound condition:	
RQD > 50 percent	2394 (25 tfs)
RQD < 50 percent	958 (10 tfs)
Soft or broken bedrock (excluding shale), and soft limestone:	
ROD > 50 percent	1149 (12 tfs)
ROD < 50 percent	766 (8 tfs)
Soft shale	383 (4 tfs)

ATTACHMENT WR-1C

Wash Data

ATTACHMENT WR-1C
Ivanpah Solar Electric Generating System Project
Wash Data

Ivanpah 1	Date	Time	Photo No. & Direction	Wash Class	Width- Bank to Bank (ft.)	Depth - Top of Bank (ft.)	Comments
Ivanpah 2	**Date**	**Time**	**Photo No. & Direction**	**Wash Class**	**Width- Bank to Bank (ft.)**	**Depth - Top of Bank (ft.)**	**Comments**
	08/13/09	14:35	100-8246-8247 NE	2	35.4	1.91	Class 2-3A
	08/13/09	14:52	100-8248-8252	2	46.5	1.9	Class 2-3B
	08/13/09	14:58	100-8254-8256	2	36.1	1.25	Class 2-3C
	08/13/09	15:06	100-8257-8262	3	64.5	2.41	Class 3-4A
	08/13/09	15:13	100-8263-8268	3	57.1	2.83	Class 3-4B
	08/13/09	15:17	100-8269-8272	3	56.3	2.66	Class 3-4C
	08/13/09	15:44	100-8273-8277 E	1	59.5	1.08	Class 1-3A
	08/13/09	15:55	100-8278-8282 E	1	54.1	0.75	Class 1-3B
	08/13/09	15:59	100-8283-8286 E	1	56.5	0.75	Class 1-3C
	08/13/09	16:37	100-8287-8290 W	3	28.8	0.33	Class 3-5A
	08/13/09	16:49	100-8291-8294 NW	3	38.4	0.75	Class 3-5B
	08/13/09	16:54	100-8295-8297 NW	3	30.8	1.41	Class 3-5C
	08/13/09	17:07	100-8298-8300 NW	2	38.9	0.58	Class 2-4A
	08/13/09	17:13	100-8301-8304 SE	2	35.1	0.75	Class 2-4B
	08/13/09	17:20	100-8305-8307 SE	2	21.6	0.75	Class 2-4C
	08/13/09	17:26	100-8308-8210 SE	3	21.2	1.5	Class 3-7
	08/13/09	17:31	100-8311-8313 NW	3	18.9	1.08	Class 3-6B
	08/13/09	17:35	100-8314-8317 SE	3	21.3	1.41	Class 3-6C
Ivanpah 3	**Date**	**Time**	**Photo No. & Direction**	**Wash Class**	**Width- Bank to Bank (ft.)**	**Depth - Top of Bank (ft.)**	**Comments**
	08/13/09	8:16	100-8175 SE	3	34.3	1.4	Class 3-1A
	08/13/09	8:30	100-8178 SE	3	37.9	1.4	Class 3-1B
	08/13/09	8:36	100-8179 SE	3	42.2	2	Class 3-1C
	08/13/09	9:01	100-8180 SE	3	23.4	0.4	Class 3-2A
	08/13/09	9:07	100-8184 SE	3	25.2	0.5	Class 3-2B
	08/13/09	9:13	100-8185 SE	3	33.1	0.25	Class 3-2C
	08/13/09	9:33	100-8186-8187 SE	2	37.3	1.83	Class 2-1A
	08/13/09	9:43	100-8188 SE	2	39.7	1.58	Class 2-1B
	08/13/09	9:48	100-8189 SE	2	36.2	1.16	Class 2-1C
	08/13/09	9:53	100-8190 SE	3	29.6	1.08	Class 3-3A
	08/13/09	9:56	100-8191-8192 SE	3	31.4	1.16	Class 3-3B
	08/13/09	10:04	100-8193-8195 SE	3	41.1	1.5	Class 3-3C
	08/13/09	11:11	100-8196-8199	1	87.2	1.41	Class 1-2A
	08/13/09	11:18	100-8200-8205	1	81.1	1.58	Class 1-2B
	08/13/09	11:25	100-8206-8210	1	77.4	1.25	Class 1-2C
	08/13/09	11:37	100-8211-8215	1	58.2	4.41	Class 1-1A
	08/13/09	11:43	100-8216-8220	1	42.4	4.33	Class 1-1B
	08/13/09	11:55	100-8224-8229	1	47.1	5.25	Class 1-1C
	08/13/09	12:08	100-8230-8234	2	42.2	2	Class 2-2A
	08/13/09	12:14	100-8235-8239	2	46.1	2.25	Class 2-2B
	08/13/09	12:19	100-8240-8245	2	46.8	1.83	Class 2-2C

Comments:

Worker Health and Safety

I. Introduction

A. Name: Steve De Young

B. Qualifications: Mr. De Young's qualifications are as noted in his resume contained in Appendix A.

C. Prior Filings: In addition to the statements herein, this testimony includes by reference the following documents submitted in this proceeding:

- Application for Certification, Volume 1 & Volume 2 [Exhibit 1]
- Comments to the PSA [Exhibit 57]
- Applicant's Response to CEC Staff Requests, Supplemental Data Response Set 2C, dated May 19, 2009 Responses to Data Requests WS-1 [Exhibit 40].

To the best of my knowledge, all of the facts contained in this Section of the Applicant's testimony (including all referenced documents) are true and correct. To the extent this testimony contains opinions, such opinions are my own based upon my professional judgment. I make these statements, and render these opinions freely and under oath for the purpose of constituting sworn testimony in this proceeding.

II. Summary of Testimony

During this project, the workers will be exposed to construction safety and operation hazards. A hazard analysis has been prepared to evaluate the project hazards and control measures. The analysis identifies the hazards anticipated during construction and operation and indicates which safety programs should be developed and implemented to mitigate and appropriately manage those hazards.

Overview of Hazards and Related Programs and Training

Programs are overall plans that set forth the method or methods that will be followed to achieve particular health and safety objectives. For example, the Fire Protection and Prevention Program will describe what has to be done to protect against and prevent fires. This will include equipment required, such as alarm systems and firefighting equipment, and procedures to protect against fires. The Emergency Action Program/Plan will describe escape procedures, rescue and medical procedures, alarm and communication systems, and response procedures for very hazardous materials that can migrate. The programs or plans are contained in written documents that are usually kept at specific locations within the facility.

Each program or plan will contain training requirements that are translated into detailed training courses. These courses are taught to plant construction and operating personnel, as needed. For example, all plant operating personnel will receive training in escape procedures under the Emergency Action Program/Plan, but only those working with flammables will receive training under the Fire Protection and Prevention Program.

Health and Safety Programs

To protect the safety and health of workers during the construction and operation of Ivanpah SEGS, health and safety programs designed to mitigate hazards and comply with applicable regulations will be implemented. Periodic audits will be performed by qualified individuals to determine whether proper work practices are being used to mitigate hazardous conditions and to evaluate regulatory compliance.

Operations Health and Safety Program

Upon completion of construction and commencement of operations at Ivanpah SEGS, the construction safety and health program will transition into an operations-oriented program reflecting the hazards and controls necessary during operation.

Safety Training Programs

To ensure that employees recognize and understand how to protect themselves from potential hazards during this project, comprehensive training programs for construction and operation will be implemented. Each of the safety procedures developed to control and mitigate potential site hazards will require some form of training. Training will be delivered in various ways, depending on the requirements of Cal-OSHA standards, the complexity of the topic, the characteristics of the workforce, and the degree of risk associated with each of the identified hazards.

Emergency Response

Because of the highly remote and rural area of Ivanpah SEGS, services are limited and spread out. San Bernardino County Firefighters receive specialized training to address emergency responses to industrial hazards. The response time to the project site, with full resources capabilities, would be 3 to 4 hours. There are roughly 150 members (10 Registered Environmental Health Specialists and the rest firefighters) and the organization is a full Level A response team, capable of handling all types of Chemical, Biological, Radiological, and Nuclear responses. Hazardous materials service is provided out of the County station in Fontana, Station #78.

Law enforcement is provided by the San Bernardino County Sheriff. The closest county sheriff location to the project site would be the Baker Resident Post. Two deputies staff this post and there is at least one officer available to respond to calls 24 hours a day. Response time would be the drive time from the City of Baker to the Project site. (approximately 45 minutes).

Hospitals

Ambulance service is provided by Baker Ambulance Medical Service, Station #53.

The closest hospitals with an emergency room are Saint Rose in Henderson, CA and University Medical Center, Las Vegas (UMCLV). Saint Rose is approximately 40 miles from the proposed project site. Specialty services at the hospital include intensive care unit, emergency/trauma, labor and delivery, cardiac care, orthopedics, surgery, and transplant. University Medical Center is approx. 50 miles distant and roughly 55 minutes drive time. This is a fully staffed teaching hospital, serving the medical needs of southern Nevada and parts of California, and Arizona.

III. Proposed Licensing Conditions

The FSA/DEIS for the project filed by the CEC and BLM recommends that 6 Conditions of Certification be adopted to address worker safety and fire protection issues: WORKER SAFETY-1 through WORKER SAFETY-6. We offer the following suggested changes:

Proposed Revision to WORKER SAFETY-1 through 6

As a general matter, the Applicant is opposed to conditions that require separate approvals of post-certification compliance activities by both BLM and the CPM because they are simply unworkable. If the approval is sequential, it will result in doubling the required approval time for everything. If the approval is concurrent, approvals may be potentially conflicting. As a general rule, consistent with current Commission practice, we have identified the Commission's CPM as the authority to review and approve post-certification compliance submissions or actions of the Applicant. It is also imperative that specific timeframes for approval be included in the Conditions so that the project will not be unnecessarily delayed. See the applicant's pre-hearing conference statement for a more-detailed explanation.

WORKER SAFETY-1 The project owner shall submit to BLM's Authorized Officer and the Compliance Project Manager (CPM) a copy of the Project Construction Safety and Health Program containing the following:

- A Construction Personal Protective Equipment Program;
- A Construction Exposure Monitoring Program;
- A Construction Injury and Illness Prevention Program;
- A Construction Emergency Action Plan; and
- A Construction Fire Prevention Plan.

The Personal Protective Equipment Program, the Exposure Monitoring Program, and the Injury and Illness Prevention Program shall be submitted to ~~BLM's Authorized Officer and~~ the CPM for review and approval concerning compliance of the program with all applicable Safety Orders. The CPM will provide a decision on the program within thirty (30) days of submission by the project owner. The Construction Emergency Action Plan and the Fire Prevention Plan shall be submitted to the San Bernardino County Fire Department for review and comment. The CPM shall review any comments received by the San Bernardino County Fire Department if those comments are timely submitted. The CPM shall provide a decision on the Construction Emergency Action Plan and the Fire Prevention Plan within thirty (30) days of submission by the project owner. ~~prior to submittal to the BLM's Authorized Officer and CPM for approval.~~

Verification: At least thirty (30) days prior to the start of construction, the project owner shall submit to ~~BLM's Authorized Officer and~~ the CPM for review and approval a copy of the Project Construction Safety and Health Program. The project owner shall provide a copy of a letter ~~to the BLM's Authorized Officer and CPM~~ from the San

Bernardino County Fire Department, if any is received, stating the Fire Department's comments on the Construction Fire Prevention Plan and Emergency Action Plan.

WORKER SAFETY-2 The project owner shall submit to ~~BLM's Authorized Officer and~~ the CPM a copy of the Project Operations and Maintenance Safety and Health Program containing the following:

- An Operation Injury and Illness Prevention Plan;
- An Emergency Action Plan;
- Hazardous Materials Management Program;
- Fire Prevention Program (8 CCR § 3221); and;
- Personal Protective Equipment Program (8 CCR §§ 3401-3411).

The Operation Injury and Illness Prevention Plan, Emergency Action Plan, and Personal Protective Equipment Program shall be submitted to ~~BLM's Authorized Officer and~~ the CPM for review and approval concerning compliance of the program with all applicable Safety Orders. The CPM shall provide a decision within thirty (30) days of submission. The Operation Fire Prevention Plan and the Emergency Action Plan shall also be submitted to the San Bernardino County Fire Department for review and comment.

Verification: At least thirty (30) days prior to the start of ~~first-fire or~~ commissioning, the project owner shall submit to ~~BLM's Authorized Officer and~~ the CPM for approval a copy of the Project Operations and Maintenance Safety and Health Program. The project owner shall provide a copy of a letter to ~~BLM's Authorized Officer and~~ the CPM from the San Bernardino County Fire Department stating the Fire Department's comments on the Operations Fire Prevention Plan and Emergency Action Plan. The CPM shall provide a decision on the Program within fifteen (15) days of submission.

WORKER SAFETY-3 The project owner shall provide a site Construction Safety Supervisor (CSS) who, by way of training and/or experience, is knowledgeable of power plant construction activities and relevant laws, ordinances, regulations, and standards, is capable of identifying workplace hazards relating to the construction activities, and has authority to take appropriate action to assure compliance and mitigate hazards. The CSS shall:

- Have over-all authority for coordination and implementation of all occupational safety and health practices, policies, and programs;
- Assure that the safety program for the project complies with Cal/OSHA and federal regulations related to power plant projects;
- Assure that all construction and commissioning workers and supervisors receive adequate safety training;

- Complete accident and safety-related incident investigations, emergency response reports for injuries, and inform ~~BLM's Authorized Officer and~~ the CPM of safety-related incidents; and
- Assure that all the plans identified in **WORKER SAFETY-1 and -2** are implemented.

Verification: At least thirty (30) days prior to the start of site mobilization, the project owner shall submit to ~~BLM's authorized officer~~ and the CPM the name and contact information for the Construction Safety Supervisor (CSS). The contact information of any replacement (CSS) shall be submitted to ~~BLM's Authorized Officer and~~ the CPM within ~~one~~ three business days.

The CSS shall submit in the Monthly Compliance Report a monthly safety inspection report to include:

- Record of all employees trained for that month (all records shall be kept on site for the duration of the project);
- Summary report of safety management actions and safety-related incidents that occurred during the month;
- Report of any continuing or unresolved situations and incidents that may pose danger to life or health; and
- Report of accidents and injuries that occurred during the month.

WORKER SAFETY-4 The project owner shall make payments to the Chief Building Official (CBO) for the services of a Safety Monitor based upon a reasonable fee schedule to be negotiated between the project owner and the CBO. Those services shall be in addition to other work performed by the CBO. The Safety Monitor shall be selected by and report directly to the CBO, and will be responsible for verifying that the Construction Safety Supervisor, as required in **WORKER SAFETY-3**, implements ~~all~~ ~~appropriate~~ applicable Cal/OSHA and Commission safety requirements. The Safety Monitor shall conduct on-site (including linear facilities) safety inspections at intervals necessary to fulfill those responsibilities.

Verification: At least thirty (30) days prior to the start of construction, the project owner shall provide proof of its agreement to fund the Safety Monitor services to ~~BLM's Authorized Officer and~~ the CPM ~~for review and approval~~.

WORKER SAFETY-5 The project owner shall ensure that a portable automatic external defibrillator (AED) is located on site during construction and operations and shall implement a program to ensure that workers are properly trained in its use and that the equipment is properly maintained and functioning at all times. During construction and commissioning, the following persons shall be trained in its use and shall be on-site whenever the workers that they supervise are on-site: the Construction Project Manager or delegate, the Construction Safety Supervisor or delegate, and all shift foremen. During operations, all power plant employees shall be trained in its use. The training program shall be submitted to ~~BLM's Authorized Officer and~~ the CPM ~~for review and approval~~.

Verification: At least thirty (30) days prior to the start of site mobilization the project owner shall submit to ~~BLM's Authorized Officer and~~ the CPM proof that a portable AED exists on site and a copy of the training and maintenance program ~~for review and approval~~.

WORKER SAFETY-6 The project owner shall prepare and implement a Best Management Practices (BMPs) for the storage and application of herbicides used to control weeds beneath and around the solar array. These plans shall be submitted to ~~BLM's Authorized Officer and~~ the CPM for review and approval.

Verification: At least thirty (30) days prior to the start of site mobilization, the project owner shall submit to ~~BLM's Authorized Officer and~~ the CPM for review and approval a copy of the Best Management Practices (BMPs) for the storage and application of herbicides. The CPM shall provide a decision on the BMPs within fifteen (15) days of submission.

IV. Correlation to FSA and Hearing Topics:

- Worker Safety and Fire Protection

Alternatives

I. Introduction

A. Name: John Carrier, Steve De Young, Gary Rubenstein, Steve Hill, and Tom Priestley

B. Qualifications: The panel's qualifications are as noted in his resume contained in Appendix A.

C. Prior Filings: In addition to the statements herein, this testimony includes by reference the following documents submitted in this proceeding:

- Application for Certification, Volume 1 [Exhibit 1]
- Comments to the PSA [Exhibit 57]
- Applicant's Response to CEC Staff Requests, Data Response Set 2A, dated June 10, 2008, Responses to Data Requests 121 through 123 [Exhibit 20].
- Applicant's Response to CEC Staff Requests, Data Response Set 2B, dated July 22, 2008, Responses to Data Requests 121 through 123 [Exhibit 21].

To the best of our knowledge, all of the facts contained in this testimony (including all referenced documents) are true and correct. To the extent this testimony contains opinions, such opinions are our own. We make these statements, and render these opinions freely and under oath for the purpose of constituting sworn testimony in this proceeding.

II. Summary of Testimony

A range of reasonable alternatives that could feasibly attain most of the basic objectives of the Ivanpah SEGS were identified and evaluated in the Alternatives section (Section 6 of the AFC) including a conservation alternative, a smaller plant alternative, the "No Project" alternative (that is, not developing a new solar power generation facility), alternative site locations for constructing and operating Ivanpah SEGS, alternative thermal configurations to the solar arrangement proposed for Ivanpah SEGS, and alternative power generation technologies. Alternatives to the linear facilities (electric, natural gas, and water) were not considered because the distances are relatively short and direct; therefore, alternative routes would not avoid or substantially reduce environmental impacts compared to the project.

The AFC considered ten alternative site locations for a 400-MW solar project. Of the 10 alternative sites considered, 6 locations were not carried forward for further analysis, and 4 locations were carried forward for full examination. They were Ivanpah Site A, Ivanpah Site C, Broadwell Lake and Siberia.

Based on the analyses presented in Section 6 of the AFC, the No Project Alternative would have the least potential for significant impacts. However, the No Project Alternative would not meet the basic project objectives, would not satisfy the purpose and need, and would not provide the benefits of the project. It also fails to implement the multiple use goals of the Federal Land

Policy and Management Act and the renewable energy goals of both the Federal Land Policy and Management Act and the California Desert Conservation Area Resource Management Plan. Of the alternatives considered that are potentially capable of meeting the project objectives, the Ivanpah SEGS site, incorporating the mitigation measures proposed in the AFC, would be expected to result in the least short-term and long-term environmental effects.

III. Proposed Licensing Conditions

There are no Conditions of Certification related to Alternatives.

IV. Correlation to FSA and Hearing Topics:

- Alternatives

APPENDIX A

Resumes and Declaration

Resumes of Key Staff

John Carrier, J.D., Project Manager, CH2M HILL

Mark Bastasch, P.E., I.N.C.E., Noise Task Lead, CH2M HILL

Loren Bloomberg, P.E., Traffic and Transportation Task Lead, CH2M HILL

John Cleckler, Senior Wildlife Biologist, CH2M HILL

Mark Cochran, Senior Biologist, CH2M HILL

Steven De Young, Vice President, Environmental, Health and Safety, BrightSource Energy, Inc.

Timothy Durbin, P.E., Managing Engineer, West Yost Associates

Matt Franck, Water Resources Task Lead, CH2M HILL

Yoel Gilon, Senior Vice President, BrightSource Industries Israel

Roger Gray, Principal, Great Northern Exchange Consulting, LLC

Wendy Haydon, Visual Resources Task Lead, CH2M HILL

Clint Helton, RPA, Cultural Resources Task Lead, CH2M HILL

Steve Hill, Air Quality Task Lead, Sierra Research

Amy Hiss, Botanist and Wetland Ecologist, CH2M HILL

Ann Howald, Botanical Resources, Garcia and Associates

Russell Huddleston, Wetland Ecologist, CH2M HILL

Mark Kubik, P.E., Principal Engineer, West Yost Associates

Thomas Lae, P.G., Geologic Hazards and Resources Task Lead, CH2M HILL

Steve Long, Soils Task Lead, CH2M HILL

Sarah Madams, Hazardous Materials Management and Waste Management Task Lead, CH2M HILL

Thomas Priestley, Ph.D., AICP/ASLA, Senior Technologist, CH2M HILL

Thomas Reagan, Development Engineer

Kathy Rose, Ph.D., Soil and Water Scientist, CH2M HILL

Gary Rubenstein, Air Quality Senior Reviewer, Sierra Research

Andrew Sanders, Herbarium Curator, Department of Botany & Plant Sciences, University of California

Jennifer Scholl, Land Use Task Lead, CH2M HILL

W. Geoffrey Spaulding, Ph.D., Biological Resources and Paleontological Resources, CH2M HILL

Todd Stewart, P.E., Director, Project Development, BrightSource Energy, Inc.

John Woolard, President and CEO, BrightSource Energy Inc.

Fatuma Yusuf, Ph.D., Socioeconomics Task Lead, CH2M HILL

DECLARATION OF
JOHN L. CARRIER

I, John L. Carrier, declare as follows:

1. I am presently employed by CH2M HILL , Incorporated as Program Manager.

2. A copy of my professional qualifications and experience are attached hereto and incorporated herein by reference.

3. The attached testimony on Project Description, Biological Resources and Alternatives was either prepared by me or under my direction for the Ivanpah Solar Electric Generating System project based on my independent analysis, supplements thereto, data from reliable sources, and my professional experience and knowledge.

4. It is my professional opinion that the prepared testimony is valid and accurate with respect to the issue(s) addressed herein.

5. I am personally familiar with the facts and conclusions related in the testimony and if called as a witness could testify competently thereto.

I declare under penalty of perjury that the foregoing is true and correct to the best of my knowledge and belief.

Dated: November 16, 2009 Signed: John L Carrier

At: Sacramento, CA

John Carrier, J.D.
Project Manager

Education

Juris Doctorate
M.B.A., Administration
B.A., Sociology

Relevant Experience

Mr. Carrier has more than 28 years of professional experience including the practice of redevelopment law, project management, power plant licensing and siting, regulatory compliance, permitting, document preparation, and technical writing. For the last 11 years, Mr. Carrier has served as Program Manager overseeing all California power plant licensing work for CH2M HILL. For the last 22 years, Mr. Carrier has been involved with the power industry. In the mid-1980s he served as the Program Manager for EBASCO's Peak Load Siting Contract. Under that 3-year, $10 million contract, EBASCO served as adjunct staff to the California Energy Commission's staff analyzing power plant applications. Since then, Mr. Carrier has managed the preparation of Applications for Certification and other CEQA studies for developers such as independent power producers and municipal power agencies. For the last 3 years he has been almost exclusively involved in siting and licensing of utility-scale concentrating solar and photovoltaic plants in both California and Nevada. He is experience with licensing solar plants under both the Warren-Alquist Act and the National Environmental Policy Act and often works closely with the Bureau of Land Management as the property owner.

Representative Projects

BrightSource Energy, Ivanpah Solar Electric Generating System, California. Project Manager for the licensing of this 400 MW concentrating solar plant using a proprietary Distributed Power Tower (DPT) technology. The approximate 4,060-acre project will be built in phases (100 MW, 100 MW and 200 MW) and is located in the Mojave Desert on land managed by the Bureau of Land Management (BLM). Due to BLM's involvement, the environmental analysis will need to comply with both CEQA and NEPA. An Application for Certification (AFC) was prepared under the California Energy Commission (CEC) process. In addition to overseeing the preparation of the AFC, Mr. Carrier prepared the Alternatives analysis, Natural Gas Supply, and the Executive Summary sections.

Licensing and Permitting for City of Vernon, Vernon Power Plant (VPP) Project, Vernon, California. The City of Vernon proposed to license a 610-MW power plant under the CEC's the 6-Month "Fast Track" permitting process. The plant was configured using two natural gas- fired combustion turbine generators (CTGs) and one steam turbine generator (STG). It included approximately 4,500 feet of new 230-kV transmission line, approximately one mile of new 20-inch-diameter natural gas pipeline; approximately one mile of new sanitary sewer line; and would use recycled water. In addition to overseeing the preparation of the Application for Certification (AFC) and managing the licensing process, Mr. Carrier also prepared the Alternatives, Facility Closure, and Natural Gas Supply sections. This application was withdrawn.

Licensing and Permitting for City of Vernon, Vernon Power Plant (VPP) Project, Vernon, California. The 914-megawatt (MW) generating facility would consist of three combustion-turbine generations, three heat recovery steam generators with duct burners; one condensing steam turbine generator; and a 14-cell mechanical-draft cooling tower. The project proposed to connect the plant to Southern California Edison's (SCE) Laguna Bell Substation. Two 230-kV transmission line routes are being considered: the River Route is 4.8 miles long and the Randolph Route is 4.4 miles long. In addition the project would require about 2,300 feet of new 24-inch-diameter natural gas pipeline and 2,400 feet of new sanitary sewer line. In addition to managing the licensing effort, Mr. Carrier prepared the Executive Summary, Alternatives, Facility Closure, and Natural Gas Supply sections of the AFC. This controversial project would require emission credits from the South Coast AQMD under its priority reserve program. The application was eventually withdrawn due to lack of emission credits.

Licensing and Permitting for San Francisco Electric Reliability Project (SFERP) for San Francisco Public Utilities Commission. Project Manager for this controversial power plant project. The SFPUC proposed to develop a 145-MW simple-cycle plant in southeast San Francisco, using three LM 6000 turbines. Although construction of another power plant in southeast San Francisco was controversial, it was licensed by the CEC. The plant would be located two blocks south of the existing Portrero Power Plant. Major issued included remediation of the power plant site (contaminated fill); Air Quality mitigation measures; water supply; Environmental Justice; and the need for in-city generation.

Licensing and Permitting for San Francisco International Airport Combustion Turbine Project for San Francisco Public Utilities Commission. Related to the SFERP, the SFPUC also decided to license a 49-MW simple-cycle plant at the San Francisco International Airport. The power plant would provide power to the City and also provide emergency backup power to the airport. Because the plant was less than 50 MW it was licensed under CEQA with the City of San Francisco as the lead agency. An Initial Study/Mitigated Negative Declaration was prepared CH2M HILL as a contractor to the SFPUC and approved by the City.

Licensing and Permitting for Cosumnes Power Plant, Sacramento Municipal Utility District, California. Project manager for a two-phase, 1,000-MW combined-cycle power plant on buffer lands for the former Rancho Seco Nuclear Plant. Preparation of the AFC required analysis of 26 miles of new gas line crossing the Cosumnes River and several creeks. Key issues were water supply, air quality, cultural resources, biological resources, visual resources, and noise. The gas pipeline went through highly sensitive cultural resource areas and required extensive coordination with local Native Americans. The plant is currently in operation.

Licensing and Permitting for Walnut Energy Center, Turlock Irrigation District, California. Project manager for the licensing of this 250-MW combined-cycle generating facility configured using two natural-gas-fired combustion turbines and one steam turbine. The project included approximately 1,950 feet of new 115-kV transmission line, 670 feet of new 69-kV transmission line, 3.6 miles of new 8-inch-diameter natural gas pipeline, 1.6 miles of new 12- to 24-inch diameter pipeline for recycled water supply, and 0.9 mile of new pipeline for potable water supply to the plant. In addition to managing the licensing of the project, Mr. Carrier authored the Facility Closure and Alternatives analysis sections. Upon review of the AFC, CEC staff stated "The Walnut Energy Center Project AFC is one of the most complete applications recently filed with the Commission." Mr. Carrier also managed the construction compliance work. CH2M HILL provided compliance monitoring support in the areas of Biological Resources, Cultural Resources, and Paleontological Resources. The plant is currently in operation.

Modesto Irrigation District, MID Electric Generation Station (MEGS). Mr. Carrier served as the Project Manager and Socioeconomic Discipline Lead for the licensing of a nominal 95-megawatt (MW), natural-gas-fired, simple-cycle generating facility consisting of two natural-gas-fired combustion turbines, approximately 0.25 mile of new 69-kV subtransmission line and fiber optic cable, 0.25 mile of new 8-inch diameter natural gas pipeline, and water supply and wastewater tap lines into City of Ripon lines in Stockton Avenue. The Project would occupy 8 acres within a 12.25-acre parcel. A Small Power Plant Exemption is an exemption from the Warren-Alquist Act that allows the CEC to license plants under 50 MW under CEQA. This plant had noise and land use issues to resolve. The plant is currently in operation.

Modesto Irrigation District, Woodland Generation Station 2. Served as Project Manager and Socioeconomic Discipline Lead for the Small Power Plant Exemption (SPPE) to license an 80 MW plant in Modesto, CA. The new plant was located adjacent to and integrated with MID's existing Woodland Generation Station 1. Because the plant was located adjacent to an existing power plant and due to the California Energy Crisis, the project received local support. The SPPE was completed in a record setting 4.5 months. The plant is currently under commercial operation.

Calpine Corporation, Metcalf Energy Center. Served as Project Manager and Socioeconomic Discipline Lead for the licensing of this 600 MW power plant located in Coyote Valley in south San Jose. This highly controversial project took 2.5 years to license through the California Energy Commission and resulted in a precedent-setting override of local government after the mayor and city council voted no to grant the requested entitlements. Key issues included changes to local entitlements (annexation of a portion of the site to the City of San Jose and change in San Jose's General Plan and Zoning Ordinance); noise impacts to adjacent land uses; visual impacts, biological impacts included nitrogen deposition impacts to the San Francisco Bay checkerspot butterfly; air quality impacts, and use of ground water as a back-up water source. In addition to the licensing effort, CH2M HILL provided surveying, engineering services for the design of access roads and a railroad spur, and the design of the recycled water line for the City of San Jose and lateral into the plant. The plant is currently in operation. The plant is currently in operation.

DECLARATION OF
Mark Bastasch

I, Mark Bastasch, declare as follows:

1. I am presently employed by CH2M HILL Incorporated as an Acoustical Engineer.
2. A copy of my professional qualifications and experience are attached hereto and incorporated herein by reference.
3. I prepared he attached testimony on Noise and Vibration for the Ivanpah Solar Electric Generating System project based on my independent analysis, supplements thereto, data from reliable sources, and my professional experience and knowledge.
4. It is my professional opinion that the prepared testimony is valid and accurate with respect to the issue(s) addressed herein.
5. I am personally familiar with the facts and conclusions related in the testimony and if called as a witness could testify competently thereto.

I declare under penalty of perjury that the foregoing is true and correct to the best of my knowledge and belief.

Dated: 11/09/09 Signed: Mark Bastasch

At: Portland, Oregon

Mark Bastasch, P.E., I.N.C.E.
Noise Task Lead

Education

M.S., Environmental Engineering
B.S. (cum laude), Environmental Engineering

Professional Registrations

Registered Acoustical Engineer: Oregon (No. 58990PE)
Professional Environmental Engineer: Oregon (No. 58990PE)
Professional Civil Engineer: Oregon, 1999 (No. 58990PE)
Certified Water Rights Examiner: Oregon, 2000 (No. 58990WRE)

Distinguishing Qualifications

- Has prepared acoustical analysis or expert testimony for more than 15,000 megawatts (MW) from gas-fired facilities (primarily in California) and more than 5,000 MW from wind generation facilities nationwide
- Specializes in industrial noise measurements, modeling and control for power, industrial, and transportation clients
- Has prepared detailed noise models of numerous power facilities
- Has prepared comprehensive and cost effective compliance reports for several gas-fired power facilities demonstrating that permit conditions were satisfied

Relevant Experience

Mr. Bastasch is a registered acoustical, environmental, and civil engineer with more than 10 years experience conducting acoustical studies. Mr. Bastasch's acoustical experience includes preliminary siting studies, regulatory development and assessments, ambient noise measurements, industrial measurements for model development and compliance purposes, mitigation analysis, and modeling of industrial and transportation noise.

Representative Projects

BrightSource Energy, Ivanpah Solar Electric Generating System. Authored noise section of California Energy Commission Application for Certification. Successfully worked with CEC staff to streamline noise analysis and eliminate unnecessary field studies given remote project site and lack of noise sensitive receptors.

Licensing and Permitting for San Francisco Electric Reliability Project (SFERP) for San Francisco Public Utilities Commission. Noise task lead for this controversial power plant. The SFPUC proposed to develop a 145-MW simple-cycle plant in southeast San Francisco, using three LM 6000 turbines. Although construction of another power plant in southeast San Francisco was controversial, it was licensed by the CEC. The plant would be located two blocks south of the existing Portrero Power Plant. Major issued included remediation of the power plant site (contaminated fill); Air Quality mitigation measures; water supply; Environmental Justice; and the need for in-city generation.

Walnut Energy Center, Turlock Irrigation District, Turlock, California. Acoustical technical lead for a combined cycle power plant. Tasks included evaluating and measuring background noise levels; development of detailed noise model, comparison of expected noise levels with the City of Turlock, County of Stanislaus, and the California Energy Commission's (CEC) noise guidelines; preparing Application for Certification and subsequent amendments submitted to the CEC; regulatory negotiation; and review of Conditions of Certification. Additional tasks included development assistance with acoustical bid and guarantee specifications and independent analysis of manufacturer steam turbine generator enclosure.

Calpine GE LM6000 Peaker Program, Calpine Corporation, Dublin, California. Project manager and acoustical lead for Calpine's Peaker Program. Prepared California Environmental Quality Act level noise assessments for more than 10 LM6000-based peaking power plants located throughout northern California. Developed a flexible and streamlined program to accurately and quickly prepare acoustical assessment. Tasks included regulatory review and interpretation of city and county noise standards, ambient measurements and analysis, development of a standardized model that included several levels of optional mitigation and field verification at operating facilities, and regulatory negotiating.

Edison Mission Energy's GE LMS100 Peaking Facilities, Southern California. Acoustical technical lead for two simple cycle power facilities each utilizing 5 GE LMS100 combustion turbines in simple cycle. Tasks included evaluating and measuring background noise levels to determine and evaluate risk associated with potential CEC permit limits; extensive coordination with GE given limited available data resulting from short operating history of the LMS100 (these were the first LMS100 evaluated in California); preparing Application for Certification to the CEC. Additional tasks included development and review of acoustical bid and guarantee specifications for cooling towers, SCR, stack, transformers and other balance of plant equipment.

Tierra Energy, Eastshore Power Project, Hayward, California. The proposed facility would be a nominal 115.5 megawatt (MW) simple cycle power plant consisting of 14 Wärtsilä 20V34SG natural gas-fired reciprocating engine generators and associated equipment. As acoustical technical lead for this facility, tasks included evaluating and measuring background noise levels to determine potential CEC permit limits; preparing Application for Certification to the CEC. Review of available vendor data and commitments.

Pacific Gas & Electric, Humboldt Bay Repowering Project, Humboldt, California. The proposed facility will be a load following power plant consisting of 10 natural gas-fired Wärtsilä 18V50DF 16.3 megawatt (MW) reciprocating engine-generator sets and associated equipment with a combined nominal generating capacity of 163 MW. As acoustical permitting lead for this facility, tasks included evaluating and measuring background noise levels to determine and evaluate risk associated with potential CEC permit limits; preparation of Application for Certification to the CEC, conducting site tour with CEC's acoustical staff and review of existing EPC commitments.

DECLARATION OF

Loren Bloomberg

I, Loren Bloomberg, declare as follows:

1. I am presently employed by CH2M HILL Incorporated as a Principal Technologist
2. A copy of my professional qualifications and experience are attached hereto and incorporated herein by reference.
3. I prepared the attached testimony on Traffic and Transportation for the Ivanpah Solar Electric Generating System project based on my independent analysis, supplements thereto, data from reliable sources, and my professional experience and knowledge.
4. It is my professional opinion that the portions of the testimony that I prepared are valid and accurate with respect to the issue(s) addressed herein.
5. I am personally familiar with the facts and conclusions related in the testimony and if called as a witness could testify competently thereto.

I declare under penalty of perjury that the foregoing is true and correct to the best of my knowledge and belief.

Dated: 11/10/09 Signed: [signature]

At: Santa Ana, CA

Loren Bloomberg, P.E.
Traffic and Transportation Task Lead

Education

M.E., Civil Engineering
M.S., Civil Engineering
B.S., Systems Engineering

Professional Registrations

Professional Engineer: California (2000, No. 2060)

Distinguishing Qualifications

- Experienced in practical and theoretical applications of traffic operations, particularly for freeways, arterials, and ramp metering
- Broad background in transportation planning, conceptual design, and transportation systems analysis
- Expert in traffic simulation modeling
- More than 18 years of experience, including transportation modeling and analysis for local areas, corridors, and entire regions

Relevant Experience

Mr. Bloomberg is a senior traffic engineer and transportation planner with more than 18 years of experience who has led or played a key role in numerous large-scale planning and operations analyses. He has conducted studies and developed plans for local areas, corridors, and entire regions (including roadways, maritime facilities, and airports). Mr. Bloomberg's technical expertise is in simulation modeling and traffic operations, with a particular focus on conceptual engineering and traffic analysis. He is often called upon as a technical expert for CH2M HILL's modeling projects and is known as a project manager for his ability to complete traffic analyses accurately and efficiently while meeting client requirements. He is also an expert in the application of Context Sensitive Solutions (CSS), with successful project applications on a wide range of feasibility studies and preliminary engineering, and has taught CSS to over 400 agency staff across the U.S. Mr. Bloomberg is a member of the Highway Capacity Committee of the Transportation Research Board, the international group of 30 professionals charged with developing and maintaining the *Highway Capacity Manual*.

Representative Projects

Task Lead, Ivanpah Solar Electric Generating System, BrightSource Energy, San Bernardino County, California. As Traffic and Transportation task lead for analysis of a solar energy project in the Mojave Desert near the California/Nevada border, prepared the traffic and transportation analysis section of the Application for Certification. The analysis focused on construction-related impacts to traffic operations, construction workers, truck trips, and transport of hazardous materials. Also assessed the impacts on freeways, ramps, and local streets.

Task Lead, Chula Vista Energy Upgrade Project, MMC Energy, San Diego County, California. Traffic and Transportation Task Lead. Prepared the traffic and transportation analysis section of the Application for Certification.

Task Lead, Eastshore Energy Center, Hayward, California. Traffic lead for the Application for Certification for a new 115.5-megawatt intermediate/peaking load facility. Led the assessment of the traffic and transportation impacts associated with the construction and operation of the facility. Assessed traffic operations impacts, transport of hazardous materials and public safety. Developed strategic approached for the Transportation Management Plan, and represented the applicant (for transportation issues) at California Energy Commission meetings.

Project Engineer, Walnut Energy Center Traffic Control and Implementation Plan (TCIP), Turlock, California. Developed the traffic control plan for the utility (potable and recycled water) lines for the Walnut Energy Center in Turlock, California. The TCIP addressed the mitigation of traffic impacts to the existing transportation facilities to satisfy the requirements of the California Energy Commission Conditions of Certification.

Traffic Task Lead, San Francisco Energy Reliability Project, San Francisco, California. Was the task lead for traffic for completing the traffic and transportation section of the Application for Certification, a process similar to an EIR. The project is an energy plant in San Francisco, and traffic impacts focused on the construction activities.

Traffic Control Task Lead, Metcalf Energy Center Offsite Utilities, San Jose, California. Task lead for traffic control. As part of a fast-track, design-build effort to design and construct linear facilities (recycled water, sewer, and potable water) to support a new energy center, led the traffic control task for the project. Developed plans to support two pipeline alignments through 6 to 10 miles of urban streets. Worked with local agencies to develop a transportation management plan (TMP) to support agency requirements and maintain construction schedules.

Project Manager, Corridor System Management Plans (CSMPs), California. Project manager on two separate Caltrans contracts to develop CSMPs for freeway corridors throughout the state. CH2M HILL has the primary responsibility for modeling and analysis on three separate corridors: approximately 40 miles of I-5 near Los Angeles, approximately 35 miles of I-10 between Ontario and Beaumont, and approximately 23 miles of US 50 east of Sacramento. CH2M HILL is developing baseline travel demand and simulation models, coordinating data collection efforts, calibrating models, and evaluating proposed corridor strategies.

DECLARATION OF

John Cleckler

I, John Cleckler, declare as follows:

1. I am presently employed by CH2M HILL Incorporated as a biologist.
2. A copy of my professional qualifications and experience is attached hereto and incorporated by reference herein.
3. I am adopting the attached testimony on Biological Resources for the Ivanpah Solar Electric Generating System project based on my independent analysis and my professional experience and knowledge.
4. It is my professional opinion that the prepared testimony is valid and accurate with respect to the issue(s) addressed herein.
5. I am personally familiar with the facts and conclusions related in the testimony and if called as a witness could testify competently thereto.

I declare under penalty of perjury that the foregoing is true and correct to the best of my knowledge and belief.

Dated: 11/11/2009

Signed: ____________

At: Sacramento, CA

John Cleckler
Senior Wildlife Biologist

Education

B.S., Wildlife Biology

Distinguishing Qualifications

Experience conducting multiple species surveys
Experienced with U. S. Fish and Wildlife Service Section 7 consultation
Expertise preparing environmental permits

Relevant Experience

Mr. Cleckler has 18 years of experience, and has worked throughout the Western U.S. on a variety of plant and wildlife species and environmental issues.

Representative Projects

Wildlife Biologist, Ivanpah Solar Electric Generating System, BrightSource Energy, San Bernardino County, California. Coordinated with BLM, USFWS, CEC, and CDFG staff to address and resolve issues on a large scale solar project. Drafted a biological opinion, raven management plan, and desert tortoise translocation plan in cooperation with the BLM. Participated in public workshops.

Contract Biologist, USFWS Sacramento Field Office, Sacramento, California. As an Endangered Species Program Contract Biologist for the USFWS Sacramento Field Office, Mr. Cleckler has been conducting consultations pursuant to Section 7 of the Endangered Species Act of 1973, as amended, for four years. Consultations completed exclusively for Caltrans District 4 transportation projects that involved interaction with Caltrans staff, their contractors, and local agencies with interests in individual projects. Responsible for review of environmental documents and preparation of Biological Opinions for signature by the Field Office Supervisor or Regional Director. Negotiated mitigation of complex project impacts with project proponents and regulatory agencies, and developed reasonable and prudent alternatives and conservation recommendations or measures. Inspected projects for compliance with policies, standards, guidelines, agreements, and plans and makes recommendations for action to ensure compliance. Developed knowledge of the Endangered Species Act and the regulations, policies, case law, and Solicitor's opinions relating to its administration. Coordinated with transportation liaisons from the National Marine Fisheries Service and California Department of Fish and Game. Also participated in the coordinated USFWS national transportation liaison team by attending the 2009 International Conference on the Environment and Transportation and participating in discussions relating to ESA policies as they relate to transportation projects on behalf of the Sacramento Field Office.

Rice Solar Energy Project, Riverside County, California. Drafted an Application for Certification for a large scale solar project.

John Cleckler

Travis AFB, Solano County, California. Coordinated with USFWS staff to complete Section 7 consultations on Travis AFB projects. Drafted a biological opinion for C-17 Assault Landing Strip Project for the USFWS.

Hyampom Road, Trinity County, California. Performed protocol northern spotted owl surveys for a proposed highway improvement project. Drafted a biological assessment for the Federal Highway Administration.

Project Biologist Walnut Energy Center, Turlock Irrigation District, Stanislaus County, California. Conducted site reconnaissance surveys and participated in the preparation of the AFC. Prepared the AFC and the Biological Resource Mitigation Implementation and Monitoring Plan. Managed monitoring efforts, provided document review, and prepared the environmental training program associated with the Walnut Energy Center as the Designated Biologist.

Project Biologist, Metcalf Energy Center, Santa Clara County, California. Assisted in preparation of the Biological Resource Mitigation Implementation and Monitoring Plan, Resource Management Plan for the MEC Preserve, Fisher Creek Riparian Corridor Enhancement Plan, and Horizontal Directional Drilling Inadvertent Returns Contingency Plan. Managed monitoring efforts, document review, and prepared the environmental training program associated with the proposed Metcalf Energy Center.

Team Leader, Teayawa Energy Center Desert Tortoise Surveys, Riverside County, California. Performed protocol desert tortoise surveys along proposed utility lines associated with the Teayawa Energy Center project. Assisted with preparation and review of the Biological Resources section of the EIS/EIR.

Project Biologist, Environmental Quality Assurance Program Environmental Monitor, San Luis Obispo County, California. Provided environmental monitoring for the Arco Quadalupe Dunes cleanup and restoration project. Also provided consultation with project proponents regarding county permit limitation and requirements.

Project Biologist, Environmental Quality Assurance Program Onsite Environmental Coordinator, Santa Barbara County, California. Managed a crew of environmental monitors on a fiber optics installation project throughout the County of Santa Barbara. Also provided consultation with project proponents regarding the county permit limitation and requirements.

Project Biologist, Desert Tortoise Monitoring, Mission Geoscience, California. Performed desert tortoise monitoring for exploratory drilling project near Barstow, California. Included presentation of an environmental awareness training program.

DECLARATION OF

MARK H. COCHRAN

I, MARK H. COCHRAN, declare as follows:

1. I am presently employed by CH2M HILL Incorporated as a Senior Biologist.
2. A copy of my professional qualifications and experience are attached hereto and incorporated herein by reference.
3. I helped prepare the attached testimony on special-status wildlife for Biological Resources for the Ivanpah Solar Electric Generating System project based on my independent analysis, supplements thereto, data from reliable sources, and my professional experience and knowledge.
4. It is my professional opinion that the prepared testimony is valid and accurate with respect to the issue(s) addressed herein.
5. I am personally familiar with the facts and conclusions related in the testimony and if called as a witness could testify competently thereto.

I declare under penalty of perjury that the foregoing is true and correct to the best of my knowledge and belief.

Dated: November 10, 2009 Signed: Mark H. Cochran

At: Tucson, Arizona

Mark Cochran
Senior Biologist

Education

Graduate studies in Zoology and Wildlife Management at the University of Michigan and Humboldt State University, 1974–1980
BA, Biology, Grinnell College, 1972

Distinguishing Qualifications

- Expertise in NEPA/CEQA Compliance
- Biological Impact Assessments
- Natural Resources Planning
- Endangered Species Consultation
- Biological Surveys and Research

Relevant Experience

Mark Cochran has more than 30 years of experience providing a wide range of environmental services for utility companies, departments of transportation, the Department of Defense, and mining companies. Mr. Cochran also served as a Wildlife Biologist with the US Fish and Wildlife Service (USFWS) and Bureau of Land Management (BLM). His primary expertise is in the preparation of environmental impact statements and environmental assessments pursuant to the NEPA; environmental impact reports pursuant to CEQA; biological assessments (BA) pursuant to the federal and California endangered species acts; and natural resources management plans for the Department of Defense (Sikes Act) and Department of Interior (FLPMA). He has participated in all aspects of project management, including regulatory permitting assistance, agency liaison, report preparation and review, public presentations, budgeting, field investigations and research, supervision of field biologists, and client liaison.

Representative Recent Projects

Technical Review/Preparation of Biological Assessment (BA), Desert Tortoise Relocation Plan, Raven Control Plan, and special-status wildlife evaluations, Ivanpah Solar Electric Generating System (ISEGS) Project, BrightSource Energy Inc., 8/2008-Present. BrightSource proposes to develop the ISEGS in the Ivanpah Valley, California, on BLM administered lands. Authored or reviewed portions of the BA, desert tortoise relocation plan, raven control plan, and evaluations of special-status wildlife habitat. Worked closely with project management and client to provide expert opinions on desert tortoise biology, impact assessment, mitigation, and the permitting agencies' management and conservation strategies for protecting this federally and state listed species.

Senior Review for Biological Resources, Silver State Solar Project, NextLight Renewable Power LLC (NextLight), 9/2009-Present. NextLight proposes to construct, own, and operate a solar facility in the Ivanpah Valley near Primm, Nevada. Reviewed and then prepared draft BA and provided senior review of wildlife resources technical report.

Mark Cochran

Task Manager for Biological Resources, **Solar Thermal Facilities, Iberdrola Renewables, Inc. (IBR), 3/2008-Present.** Evaluated 5 potential sites to construct large thermal solar facilities on Bureau of Land Management (BLM) administered land in Arizona and New Mexico. Prepared survey reports including fatal flaw analysis and literature review. Prepared Biological Evaluations for BLM on two potential sites.

Task Manager for Biological Resources, UNEV Petroleum Pipeline Project, Holly Energy, 3/2007-Present. Holly Energy proposes to construction a 400 mile long pipeline to transport petroleum products from its refinery in Salt Lake City, Utah, to markets in southwest Utah and southern Nevada;. Mark prepared the majority of the biological resources technical reports and studies to support an EIS as well as paring the BA for ESA compliance.

Task Manager for Biological Resources, Saguaro to North Loop Transmission Line Project, Tucson Electric Power (TEP) and Southwest Transmission Cooperative, Inc. (SWTC), 8/2008-8/2009. TEP and SWTC proposed to construct three 138-kilovolt (kV) transmission lines, and one 115-kV transmission. Prepared a Biological Evaluation (BE) to assess the potential impacts to species protected under the Endangered Species Act (ESA) and other listed special-status species of plant and wildlife potentially occurring in the project area. Prepared other biological resources reports for to support NEPA (USDA, Rural Utilities Service) compliance and a Certificate of Environmental Compatibility (CEC) application with the Arizona Corporation Commission (ACC).

Task Manager for Biological Resources, East Line El Paso to Phoenix Expansion Project, Kinder Morgan Energy Partners, 8/2005-4/2007. To further increase capacity to serve the growing demand for petroleum products in Arizona, Kinder Morgan has proposed to further expand its East Line between El Paso, TX, and Phoenix, AZ. Mr. Cochran authored portions of the Feasibility Study; and is currently preparing portions of an Environmental Assessment; preparing a Biological Assessment; and conducting is field surveys.

Assistant Project Manager and Task Manager for Biological Resources, East Line Expansion Pipeline Project, Kinder Morgan Energy Partners, 9/2002-7/2006. Kinder Morgan's East Line is the only petroleum products pipeline serving the Phoenix and Tucson, Arizona areas from the East. To increase capacity to serve the growing demand for petroleum products in Arizona, Kinder Morgan proposed to expand this Line by adding 235 miles of pipe between El Paso, TX, and Phoenix, AZ. Mr. Cochran authored portions of the Feasibility Study; Environmental Assessment; conducted field Surveys, and continues to provide environment compliance support for construction. He also wrote the Biological Assessment for the project on behalf of the BLM.

Task Manager for Biological Resources, Habitat Conservation Plan (HCP), Town of Marana, Arizona, 11/2002-10/2004. CH2M HILL assisted the Town in Phase 1 of their conservation planning process in preparation of an HCP to guide the Town in meeting Endangered Species Act compliance requirements. As Task Manager Mark developed information for identifying target species, conservation goals, a threat/needs assessment of target species, and served as project liaison with the Town. He wrote technical sections of the draft HCP.

DECLARATION OF
STEVE DE YOUNG

I, Steve De Young, declare as follows:

1. I am presently employed by Bright Source Energy, Incorporated as a Vice President, Environmental, Health and Safety.
2. A copy of my professional qualifications and experience are attached hereto and incorporated herein by reference.
3. I prepared the attached testimony on Project Description, Biological Resources, Worker Safety and Alternatives for the Ivanpah Solar Electric Generating System project based on my independent analysis, supplements thereto, data from reliable sources, and my professional experience and knowledge.
4. It is my professional opinion that the prepared testimony is valid and accurate with respect to the issue(s) addressed herein.
5. I am personally familiar with the facts and conclusions related in the testimony and if called as a witness could testify competently thereto.

I declare under penalty of perjury that the foregoing is true and correct to the best of my knowledge and belief.

Dated: November 16, 2009 Signed: ______

At: Sacramento, CA

STEVEN DE YOUNG

Steve De Young has over twenty-eight years experience in managing interdisciplinary environmental projects and participation in environmental investigation, permitting, regulatory reviews and mitigation activities. Mr. De Young has a strong working knowledge of environmental laws and regulations, regulatory review processes, and compliance processes for a broad range of environmental issues. He also has extensive experience in the coordination and integration of varied environmental and engineering disciplines in preparing compliance support documents and procedures and liaison between federal, state, and local agencies, applicants, and the public.

PROFESSIONAL HISTORY

BrightSource Energy, Inc.
Vice President, Environmental, Safety and Health
October 2006 to present

Consultant - Environmental Project Manager
De Young Environmental Consulting
September 2000 to September 2006

Bechtel Group/Bechtel Environmental, Inc.
Project Manager – Bechtel Corporate Environmental Affairs
March 1993 to August 2000

Lawrence Livermore National Laboratory
Group Leader – August 1991 to February 1993

Bechtel Group/Bechtel Environmental, Inc.
Group Leader – October 1981 to July 1991

PROFESSIONAL

BrightSource Energy, Inc. **–** Mr. De Young is the Director Environmental, Safety and Health for BrightSource Energy. He is responsible for permitting of the Ivanpah Solar Electric Generating System (ISEGS). This project is being reviewed by the California Energy Commission as the lead state agency and the U.S. Bureau of Land Management (BLM) as the lead federal agency. This project sits on nearly 4000 acres of land managed by the BLM and is the first project to undergo joint CEC/BLM permitting under the existing BLM/CEC Memorandum of Understanding). Mr. De Young is also responsible for the processing of other applications submitted to the BLM for sites in California, Nevada, and Arizona as well as private sites in Nevada and Arizona. In addition, Mr. De Young is responsible for the implementation of the BrightSource Health and Safety program and is committed to fostering BrightSource's core value of health and safety.

City and County of San Francisco – Mr. De Young previously was the environmental project manager for two projects for the City and County of

San Francisco. One project was a 145 MW peaking facility under review by the California Energy Commission. The second project was a 45 MW peaking facility to be located at the San Francisco International Airport.

Calpine Corporation – Between 1999 and 2006 Mr. De Young worked with the Calpine Corporation as the Environmental Project Manager for several natural gas-fired power plants in California. These include the following:

- Metcalf Energy Center (600 MW combined-cycle plant, licensed 2001)
- East Altamont Energy Center (1100 MW combined-cycle plant, licensed 2003)
- Los Esteros Critical Energy Facility Phase 1 (180 MW simple-cycle peaking plant, licensed 2002)
- Los Esteros Critical Energy Facility Phase 1 relicense and Phase 2 combined-cycle conversion (320 MW combined-cycle plant, licensed 2005)
- San Joaquin Valley Energy Center (1100 MW combined-cycle plant, licensed 2004)

Mr. De Young's responsibilities included obtaining the California Energy Commission licenses for the projects and obtaining permits from other regulatory agencies such as the U.S. Fish and Wildlife Service, Western Area Power Authority, California Department of Fish and Game, State Lands Commission, U.S. Bureau of Reclamation and city planning agencies. Mr. De Young was the primary project interface with CEC staff and management and representatives of other regulatory agencies.

The Metcalf project was arguably the most disputed project ever licensed by the California Energy Commission (CEC). Mr. De Young and his team prepared over six thousand pages of testimony that ultimately led to the approval of the project despite significant opposition. The project was permitted using the CEC override authority and has since withstood lawsuits brought before federal, state and local courts.

Calpine Corporation – Compliance Mr. De young has also assisted Calpine Corporation in compliance activities associated with projects licensed before the California Energy Commission. His activities included the preparation of several amendments to the project license for the Metcalf Energy Center, preparation of pre-construction documents and approvals for the Metcalf and Los Esteros projects, construction compliance support for the Metcalf and Los Esteros projects, and permit maintenance (i.e., ensuring that permits remained valid for projects placed on hold) for the East Altamont and Russell City projects.

Modesto Irrigation District – Mr. De Young managed the permitting of the Modesto Irrigation District Electric Generation Station, a 95 MW simple-cycle power plant located in Ripon, CA. His responsibilities included: managing the preparation of the application, providing project coordination with regulatory agencies such as the California Energy Commission and the Central Valley Regional Water Quality Control Board, and resolving issues/concerns of the agencies.

Bechtel Corporation **–** In his nearly 17 years with the Bechtel Corporation Mr. De Young had a varied career in environmental permitting and compliance. He successfully completed various nuclear power plant licensing activities including preparation of Preliminary and Final Safety Analysis Reports. During Mr. De Young's six year assignment with Bechtel Power, he performed work assignments for the Hope Creek Nuclear Generating Station, Limerick Nuclear Power Plant, and Diablo Canyon. In addition to his nuclear licensing work, Mr. De Young was also involved in the permitting of a number of cogeneration facilities. On the Greenleaf Power cogeneration facility, Mr. De Young prepared a comprehensive permitting acquisition plan and schedule, prepared permit applications for submittal to regulatory agencies, and followed the permit applications through the various agency review processes. In addition, he prepared Fuel Use Act Exemption Petitions and Federal Energy Regulatory Commission (FERC) Certifications for the Basic American Foods and Greenleaf Power cogeneration facilities, and assisted in the preparation of the Basic American Foods Application for Certification that was submitted to the California Energy Commission. A summary of Mr. De Young's project management activities at Bechtel include:

- Preparation of comprehensive RCRA Part B permit applications, interim status documents, and pond closure plans for the FMC Pocatello Project.
- Manager of Regulatory Information and Auditing with Bechtel's Corporate Environmental Affairs organization. He was responsible for tracking changes in environmental laws and regulations, assessing potential impacts of these changes on Bechtel projects and clients, and issuing guidance on the changes in the form of compliance alerts and training "tool kits".
- Manager of the Bechtel Environmental, Inc. San Francisco Regulatory Analysis Group. Mr. De Young was responsible for the supervision, technical accuracy and overall development of a group of seven regulatory analysts. His responsibilities in this position included tracking and analyzing significant developments in federal and state environmental laws and regulations, assessing potential impacts on Bechtel's clients, developing strategies for successful environmental compliance, and technical oversight of the work products prepared by the regulatory analysts.

Lawrence Livermore National Laboratory **–** In his last position at the Lawrence Livermore National Laboratory (LLNL), Mr. De Young was Group Leader of the LLNL Environmental Legislation and Regulation Analysis Group. He was responsible for managing a group of LLNL and contractor technical personnel involved in the identification and analysis of emerging environmental laws and regulations at the federal, state, and local levels. Significant activities included the development of guidance documents and procedures designed to ensure consistent interpretation and implementation of regulatory requirements throughout the Laboratory, preparation of comment letters to regulatory bodies, and presentations to LLNL management on the potential impacts of new environmental requirements on Laboratory programs.

EDUCATION B.S., Environmental Sciences
California State University, Fresno

DECLARATION OF

TIMOTHY J. DURBIN

I, Timothy J. Durbin, declare as follows:

1. I am presently employed by West Yost Associates as a Managing Engineer.

2. A copy of my professional qualifications and experience are attached hereto and incorporated herein by reference.

3. I prepared the attached testimony on stormwater runoff for the Ivanpah Solar Electric Generating System project based on my independent analysis, supplements thereto, data from reliable sources, and my professional experience and knowledge.

4. It is my professional opinion that the prepared testimony is valid and accurate with respect to the issue(s) addressed herein.

5. I am personally familiar with the facts and conclusions related in the testimony and if called as a witness could testify competently thereto.

I declare under penalty of perjury that the foregoing is true and correct to the best of my knowledge and belief.

Dated: November 12, 2009 Signed: Timothy J Durbin

At: Davis, California

Timothy J. Durbin, P.E.

Professional Registration

Professional Civil Engineer, 1972
California No. 20651

Professional Civil Engineer, 1989
Oregon No. 16497PE

Education

M.S., Civil Engineering, 1971
Stanford University, California

B.S., Civil Engineering, 1967
Stanford University, California

Professional Affiliations

American Society of Civil Engineers

American Geophysical Union

International Association of Hydrogeologist

National Groundwater Association

Publications

Durbin, T.J., 1974, Digital simulation of the effects of urbanization on runoff in the upper Santa Ana Valley, California: U.S. Geological Survey Water-Resources Investigations 41-73, 44 p.

Durbin, T.J., and Hardt, W.F., 1974, Hydrologic analysis of the Mojave River, California, using a mathematical model: U.S. Geological Survey Water-Resources Investigation 17-74, 50 p.

Durbin, T.J., 1975, Selected effects of suburban development on runoff in south-coastal California: in Proceedings of Second National Symposium on Urban Hydrology and Sediment Control, Lexington, Kentucky, p. 209-217.

Durbin, T.J., 1975, Ground-water hydrology of Garner Valley, San Jacinto Mountains, California – a mathematical analysis of recharge and discharge: U.S. Geological Survey Open-File Report 75-305, 40

Tim Durbin has over 40 years of engineering experience and directs projects relating to groundwater and surface-water hydrology. Areas of expertise include design of multidisciplinary investigations, design of large-scale programs for the collection and interpretation of hydrologic data, and application of mathematical modeling to the analysis of problems in groundwater and surface-water hydrology.

Project Experience

Antelope Valley Groundwater Basin, California. The Antelope Valley groundwater basin is being adjudicated to address the overdraft within the basin. Developed criteria for defining the geographic extent of the groundwater. Developed estimate of natural recharge within adjudicated area. Work was done in support of litigation related to the adjudication. *City of Los Angeles, California.*

Seaside Groundwater Basin, California. The Seaside groundwater basin was adjudicated to balance the threat of seawater intrusion against the need for groundwater production to supply water to communities overlying the basin and within the Monterey Peninsula area. Developed a groundwater model to assess the relation between groundwater production and seawater intrusion. Work was done in support of litigation related to the adjudication. *California American Water, Monterey, California.*

Carbonate Aquifer System, Eastern Nevada. Analyzed the water-related impacts of groundwater development within the regional Carbonate Aquifer System that underlies central and eastern Nevada. The Southern Nevada Water Authority, which delivers water to Las Vegas and neighboring communities, is considering a project to import of groundwater from the Carbonate Aquifer. The analysis is focused on the possible impacts of the project on springs and phreatophytes. The work includes developing a groundwater model of the Carbonate Aquifer System. The model extends over an area covering 20,000 square miles. The work was done in support of hearings before the Nevada State Engineer on water-right applications by the Authority. The work was done also in support of the environmental compliance for the project. *Southern Nevada Water Authority, Las Vegas, Nevada.*

North Platte River, Wyoming and Nebraska. Analyzed the impacts of water-resource development and reservoir operations on water supply, streamflows, regional economics, and wildlife resources within the North Platte River Basin, Nebraska and Wyoming. Designed and directed a multi-disciplinary investigation involving agricultural engineers, groundwater hydrologists, surface-water hydrologists, agricultural economists, and environmental scientists in six different consulting firms. Work was done in support of litigation before the U.S. Supreme Court between the states of Nebraska and Wyoming. *Attorney General, Lincoln, Nebraska.*

Santa Monica Groundwater Basin, California. Analyzed the occurrence of MTBE in the Santa Monica groundwater basin, California. MTBE contamination from multiple sites has resulted in abandonment of public-supply wells. An analysis of the sources and fate of MTBE within the Santa Monica groundwater basin is being conducted. Work was done within the context of State and Federal regulatory proceedings and litigation. *ConocoPhillips, Houston, Texas.*

p.

Durbin, T.J., 1978a, Application of Gauss algorithm and Monte Carlo simulation to the identification of aquifer parameters: in Proceedings of 26th Annual American Society of Civil Engineers Hydraulic Division Specialty Conference, College Park, Maryland, p. 101-111.

Durbin, T.J., 1978b, Calibration of a mathematical model of the Antelope Valley ground-water basin, California: U.S. Geological Survey Water-Supply Paper 2046, 51 p.

Durbin, T.J., and Morgan, C.O., 1978, Well-response model of the confined area, Bunker Hill ground-water basin, San Bernardino County, California: U.S. Geological Survey Water-Resources Investigation 77-129, 39 p.

Arteaga, F.E., and Durbin, T.J., 1978, Development of a relation for steady-state pumping rate from Eagle Valley ground-water basin, Nevada: U.S. Geological Survey Open-File Report 79-261, 44 p.

Durbin, T.J., Kapple, G.W., and Freckleton, J.R., 1978, Two-dimensional and three-dimensional digital flow models of the Salinas Valley ground-water basin, California: U.S. Geological Survey Water-Resources Investigation 78-113, 134 p.

Van Denburgh, A.S., Seitz, H.R., Durbin, T.J., and Harrell, J.R., 1982, Proposed monitoring network for ground-water quality, Las Vegas Valley, Nevada: U.S. Geological Survey Open-File Report 80-1286, 25 p.

Durbin, T.J., 1983, Application of Gauss algorithm and Monte Carlo simulation to the identification of aquifer parameters: U.S. Geological Survey Open-File Report 81-688, 26 p.

Katzer, T., Durbin, T.J., and Maurer, D.K., 1984, Water-resources appraisal of the Galena Creek basin, Washoe County, Nevada: U.S. Geological Survey Open-File Report 84-433, 59 p.

Special Master, California. Assigned as Special Master in a technical dispute between City of San Bernardino, California and the Regional Water Quality Control Board. The issue is the cause of a wastewater discharge to the Santa Ana River. The work was being done within the context of a State regulatory proceeding. *Regional Water Quality Control Board, Santa Ana, California.*

Bookman-Edmonston Engineering, Inc., Sacramento, California. Vice President (May 1998 – January 1999)

Directed projects related to groundwater and surface-water hydrology. Directed a staff of about 30 engineers, hydrologists, biologists, and geologists. Examples of such projects include:

Flooding, Arizona. Analyzed the causes of flooding near Phoenix, Arizona. Residential and commercial areas were flooded during a summer storm. The analysis involved assessing the effect of irrigation ditches and other facilities on the depth of flooding. The work was done in support of litigation.

Pipeline Break, California. Analyzed the impact of floodflows on the failure of a stream pipeline crossing within Thousand Oaks, California. A large sewer line failed owing to channel erosion during an extreme flood event. The recurrence interval of the erosion event was analyzed. The work was done within the context of a State regulatory proceeding.

Hydrologic Consultants, Inc., Sacramento, California. President (March 1989 – May 1998)

Directed projects related to groundwater and surface-water hydrology. Directed a staff of about 10 hydrologists, geologists, and engineers. Examples of such projects include:

Lake Tahoe, California and Nevada. Analyzed the impacts of urban development on the water quality of Lake Tahoe, California. Work involved the analysis of sediment and nutrient transport in streams tributary to the lake and nutrient cycling within the lake. Work was done for litigation.

Streamflow Temperature, California. Analyzed streamflow temperature within the Owens River, Owens Valley, California. Work was done to evaluate the hydrologic feasibility of reestablishing a fishery within the Owens River.

Groundwater Salinity, California. Analyzed the source and management of surface-water and groundwater salinity within the Lompoc groundwater basin. Work involved developing groundwater and surface-water models of the Santa Ynez River basin, including salinity models. Work was done in support of litigation.

Agricultural Drainage, California. Analyzed the causes and management of drainage water discharges from the Firebaugh and Central California Water District to natural watercourses and the San Joaquin River. Work was done in support of litigation.

FERC Re-licensing, California. Developed a model for the optimal use of ground water and surface water within the Turlock and Modesto Irrigation Districts for the benefit of water supply and environmental resources. Work was done in support of the FERC re-licensing of New Don Pedro Reservoir.

Seawater Intrusion, California. Analyzed seawater intrusion in the Salinas Valley. Analyzed the impacts of groundwater pumping on seawater intrusion. Analyzed the impacts of reservoir operations on streamflow recharge and seawater intrusion. Work was done in support of litigation.

Petroleum Contamination, California. Analyzed the source of soil and groundwater contamination by petroleum hydrocarbons at Santa Barbara, California. Work was done in support of litigation. Analyzed the source of soil and groundwater contamination by petroleum hydrocarbons at Oxnard, California. Work was done in support of litigation.

San Bernardino Groundwater Basin, California. Analyzed the occurrence of high groundwater levels in the San Bernardino Valley, California using surface-water and

Kapple, G.W., Mitten, H.T., Durbin, T.J., and Johnson, M.J., 1984, Analysis of Carmel Valley alluvial ground-water basin, California, using digital flow model techniques: U.S. Geological Survey Water-Resources Investigation 83-4280, 45 p.

Hromadka, T.V., and Durbin, T.J., 1984, Adjusting the nodal point distribution in domain ground-water flow numerical models: in Proceedings of Fifth International Conference on Finite Elements in Water Resources, p. 265-284.

Durbin, T.J., and Berenbrock, C., 1985, Three-dimensional simulation of free-surface aquifers by the finite-element method: U.S. Geological Survey Water-Supply Paper 2270, p. 51-67.

Mitten, H.T., Lines, G.C., Berenbrock, C., and Durbin, T.J., 1988, Water resources of Borrego Valley and vicinity, San Diego County, California: Phase 2, Development of ground-water flow model: Water Resources Investigations 87-4199.

Martin, P., and Durbin, T.J., 1990, Identification of net-flux rates for ground-water models: U.S. Geological Survey Water-Supply Paper, 2340, pp. 119-130.

Hromadka, T.V., and Durbin, T.J., 1986, Two-dimensional dam-break analysis for Orange County Reservoir: Water Resources Bulletin, v. 22, n. 2, p. 249-256.

Hromadka, T.V., and Durbin, T.J., 1986, Modeling steady-state advective transport by the CVBEM: Engineering Analysis, v. 3, n. 1, p. 9-15.

Durbin, T.J., 1988, Two-dimensional simulation of ground-water flow by finite-element method: Microsoftware for Engineers, v. 2, n. 1, p. 40-48.

Azrag, E.A., Durbin, T.J., and Nour El-Din, N.N., 1986, Two-dimensional simulation of solute transport by finite-element method: Microsoftware for Engineers, v. 2, n.

groundwater models. High groundwater levels resulted from excess artificial recharge and other factors. Work was done in support of litigation.

Arkansas River, Colorado and Kansas. Analyzed the effects of groundwater pumping and other factors in the depletion of streamflow in the Arkansas River at the Colorado-Kansas state line using surface-water, groundwater, and institutional models. Work was done in support of litigation in the U.S. Supreme Court between the states of Kansas and Colorado.

Geothermal Development, California. Analyzed the effects of geothermal development on thermal-spring discharges in the Mammoth Lakes area, California using groundwater and heat-transport models. Work was done in support of litigation.

S.S. Papadopulos & Associates, Inc., Davis, California. Vice President and Manager of Davis office (October 1985 – March 1989)

Directed and conducted investigations of numerous aspects of groundwater hydrology. Examples of such projects include:

Love Canal, New York. Analyzed the migration of groundwater contaminants at the Love Canal hazardous waste site in Niagara Falls, New York using a groundwater model. The Love Canal site is a Superfund Site. Work was done in support of litigation.

Groundwater Contamination, New Jersey. Analyzed the migration of groundwater contaminants at the Lone Pine landfill near Freehold, New Jersey. The Lone Pine landfill is a Superfund site. Work was done as part of a remedial investigation.

Modeling Code. Developed a computer program for the simulation of soil-water movement within and near a land-disposal facility. Work was done for the U.S. Environmental Protection Agency in support of the preparation regulations relating to the design of cover, liner, and leak-detection systems for land-disposal facilities.

Sediment Transport, California. Analyzed the impacts of urban development on flooding and sediment transport for streams in Orange County, California. Work supported the permitting of a large residential and commercial development project.

Williamson and Schmid, Hydrotec Division, Davis, California. Manager of Davis office (July 1984 – October 1985)

Directed and conducted investigations for evaluation of groundwater resources, management of regional groundwater systems, and evaluation of hazardous waste sites. Studies involved identification of essential hydrologic issues, collection of hydrologic data, and application of quantitative methods to evaluate alternatives and to select an optimal solution. Examples of such projects include:

Groundwater Contamination, California. Developed a three-dimensional groundwater model of a physical barrier at a hazardous waste landfill in order to evaluate performance of the existing barrier and proposed modifications. Work was done for regulatory compliance.

Isotope Geochemistry, California. Analyzed a hazardous waste site using isotope geochemistry and groundwater models as investigative tools. Work was done for regulatory compliance.

Groundwater Salinity, Nevada. Analyzed the utilization of fresh water body overlying saline water using surface geophysical techniques and a density-dependent groundwater flow model.

U.S. Geological Survey, Water Resources Division, California District. District Chief (GS-15) (August 1982 – July 1984)

Managed California District (350 persons in 14 offices) with annual budget of $25 million (in 1995 dollars) for hydrologic investigations. Responsible for developing plans for hydrologic investigations and ensuring plans were implemented. Provided

3, p. 171-180.

Atkinson, L.C., Durbin, T.J., and Azrag, E.A., 1992, Estimating the effects of non-Darcian flow on inflow to a pit and slope stability: Society for Mining, Metallurgy, and Exploration 1992 Annual Meeting, Paper 92-156, 4 p.

Durbin, T.J., and Atkinson, L.C., 1993, Optimizing the design of mine dewatering systems: Society for Mining, Metallurgy, and Exploration 1993 Annual Meeting, Paper 93-103, 5 p.

Avon, L., and Durbin, T.J., 1994, Evaluation of the Maxey-Eakin method for estimating recharge to ground-water basins in Nevada: Water Resources Bulletin, v. 30, n. 1, pp. 99-112.

Durbin, T.J., Bond, L.D., 1997, FEMFLOW3D: A finite-element program for the simulation of three-dimensional aquifers, Version 1.0: U.S. Geological Survey Open-File Report 97-810, 338 p.

Hromadka, T. V., Durbin, T.J., 2000, Estimating changes in sediment transport trends due to catchment changes: in Proceedings of Floodplain Management Association Conference on Non-Structural Solutions to Floodplain Management, San Diego, Calif.

Rajagopal-Durbin, A., and Durbin, T. J., 2008, Wells are not always water follies: Sustainable groundwater policies for the American West: Water Policy, v. 10, n. 2, p. 145-164.

Durbin, T. J., and Delemos, D. W., 2007, Adaptive under relaxation of Picard iterations in ground-water models: Ground Water, v. 45, n. 5, p. 648-651. .

Durbin, T. J., Delemos, D. W., and Rajagopal-Durbin, A., 2008, Application of superposition to non-linear ground-water models: Ground Water, v. 46, n. 2, p. 251-258.

Bredehoeft, J., and Durbin, T., 2009 Groundwater development – the time to full capture problem: Groundwater, v. 47, n. 1, pp. 2-9.

organizational and technical input to development of large scale, multi-agency investigations. Examples of such projects include:

Agricultural Drainage, California. Investigation of water quality related to agricultural drainage from the west side of San Joaquin Valley, California.

San Francisco Bay, California. Investigation of hydrodynamics of San Francisco Bay and Sacramento-San Joaquin, California Delta hydrologic systems.

Groundwater Exports, California. Investigation of the effects of exporting water from Owens Valley groundwater basin, California, including both hydrologic and biological impacts.

Central Valley Groundwater, California. Assessment of the groundwater resources of the Central Valley, California. Work was part of the Central Valley Regional Aquifer System Analysis (RASA).

Modeling Code. Development of numerical finite element codes (now used within the U.S. Geological Survey) for simulation of two- and three-dimensional groundwater flow and solute transport.

U.S. Geological Survey, Water Resources Division, Nevada District. District Chief (GS-14) (January 1980 – August 1982) and Assistant District Chief (GS-13) (July 1977 – August 1982)

Managed Nevada District (80 persons in three offices) with annual budget of $10 million (in 1995 dollars) for hydrologic investigations. Projects included:

Truckee River, Nevada. Design and organization of Truckee-Carson River Quality Assessment and Great Basin Regional Aquifer System Analysis (RASA).

Groundwater Management, Nevada. Development of groundwater and solute transport models for Washoe Valley, Galena Creek, Eagle Valley, and Carson Valley groundwater basins in Nevada.

Geothermal Development, Nevada. Design and organization of regional geothermal investigations of areas throughout Nevada including Dixie Valley, Ruby Valley, Black Rock Desert, and Carson Desert.

U.S. Geological Survey, Water Resources Division, California District. Hydrologist (GS-13) (December 1975 – July 1977), Hydrologist (GS-12) (October 1974 – December 1975), Hydrologist (GS-11) (September 1973 – October 1974), and Hydrologist (GS-9) (July 1972 – July 1977)

Served as Project Chief for numerous groundwater projects involving hydrogeologic and geophysical investigations and groundwater modeling. Conducted research in development of finite-element models for simulation of groundwater flow and mass transport. Applied results of research to solution of management problems and provided assistance to hydrologists within USGS and other public agencies in use of these models.

Books

Hromadka, T.V., Durbin, T.J., and DeVries, J.J., 1984, Computer methods in water resources: Lighthouse Publications, Mission Viejo (California), 344 p.

Hromadka, T.V., McCuen, R.H., Devries, J.J., and Durbin, T.J., 1993, Computer methods in environmental and water resources engineering: Lighthouse Publications, Mission Viejo (California), 590 p.

DECLARATION OF

Matthew Franck

I, Matthew Franck, declare as follows:

1. I am presently employed by CH2M HILL Incorporated as a Project Planner 2.
2. A copy of my professional qualifications and experience is attached hereto and incorporated by reference herein.
3. I am adopting the attached testimony on Water Resources for the Ivanpah Solar Electric Generating System project based on my independent analysis and my professional experience and knowledge.
4. It is my professional opinion that the prepared testimony is valid and accurate with respect to the issue(s) addressed herein.
5. I am personally familiar with the facts and conclusions related in the testimony and if called as a witness could testify competently thereto.

I declare under penalty of perjury that the foregoing is true and correct to the best of my knowledge and belief.

Dated: November 10, 2009 Signed: M Franck

At: Sacramento, California

Matt Franck
Water Resources Task Lead

Education

B.S., Environmental Policy Analysis and Planning

Distinguishing Qualifications

- Conducted environmental studies throughout California, Oregon, and Washington
- Experienced in preparing environmental documents to fulfill CEQA, NEPA, and other resource agency requirements

Relevant Experience

Mr. Franck has 19 years of experience in managing and writing environmental impact assessment documents in compliance with NEPA and CEQA. He also coordinates local, state, and federal regulatory processes. Mr. Franck's education and multidisciplinary experience, as well as his expertise in land use and resource planning, provide a solid background for evaluating complex environmental policy issues.

Representative Projects

Ivanpah Solar Electric Generating System, Bright Source Energy, Inc. Senior Technical Reviewer for Water Resources. Assisted in the preparation of a Water Resources analysis as a Senior Technical Reviewer. Project is a concentrated solar thermal facility proposed on 1,843 acres of land in the Mojave Desert. Key water resources issues of concern included availability of groundwater for the thermal facility and the disturbance to hydrology from the large construction site.

Humboldt Bay Repowering Project, PG&E. Task Manager for Water Resources. Prepared Water Resources analysis for a project to repower the existing Humboldt Bay Power Plant south of Eureka, California, using ten natural gas powered reciprocating engine generators. Key water resources issues of concern included stormwater quality to an extended detention basin, process wastewater discharges to a municipal system, and the decrease in lagoon flows because of reduced use of the existing once-through cooling system.

San Francisco Electric Reliability Project, Public Utilities District for the City and County of San Francisco, California. Task Manager for the preparation of the Water Resources section of this Application for Certification, a California Energy Commission process that is functionally equivalent to CEQA. The CEQA-equivalent evaluation is focuses on water, wastewater, and stormwater generation and use by the proposed facility in the context of Citywide compliance with the federal Clean Water Act and state Porter-Cologne Water Quality Control Act. Work efforts included testimony at evidentiary hearings.

AFCs for Walnut Creek Energy Park and Sun Valley Energy Project, Edison Mission Energy, City of Industry/Romoland, California. Provided support for two Applications for Certification before the California Energy Commission for similarly designed 500-MW natural gas-fired peaking power plants using the GE LMS100 advanced gas turbine technology. These applications were prepared in parallel and were filed at the Energy Commission within one week of one another. The AFCs were filed in December of 2005 and the projects are scheduled to begin construction in 2007.

Carlsbad Energy Center Project, NRG, Inc. Task Manager for Water Resources. Prepared Water Resources analysis for a project to repower the existing Encina Power Station in Carlsbad, California, using natural gas turbines. Project involved the use of reclaimed water from the nearby wastewater treatment plant, with an alternative source to use desalinated seawater. Key issues included marine impacts from seawater intake, brine disposal, and the capacity of the existing reclaimed water distribution system.

Lompoc Wind Energy Project, Pacific Renewable Energy Generation, LLC. Task Manager for Water Resources. Prepared Water Resources analysis for a project to install 60-80 wind turbines and ancillary facilities on 2,950 acres in Santa Barbara County, California. Key water resources issues of concern included disturbance to onsite water resources from the large extent of construction activities, stormwater quality control, and development of an onsite facilities (including a well and septic system) for the operations units.

Eastshore Energy Project, Tierra Energy, Inc. Task Manager for Water Resources. Prepared Water Resources analysis for a new natural gas power plant in Hayward, California, using fourteen reciprocating engine generators. Key water resources issues of concern included the development of structural features for onsite stormwater quality control, and process wastewater discharges to a municipal system.

Vernon Power Plant, City of Vernon. Task Manager for Water Resources. Prepared Water Resources analysis for a new natural gas power plant in Vernon, California, using three gas-fired turbines and one steam turbine. The project would redevelop an existing industrial site in this highly industrial community. Key water resources issues of concern included calculating drainage credits based on changes to the existing site drainage patterns, stormwater quality control during construction and operation, availability of recycled water, and the quantity and quality of wastewater discharges.

Westley-Marshall Substation and Transmission Line Project, Turlock Irrigation District. Task Manager for Water Resources. Prepared Water Resources analysis for a transmission line project (approximately 12 miles) in rural Stanislaus County, California. The project also involved nine potential substation sites. Key water resources issues of concern included floodplain risks and stormwater quality control during construction.

South Bay Replacement Project, LS Power Generation, LLC. Task Manager for Water Resources. Prepared Water Resources analysis for a project to repower the existing South Bay Power Plant in Chula Vista, California, using two natural gas turbines and one steam turbine. Project would result in the abandonment of the existing once-through cooling system used at the existing power plant. Key water resources issues of concern included stormwater quality during construction and plant operations and wastewater discharges (quantity and quality).

DECLARATION OF

Yoel Gilon

I, Yoel Gilon, declare as follows:

1. I am presently employed by BrightSource Energy, Inc. as a Senior Vice President.

2. A copy of my professional qualifications and experience are attached hereto and incorporated herein by reference.

3. I helped prepare the attached testimony on Traffic and Transportation and Visual Resources for the Ivanpah Solar Electric Generating System project based on my independent analysis, supplements thereto, data from reliable sources, and my professional experience and knowledge.

4. It is my professional opinion that the prepared testimony is valid and accurate with respect to the issue(s) addressed herein.

5. I am personally familiar with the facts and conclusions related in the testimony and if called as a witness could testify competently thereto.

I declare under penalty of perjury that the foregoing is true and correct to the best of my knowledge and belief.

Dated: November 15, 2009 Signed: ______________________

At: Jerusalem, Israel

Yoel Gilon

11 Yordei Hasira, Jerusalem 93225, Israel
Tel +972 77 202 5102   Fax +972 2 571 1059   Cell +972 52 813 4750
email: *ygilon@brightsourceenergy.com*

Professional experience

2006 – present: Senior VP, BrightSource Industries Israel

Solar central receiver tower R&D and performance models

2006 - Technology Business Development Consultant (Israel/US)

Client List (partial listing):

- Solel Solar Systems Ltd
- Israel Kroizer Ltd. (for American Israel Paper Mill, OPC Rotem, Israel Refinery)
- Electric Fuel Corporation, Arotech
- Ormat Industries Ltd.

1994 – 2006: ELECTRIC FUEL LTD. (Israel/US), subsidiary of Arotech Corp (Nasdaq: ARTX)

2001 – 2006: Vice President – Electric Vehicle Technologies

Head of Electric Fuel activities in the EV domain, completed four phases of $12 million zinc-air all-electric bus (zero emission) demonstration program with US FTA (Federal Transit Administration) in partnership with General Electric. Led development of the third- generation zinc-air battery; management of technology and engineering activities; coordination with transit agencies; and technology business development.

1994 – 2001: Director - Electric Vehicle Technologies

Coordination with German industry of $20 million electric vehicle demonstration program in Germany, including Deutsche Post (client), DaimlerChrysler and Opel, and subcontractor suppliers for vehicle-battery integration and refueling logistics. Initialization of advanced zinc-air battery development and EV implementation cooperation.

1991 – 1994: Project Development Manager - Ormat Industries (Yavne, Israel)

Business development of projects in the power energy domain diversifying Ormat products.

1985 – 1991: Vice President – System Engineering & Development, Luz Industries (Jerusalem, Israel)

Member of Luz executive management, responsible for the development of future Luz solar power projects. Responsible for the conceptual design of all Luz solar power plants in southern California including the guarantee performance model.

1982 – 1984: Programmer and Applied Mathematics Consultant (Jerusalem, Israel)

Chief programmer for the public health laboratory of Hebrew University. Development of a ray tracing program. Statistical analysis and graphics for the statistical services of the Hebrew University (for Teva, Hadassa Hospital).

1978: High School Physics Teacher, The Hebrew University High School (Jerusalem, Israel)

Education

M.Sc. Mathematics, Department of Mathematics, Hebrew University, Jerusalem, 1980

B.Sc. with distinction, Physics and Mathematics, Hebrew University, Jerusalem, Special advanced curriculum in both subjects, 1978

B.A. Fine Arts, Bezalel Academy of Art and Design, Jerusalem, 1978

DECLARATION OF

Roger Gray

I, Roger Gray, declare as follows:

1. I am presently employed by GNEX, LLC as an Electric Transmission Consultant.
2. A copy of my professional qualifications and experience are attached hereto and incorporated herein by reference.
3. I helped prepare the attached testimony on Transmission **System Engineering and Transmission Line Safety and Nuisance** for the Ivanpah Solar Electric Generating System project based on my independent analysis, supplements thereto, data from reliable sources, and my professional experience and knowledge.
4. It is my professional opinion that the prepared testimony is valid and accurate with respect to the issue(s) addressed herein.
5. I am personally familiar with the facts and conclusions related in the testimony and if called as a witness could testify competently thereto.

I declare under penalty of perjury that the foregoing is true and correct to the best of my knowledge and belief.

Dated: Nov. 14 2009 Signed: Roger Gray

At: Alamo, California

ROGER JAMES GRAY

630 Banister Lane, Alamo, CA 94507
925 324 6693 (mobile)
rjgray9@yahoo.com

EXTENSIVE AND DIVERSE UTILITY EXPERIENCE

In these areas

ELECTRIC SYSTEM OPERATIONS AND PLANNING, POWER CONTRACTS AND TRADING,
BUSINESS DEVELOPMENT, INFORMATION TECHNOLOGY,
SUPPLY CHAIN, TELECOMMUNICATIONS

Named by Computerworld Magazine Premier 100 CIOs in U.S.

EXPERIENCE

GREAT NORTHERN EXCHANGE CONSULTING, LLC, Alamo, CA
September 2004 – Present

After retiring from PG&E in August 2004, I founded my own management consulting practice using my experience in electric utility planning and system operations, power contracts and trading, information technology, electricity markets, telecommunications, supply chain and business development. Current and recent engagements have included: transmission and solar project consulting (Ausra, Inc. and BrightSource Energy, Renewable Ventures), Advanced Metering Systems (AMI) strategy and lead contract negotiator (Southern California Edison Company), power portfolio strategy, telecommunications strategic planning, IT systems replacement strategies and running two different technology start-up companies. My business model is based on working closely with internal teams to create rapid and sustainable results.

IP NETWORKS, San Francisco, CA
Oct, 2004 – February 2007
Consultant and later Chief Operating Officer for last-mile telecommunications company serving SF Bay Area. Responsible for operations, sales and marketing and business development functions to expand products and services and geographic footprint.

Business Results Achieved:

- Transformed company from chaotic start-up mode to mature business mode by developing work processes, standardizing operations, implementing construction and project management, developing budgeting, organizing sales, creating product pricing and accounting systems and improving all aspects of business performance.
- Developed and implemented IP Networks' first business plan.
- Successfully attracted capital to fund conversion from small growth to substantial growth model (doubled revenues for 2 years in a row).

POWER TRANSMISSION SOLUTIONS, Berkeley, CA (*Contract Executive through GNEX*)
December 2004 – December 2005

CEO and President of PTS, a technology start-up company. Responsibilities for the initial 6 month engagement included taking PTS from a company with a proprietary technology and a prototype with no current capital or formal business plan to a company with all the basic features necessary to move into sales and production, including, development of a strong business plan, raising capital, marketing and sales and a production plan. General objectives are to: avoid VC and use of debt and to be cash-flow positive after 16 quarters. Sold initial units and established different live test sites with utilities in U.S., South Africa and China.

PACIFIC GAS and ELECTRIC COMPANY, San Francisco, CA June 1985 – August, 2004

One of the largest combination natural gas and electric utilities USA; serves 14+ million people through out a 70,000-square-mile service area; 139,000+ circuit miles of electric lines; 45,800+ miles of natural gas pipelines; 4.9 million electric customer accounts; 3.9 million gas customer accounts; incorporated in 1905

Promoted nine times over 19 year career into increasingly responsible and strategic positions:

Vice President & CIO, Member of PG&E Management Committee, 1/2000 – 8/2004
CEO/General Manager- Electric Transmission Spin-off Company 5/2001 – 5/2003
Vice President, General Services, Member of PG&E Management Committee, 6/1996 – 1/2000
Director of Purchasing, Materials and Fleet, 1995 – 1996
Director, Power Market Planning and Energy Trading, 1994 – 1995
Director, Power Control & System Operations (PG&E's main operations nerve center), 1993-1994
Director, Electric Resources Planning (Strategic and Long-term planning), 1993 - 1994
Manager, Power Contracts (all wholesale electric contracts), 1989 - 1993
Senior Engineer, Power Contracts - Lead Negotiator, Lead FERC Witness, 1986 - 1989
Senior Analyst, Finance &Rates, 1985 – 1986

Vice President and CIO:

Lead operations of the 4th largest telecommunications system in California supporting mission critical gas and electric infrastructure of one of the nation's largest utilities serving 1 in 20 Americans. Oversee IT computing infrastructure including data center (client/server and mainframe) and distributed computing. (1,000+ applications, 20,000+desktop/laptops) including: IT application development and operations, IT user support services including 24x7 user help desk, Information Protection and IT physical and cyber asset security, and maintenance and optimization of enterprise applications such as SAP. Ran PG&E's Business Development Department and the generation of new revenues through

PG&E-continued

fiber/broadband and wireless businesses. Oversee management, development and leadership of 1,200 employees.

Business Results Achieved:

- **Reduced total IT costs 15% in first 6 months** (while absorbing average IT professional wage increases of 8%.)
- Achieved flat and declining budgets over 4 years while providing more products and services at measured higher levels of quality
- **Completed 100% of all IT projects on time and on budget:** all projects delivered promised performance using quantifiable metrics. (Projects ranged from extensive fiber and microwave
- telecommunication replacement projects to a complete replacement of our 37 year old Customer Information System)
- **Built and launched the new CIS system:** project had failed twice under previous leadership; initial independent assessments gave only a 50% chance of success. According to Metagroup, this is the largest utility CIS system
- **Achieved 100% recovery of all operating costs and capital investments**
- Reduced attrition rate from 20+% to less than 5%
- Increased internal customer satisfaction survey results from borderline “poor” to very good.
- Launched consolidation of distributed servers to save money, improve performance and improve disaster recovery
- Established IT governance processes to stop IT chaos, poor cost control and poor project selection and execution.

- **Business Development. Launched external fiber/broadband business and significantly expanded wireless business: These businesses had positive net cash flow and net revenue in year one of operation (2000) and have grown at approximately 13%** per year in spite of a weak telecom market; businesses create approximately $13M in net revenue and have required $0 in capital investment; year 2004 plan includes recently announced launch to go into broadband over power lines with key partners
- **Diversity.** Major improvements in recruiting and promoting female and minority employees. Met and exceeded EEO goals. Mentored female and minority employees.

<u>CEO and General Manager-ETrans</u>

As CEO (General Manager) of ETrans, developed and implemented complete business and operational plans to spin-off of PG&E’s electric transmission business. (1,200 Employees, $2.4B Assets, $700M/year revenue).

***VP General Services and Director of Purchasing, Materials and Fleet**:*

Held overall responsibility for purchasing of 95% of PG&E direct and indirect goods as well as Supply Chain and Logistics for PG&E and varied other technical and support services as well as PG&E's Research and Development functions. Related responsibilities included: materials management and distribution, Quality Control and inspections processes for materials and services, management of PG&E land assets (approximately 3rd largest private land holdings in California), management and operation of extensive Corporate Real Estate/facility holdings, Corporate Security (physical and employee security and coordination of all company emergency plans and disaster recovery plans), and technical support services including geotechnical and seismic support for nuclear facilities, dams, electric and gas assets and buildings. Held additional authority for seismic safety plans, risk management and prioritization, operations and maintenance of largest utility fleet in United States, operation of company aircraft and management, development and leadership of 2,000 employees.

Business Results Achieved:

- **Reduced total costs of PG&E's materials and services from $1.7 to $1.15** billion over the period 1997-1999 while increasing spending in core areas such as vegetation management and investments in PG&E's key electric and gas infrastructure
- **Increased quality metrics** of all key materials such as transformers, poles, cable and other electric and gas infrastructure; this was critical to reduce Total Cost of Ownership and not just first costs of purchases - Example: reduced TCO of distribution transformers by 33%
- **Business Development. Created new revenue sources**: Increased sales of assets from $0 to $30 million per year with no required capital investment.
- Reduced total Corporate Real Estate costs by $15 million year in spite of Bay Area Real Estate costs nearly doubling during the same period
- **Reduced overall expenses for General Services units by 25% over the period 1996-1999**
- Increased customer satisfaction survey results to near excellent across the board
- Increased inventory turns to 6.5 from 2.0; improved stock availability from high 80s to 98.5; reduced inventory to $45 million from $135 million to turn back dollars into working capital
- Put in place fully developed emergency plans, disaster recovery plans and alternate company headquarters plan; this became the foundation of PG&E's Y2K plan
- Business Development. Developed outside revenue stream from technical services group (1999 $1million/year with no capital investment)

PG&E-continued

- Moved R&D function from an under-valued department that largely was not tied to PG&E's business goals to a department that clearly demonstrated value because clients funded R&D.
- **Employee Development.** Co-founded PG&E's Leadership Development Initiative. This 13 year old program is PG&E's primary development program for future PG&E leadership. Helped create and deliver curriculum and training.
- **Diversity.** Improved PG&E's diversity purchasing results by nearly 100% from WMDVBE suppliers. Sponsored and developed relationships with small businesses.
- **Diversity.** Major improvements in recruiting and promoting female and minority employees. Met and exceeded EEO goals. Mentored female and minority employees.

Director, Power Market Planning and Energy Trading:

Planned for, and acquired, electric resources for 1 month to 5 year time horizon to meet customer requirements. Mitigated financial risk associated with an inherently volatile commodity (electricity). Kept prices stable. Created, and maintained, the highest ethical standards in electricity trading group.

Business Results Achieved:

- No electric resource shortages
- No financial losses incurred
- Stable prices for customers
- Absolute integrity maintained

Director, Power Control & System Operations (PG&E's main operational nerve center):

Keep the lights on in Northern California 24x7. Do it in a cost effective manner.

Business Results Achieved:

- Only significant loss of customer load was due to major firestorm in San Luis Obispo County in 1994; this loss of load was due to a request by the California Department of Forestry for safety of firefighting personnel; achieved full recovery of all customers within 24 hours
- Reduced departmental costs while maintaining and improving measure levels of reliability; achieved through change in operations philosophy and detailed risk and failure analysis

PG&E-continued

Director, Electric Resource Planning:

Responsible for developing long-term electricity portfolio plan.

Business Results Achieved:

- Developed and implemented new reserve planning standards to lower customer costs while improving overall reliability
- Fundamentally changed planning models from engineering-based models to economic and market-based models
- Created and advocated deregulation model before regulators (a different model was ultimately adopted. unfortunately!

Manager, Power Contracts (all wholesale electric contracts)

Business Results Achieved:

- Created total new revenues of over $500 Million/year with approximately 40% margin
- Settled over $1 Billion in litigation (sum of cross claims); net result payment of $250 million
- Developed, negotiated and implemented new market structures between PG&E and other utilities.

EARLY CAREER and OTHER WORK EXPERIENCE

LOUIS DREYFUS ELECTRIC POWER, INC, Vice President, Marketing, 1995

LOS ANGELES DEPARTMENT OF WATER AND POWER, Assistant Electrical Engineer, 1984 - 1985

SOUTHERN CALIFORNIA EDISON, Research and Development Analyst, 1983

BECHTEL, Assistant Electrical Engineer, 1982

EDUCATION

B.S., Electrical Engineering and B.S. Computer Science, University of California, Berkeley, Berkeley, CA
Emphasis: Electric Powers Systems, Energy Policy and Renewable Resources

Additional Course work at UC Berkeley, Graduate School of Public Policy
Additional Course work at Diablo Valley College, Economics
Awarded: University of California Regents Scholarship
University of California Alumni Scholarship

CURRENT & PAST CIVIC ACTIVITIES

Advisory Board Member, UC Berkeley, Electrical Engineering and Computer Science
Starter, Livorna Swim Team, Walnut Creek
Manager and Coach, San Ramon Valley Girls' Athletic League, Softball
Board Member, San Ramon Valley Girls' Athletic League, Softball
Manager and Coach, San Ramon Valley Little League
Coach, Danville Basketball League
Volunteer, Habitat for Humanity constructions sites (home building)
Various efforts to bridge "digital divide" in our country
Officer Advisor, PG&E Hispanic Employees Association

UNIVERSITY PRESENTATIONS AND PUBLICATIONS

UC Santa Barbara, Engineering Department: Electric Utility Deregulation and Energy Markets.
UC Berkeley, Engineering Department: Electric Utility Operations
University of Pennsylvania, Wharton School of Business: Digital Economy Meets Old Economy (Electric Utility Industry)
SFSU, College of Business: Information Technology in Business
SFSU, College of Business: Converting Business Strategy into Information Technology Strategies
California State University – East Bay: Business Planning and Business Development
SFSU, College of Business: Business Ethics
CIO Handbook. Co-author. Chapter on Business Ethics

DECLARATION OF
WENDY HAYDON

I, WENDY HAYDON, declare as follows:

1. I am presently employed by CH2M HILL Incorporated as an Environmental Planner.

2. A copy of my professional qualifications and experience are attached hereto and incorporated herein by reference.

3. I prepared the attached testimony on Visual Resources for the Ivanpah Solar Electric Generating System project based on my independent analysis, supplements thereto, data from reliable sources, and my professional experience and knowledge.

4. It is my professional opinion that the prepared testimony is valid and accurate with respect to the issue(s) addressed herein.

5. I am personally familiar with the facts and conclusions related in the testimony and if called as a witness could testify competently thereto.

I declare under penalty of perjury that the foregoing is true and correct to the best of my knowledge and belief.

Dated: 11/5/09 Signed: Wendy E Haydon

At: Sacramento, California

Wendy Haydon
Visual Resources Task Lead

Education

M.S., Recreation Administration
B.A., Environmental Studies

Relevant Experience

Ms. Haydon manages environmental document preparation and conducts recreation, visual resources, and land use analyses. She has 22 years of experience working on environmental documents meeting federal and state requirements, including Environmental Impact Reports (EIRs), Environmental Impact Statements (EISs), Environmental Assessments (EAs), Initial Studies (ISs), mitigation plans, and CEC environmental documents. She has participated in the planning or study of a wide variety of projects, including energy facilities, hydroelectric projects, infrastructure improvements, transportation facilities, land transfers, and aggregate production facilities. Ms. Haydon has considerable knowledge of the California Environmental Quality Act (CEQA), the National Environmental Policy Act (NEPA), and land use, recreation resource, and visual resource analyses.

Representative Projects

Application for Certification, Ivanpah Solar Electric Generating System, BrightSource Energy (2007 to 2009). Conducted the visual resources analysis of a proposed solar power plant to be located on land within the U. S. Bureau of Land Management's jurisdiction, for an Application for Certification (AFC) submitted to the California Energy Commission. The task consisted of characterizing the existing surrounding landscape, identifying several Key Observation Points (KOPs) (sensitive receptor viewing locations) from a local golf course and the surrounding area, taking daytime photos from the KOPs, directing the preparation of visual simulations of the project as seen from the KOPs, assessing the visual impacts of the project, identifying mitigation for significant impacts, and assisting in the preparation of a conceptual landscape plan to minimize potential impacts from the project on views from certain locations within the golf course.

Application for Certification, San Francisco Electric Reliability Project, San Francisco Public Utilities Commission (2004). Conducted the visual resources analysis of a proposed 145-MW power plant, to be constructed and operated adjacent to the San Francisco Bay. The task consisted of characterizing the existing surrounding landscape, identifying Key Observation Points, taking daytime photos from the KOPs and from other locations in the city, directing the preparation of daytime visual simulations of the project as seen from the KOPs, assessing the visual impacts of the project, and identifying mitigation for significant impacts.

Property Value Analysis, Iowa Hill Pumped Storage Project, Sacramento Municipal Utility District (2004). Co-conducting the property value analysis of a pumped storage facility, ancillary facilities, transmission line, and improved roadway to be constructed and operated on/near Iowa Hill. Responsible for identifying privately owned properties within

3 miles of project facilities and determining which properties would have views that could be affected.

Initial Study/Mitigated Negative Declaration, Denair Substation, Turlock Irrigation District (2004). Conducted the visual resources analysis of the construction and operation of a proposed 115-kV electrical substation. The task consisted of characterizing the existing surrounding landscape with text and photos, assessing the visual impacts of the project, and identifying mitigation for significant impacts.

Application for Certification, Walnut Energy Center, Turlock Irrigation District (2002 to 2004). Conducted the visual resources analysis of the construction and operation of a proposed power plant. The task consisted of characterizing the existing surrounding landscape, identifying Key Observation Points, taking daytime photos from the KOPs, directing the preparation of daytime visual simulations of the project as seen from the KOPs, assessing the visual impacts of the project, and identifying mitigation for significant impacts.

Small Power Plant Exemption, MID Electric Generation Station (MEGS Ripon), Modesto Irrigation District (2002 to 2004). Conducted the visual resources analyses of the construction and operation of a proposed power plant. The task consisted of characterizing the existing surrounding landscape, identifying two Key Observation Points, taking daytime photos from the KOPs, directing the preparation of daytime visual simulations of the project as seen from the KOPs, assessing the visual impacts of the project, and identifying mitigation for significant impacts.

Application for Certification, Cosumnes Power Plant, Sacramento Municipal Utility District (2001 to 2003). Conducted the visual resources analysis of a proposed power plant to be constructed and operated adjacent to the existing Rancho Seco Nuclear Power Plant facilities. The task consisted of characterizing the existing surrounding landscape, identifying Key Observation Points (KOPs) (sensitive receptor viewing locations), taking daytime and nighttime photos from the KOPs, directing the preparation of daytime visual simulations of the project as seen from the KOPs, assessing the visual impacts of the project, identifying mitigation for significant impacts, and providing expert testimony before the CEC.

Small Power Plant Exemption, Woodland Generation Station 2, Modesto Irrigation District (2001). Conducted the visual resources analysis of a proposed power plant to be constructed and operated adjacent to the existing Modesto Irrigation District Woodland Generation Station. The task consisted of characterizing the existing surrounding landscape, identifying several Key Observation Points (KOPs) (sensitive receptor viewing locations), taking daytime photos from the KOPs, directing the preparation of daytime visual simulations of the project as seen from the KOPs, assessing the visual impacts of the project, and identifying mitigation for significant impacts.

DECLARATION OF

Clint Helton

I, Clint Helton, declare as follows:

1. I am presently employed by CH2M HILL Incorporated as a Senior Technologist.
2. A copy of my professional qualifications and experience are attached hereto and incorporated herein by reference.
3. I prepared the attached testimony on Cultural Resources for the Ivanpah Solar Electric Generating System project based on my independent analysis, supplements thereto, data from reliable sources, and my professional experience and knowledge.
4. It is my professional opinion that the prepared testimony is valid and accurate with respect to the issue(s) addressed herein.
5. I am personally familiar with the facts and conclusions related in the testimony and if called as a witness could testify competently thereto.

I declare under penalty of perjury that the foregoing is true and correct to the best of my knowledge and belief.

Dated: 11/9/09 Signed: ______________________

At: Santa Ana, CA

Clint Helton, RPA
Cultural Resources Task Lead

Education

M.A., Anthropology
B.A., Language and Literature

Professional Registration

Registered Professional Archaeologist (1999, No. 11280)

Distinguishing Qualifications

- Strong background in environmental impact evaluations, with particular expertise in conducting cultural resources studies in California, Colorado, Idaho, Nevada, Utah, and Wyoming
- Has 13 years of environmental management experience in the western U.S.
- Meets Secretary of Interior Professional Qualification Standards (36 CFR 61)
- Highly experienced managing cultural resources studies for large linear transportation and utility projects to meet requirements of National Environmental Policy Act (NEPA), National Historic Preservation Act (NHPA), California Environmental Quality Act (CEQA), and standards of the California Energy Commission (CEC), and Federal Energy Regulatory Commission (FERC)

Relevant Experience

Mr. Helton is an environmental consultant with more than 13 years of environmental management experience in the western United States. He has a strong background in environmental impact evaluations, having directed technical studies; negotiated with lead agencies, responsible agencies, and clients; and written, edited, and produced a substantial number of environmental review and technical documents. Mr. Helton has extensive experience of regulatory compliance, cultural and paleontological resources, NEPA and NHPA compliance activities, and federal regulations governing treatment of cultural resources, especially Section 106 of NHPA (36CFR800) and the Native American Graves Protection and Repatriation Act (NAGPRA) (43CFR10). Additionally, Mr. Helton is experienced with the challenges of preparing environmental documentation for large linear utility projects, including large interstate pipelines and is familiar with the process and guidelines of CEC and FERC among others. Mr. Helton has authored numerous environmental technical reports, cultural resources management plans, cultural resources studies, Programmatic Agreements, and Memorandums of Understanding (MOU) and contributed to many NEPA and CEQA documents for a variety of private and public sector clients.

Clint Helton, RPA

Representative Projects

Task Manager, BrightSource Energy, Ivanpah Solar Electric Generating System Project, San Bernardino County, California. Assisted with preparation of Application For Certification for California Energy Commission in support of a large proposed solar power generation facility covering over 4,000 acres of land managed by Bureau of Land Management in San Bernardino County, California. Responsible for preparation of cultural resources component of project, including archival research, field surveys, report preparation, and conducting Native American consultation.

Task Manager, Terra-Gen LLC Alta Wind Project, Kern County, California. Task Lead, quality control manager, and overall management of cultural resources studies for this 5,000-acre-plus alternative energy development project near the City of Tehachapi, Kern County, California. Provide regulatory guidance, regional technical expertise in cultural resources and coordination with Kern County. Supervised inventory for cultural resources, technical report preparation, and conducted Native American Consultation.

Task Manager, Iberdrola Renewables, Multiple Solar Energy Development Projects, Arizona, California, New Mexico, Nevada. Led preparation of cultural resources assessments for solar power generation facilities in AZ, NM, NV, and CA. Mr. Helton is acting as principal investigator for several critical issues analyses as well as full permit preparation of solar energy development projects in Arizona, California, Nevada, and New Mexico. Project acreages range from 5,800 acres to 35,000 acres.

Task Manager, PPM Energy, Solar Energy Development, Arizona, Nevada, California. Cultural resources assessments for solar power generation facilities in Arizona, Nevada, and California. Mr. Helton is acting as principal investigator for literature searches and field visits for several proposed solar energy projects in Arizona, California, and Nevada. Project acreages range from 2,000 acres to 25,000 acres.

Task Manager, Edison Mission Energy, Walnut Creek Energy Park Power Plant, California. Assisted with preparation of Application for Certification for California Energy Commission in support of this proposed 500-MW power generation facility in Los Angeles County, California. Responsible for preparation of cultural resources component of project, including field surveys, report preparation, and conducting Native American consultation.

Task Manager, Edison Mission Energy, Sun Valley Energy Center Power Plant, California. Assisted with preparation of Application for Certification for California Energy Commission in support of this proposed 500-MW power generation facility in San Bernardino County, California. Responsible for preparation of cultural resources component of project, including field surveys, report preparation, and conducting Native American consultation.

Task Manager, Chula Vista Energy Upgrade Project, MMC Energy, San Diego County, California. Task Lead and overall management of cultural resources studies for this 100-MW power plant upgrade project in San Diego County, California. Responsible for preparation of cultural resources component of project, including field surveys, report preparation, and conducting Native American consultation.

DECLARATION OF

Steve Hill

I, Steve Hill, declare as follows:

1. I am presently employed by Sierra Research as a Senior Engineer.
2. A copy of my professional qualifications and experience are attached hereto and incorporated herein by reference.
3. I helped prepare the attached testimony on Air Quality, Public Health, and Alternatives for the Ivanpah Solar Electric Generating System project based on my independent analysis, supplements thereto, data from reliable sources, and my professional experience and knowledge.
4. It is my professional opinion that the prepared testimony is valid and accurate with respect to the issue(s) addressed herein.
5. I am personally familiar with the facts and conclusions related in the testimony and if called as a witness could testify competently thereto.

I declare under penalty of perjury that the foregoing is true and correct to the best of my knowledge and belief.

Dated: November 13, 2009 Signed: ____________________

At: Oakland, California

sierra
research

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Fax: (916) 444-8373
Ann Arbor, MI
Tel: (734) 761-6666
Fax: (734) 761-6755

Résumé

Steve Hill

Education

2003, J.D., University of California, Hastings College of the Law, San Francisco
1978, M.S., Chemical Engineering, University of California, Berkeley
1976, B.S., Engineering, University of California, Los Angeles

Professional Experience

2006-present Senior Engineer
Sierra Research

Responsibilities include compliance assistance and strategy development for facilities subject to EPA and local air district enforcement actions; preparation of air quality sections of California Energy Commission Applications for Certification and Small Power Plant Exemptions; Clean Air Act Title V permit applications; preparation of Health Risk Assessments for Toxic Hot Spots program and permit applications; and preparation of permit applications for various industrial sources including Best Available Control Technology (BACT) review, offsets and emission reduction credit analyses, Prevention of Significant Deterioration (PSD), New Source Performance Standards (NSPS), and National Emission Standards for Hazardous Air Pollutants (NESHAPS) applicability review, and state portable equipment and air toxic control measure compliance review.

1996-2006 Manager, Permit Evaluation
Bay Area Air Quality Management District

Managed an engineering and support staff of 29, and acted as agency officer at public hearings. Served as frequent contact with reporters, management-level staff at other agencies, professional groups and associations, political bodies (e.g., councils, commissions, etc.), and the general public. Technical responsibilities included developing and implementing permit policy, serving as an expert witness at hearings, interpreting legislation and regulations, and drafting regulations.

1992-1996 Director, Administrative Services
Bay Area Air Quality Management District

Provided executive management oversight of the following sections within the District: Personnel, Business, Finance, Information Services, Facilities, and Fleet Maintenance. Prepared the agency's annual budget ($30 million) and served as the agency's Affirmative Action Officer.

1986-1992 Manager, Toxic Air Contaminant Evalution
Bay Area Air Quality Management District

Managed an engineering and support staff of 29, and acted as agency officer at public hearings. Served as frequent contact with reporters, management-level staff at other agencies, professional groups and associations, political bodies (e.g., councils, commissions, etc.), and the general public. Technical responsibilities included developing and implementing permit policy, serving as an expert witness at hearings, interpreting legislation and regulations, and drafting regulations.
Developed the District's Toxics Program from inception to maturity. Served as the national co-chair of the Toxics Committee of the Association of Local Air Pollution Control Officers, and served on the Toxics Subcommittee of the Federal Advisory Committee. Also helped develop the curriculum for the Hazardous Materials Management Certificate and Air Pollution Control Certificate programs for the University of California Berkeley Extension education program.

1980-1986 Senior Air Quality Engineer/Air Quality Engineer
Bay Area Air Quality Management District

1978-1979 Research Engineer
Chevron Research Company

Credentials and Memberships

Member, California Bar

DECLARATION OF
AMY HISS

I, Amy Hiss, declare as follows:

1. I am presently employed by CH2M HILL Incorporated as a botanist.
2. A copy of my professional qualifications and experience are attached hereto and incorporated herein by reference.
3. I helped prepare the botany portion of the attached Biological Resources testimony for the Ivanpah Solar Electric Generating System project based on my independent analysis, supplements thereto, data from reliable sources, and my professional experience and knowledge.
4. It is my professional opinion that the prepared testimony is valid and accurate with respect to the issue(s) addressed herein.
5. I am personally familiar with the facts and conclusions related in the testimony and if called as a witness could testify competently thereto.

I declare under penalty of perjury that the foregoing is true and correct to the best of my knowledge and belief.

Dated: 11/6/09 Signed: Amy Hiss

At: Sacramento, CA

Amy Hiss
Botanist and Wetland Ecologist

Education

M.A., Ecology and Systematic Biology, San Francisco State University
Recipient of Ledyard Stebbins Award for studies in Evolutionary Biology; recipient of Graduate Student Award for Distinguished Achievement in Biology
B.S., Botany, Humboldt State University
B.S., Environmental Biology, Humboldt State University

Distinguishing Qualifications

- Specializes in conducting and leading rare plant surveys, wetland delineations, and habitat mapping; conducting impact assessments; and preparing habitat mitigation and monitoring plans
- Experienced in preparing permit applications for a variety of agencies
- Experienced in preparing biological sections of environmental documents to fulfill CEQA, NEPA, and other resource agency requirements

Relevant Experience

Ms. Hiss has more than 18 years of experience in botany and wetlands ecology. She has conducted rare plant and noxious weed surveys and wetland delineations throughout much of California and southern Nevada. She is experienced in using GPS technology with submeter accuracy in the field to map findings and navigate to field sites. In addition, she prepares sections of CEQA documents, including AFCs, NEPA documents, and permits for USACE, RWQCB, and CDFG, mitigation and monitoring plans, and facilitates resource agency meetings.

Representative Projects

Ivanpah Solar Electric Generating System, San Bernardino County, California. Botany and noxious weed survey tasks: responsible for planning, preparation, contracting, mobilization, and management of more than 30 botanists. Protocol-level botanical surveys were conducted within an approximately 4,000-acre site. Several hundred populations of nine special status plants were observed during two years of surveys. Responsible for data QAQC of more than 6,000 GPS data points. With assistance from team, prepared botanical and other biology sections of the AFC and supplemental data requests. Assisted with waters of the U.S. delineation planning, data QAQC, and report preparation. Responsible for preparing CDFG and RWQCB permit applications.

Confidential Solar Electric Generating System Client, Clark County, Nevada. Senior botanist for botanical and noxious weed survey tasks: responsible for planning, preparation, contracting, mobilization, and management of more than 30 botanists. Protocol-level botanical surveys were conducted within an approximately 4,000-acre site on BLM lands. Coordinated NEPA and survey documentation and survey requirements with an interdisciplinary team composed of BLM resource specialists and client project management team. Survey documentation efforts are currently ongoing.

Confidential Solar Power Energy Client, Riverside County, California. Senior Biologist for reconnaissance-level siting studies for two large solar power generating sites in eastern Riverside County, California. Special status species occurrence was researched and assessed, and key biological resources issues that could constrain development, including sensitive natural communities and waters of the U.S., were identified. Results of the field surveys and literature review were used to recommend project redesign to minimize environmental impacts and mitigation costs.

Confidential Solar Power Energy Client, Kern County, California. Senior Biologist for reconnaissance-level siting study of three potential solar power generating sites in Kern County, California. Senior biologist for the team that assessed the likelihood of special status species occurrence and mapped the location of significant sensitive natural communities (including waters of the U.S.). Results of the field surveys and literature review were used to evaluate potential impacts to biological resources, including a core population of Mohave ground squirrel, a state-threatened species.

Confidential Solar Power Energy Client, Clark and Nye Counties, Nevada. Senior Biologist for reconnaissance-level siting study surveys for three potential solar power generating sites ranging in size from 4,000 to 21,000 acres. As part of a fatal flaws study, identified habitats present, including potential waters of the U.S., and assessed special status species occurrence. Advised client on the best site location to minimize environmental impacts and mitigation costs.

Northern Arizona FERC Relicensing Project. Responsible for identifying riparian plant species and collecting vegetation data. A sampling protocol was developed to explore the relationship between various stream flow regimes and soil moisture content in the riparian zone; the riparian vegetation community structure was analyzed relative to substrate.

Botanical Surveys, Ashland to Medford, Oregon, PGT Gas Transmission Line Expansion Project. Conducted botanical surveys for this gas transmission line expansion project. During surveys of the approximately 100-mile-long linear corridor, seven special-status plant species were identified. One plant species previously thought extirpated from Oregon, was identified within the project corridor. The locations of special-status plant species and plant community types within the project corridor were mapped using GPS, and all field information was imported to GIS for further data analysis.

96-mile Rock Creek-Rio Oso Transmission Line and Rock Creek Cresta Hydroelectric Project, PG&E. Project manager for a large habitat mapping effort. Habitat information was needed to fulfill requirements of an Additional Information Request (AIR) necessary to relicense the hydroelectric facility. Rare plant surveys were conducted concurrent with the habitat mapping effort. Identified more than 30 habitat types along the corridor. Managed the team that input vegetation and rare plant data into GIS to produce maps for the AIR submittal to the Federal Energy Regulatory Commission (FERC).

DECLARATION OF
ANN HOWALD

I, Ann Howald, declare as follows:

1. I am presently employed by Garcia and Associates as a Senior Botanist.
2. A copy of my professional qualifications and experience are either attached or were provided previously.
3. I participated in the preparation of the attached testimony on **Biological Resources** for the Ivanpah SEGS project. The portions I prepared are based on my independent analysis and my professional experience and knowledge.
4. It is my professional opinion that the prepared testimony is valid and accurate with respect to the rare plant issue(s) addressed herein.
5. I am personally familiar with the facts and conclusions related in the testimony and if called as a witness could testify competently thereto.

I declare under penalty of perjury that the foregoing is true and correct to the best of my knowledge and belief.

Dated: 28 October 2009 Signed: Ann M. Howald

At: Sonoma, California

Ann Howald
Botanical Resources

Education

M.A., Botany
B.A., Zoology

Professional Certifications

California Department of Fish and Game Scientific Collector's Permit

Relevant Experience

Ms. Howald has more than 25 years of experience as a senior botanist, restoration specialist and project manager. Her expertise includes vascular plant identification and collection methods, botanical survey techniques, plant ecology, habitat restoration, vegetation and rare plant restoration and monitoring, and invasive weed management. She is familiar with state and federal regulations that apply to rare plants, vegetation, and wetlands. She has planned, implemented and written final reports for many rare plant surveys. She has prepared and implemented management, restoration and/or monitoring plans for individual rare plant species, vegetation and habitat types, protected lands, and invasive weeds. She has prepared botanical sections of Environmental Impact Statements, Environmental Impact Reports, Habitat Conservation Plans, Biological Assessments, Environmental Assessments, and other documents required by the California Environmental Quality Act, the National Environmental Policy Act, and the U.S. and California Endangered Species Acts.

Representative Projects

Ivanpah Solar Electric Generating System (SEGS), BrightSource Energy; Eastern Mojave Desert, San Bernardino County (2007 to 2009, ongoing). Co-field supervisor for large crew of surveyors on 5,000-acre site. Supervised protocol-level transect-based surveys for rare plants and invasive weeds, and complete census of two cactus species. Prepared draft and final reports. Reviewed Preliminary Staff Assessment. Providing follow-up information on rare plants and weeds.

Broadwell Lake SEGS, BrightSource Energy; Mojave Desert, San Bernardino County (2009). Field supervisor for large crew of surveyors. Supervised protocol-level transect-based surveys for rare plants and invasive weeds on 11,000-acre site. Estimated the abundance of all cactus species through sampling. Conducted reconnaissance survey of 1-mile-wide buffer. Prepared draft report.

Mormon Mesa SEGS, BrightSource Energy; Northern Mojave Desert, Clark County, Nevada (2008). Co-field supervisor for large crew of surveyors on 6,000-acre site. Supervised protocol-level transect-based surveys for rare plants and invasive weeds. Sampled all cactus species to determine abundance. Prepared draft and final reports. Conducted reconnaissance of adjacent 5,000-acre area.

Lakeville-Sonoma Transmission Line Upgrade, Pacific Gas and Electric Company, Sonoma County (2004-2008). Task leader for rare plant and weed surveys, botanical sections of Negative Declaration, and rare plant and weed mitigation and monitoring plans. Conducted reconnaissance level surveys on 45 miles of proposed routes, and protocol-level surveys on final route. Prepared impact analysis and designed mitigation measures for Negative Declaration. Conducted two-year post-construction monitoring program for rare plants and weeds.

Pittsburg-Tesla Transmission Line Upgrade, Pacific Gas and Electric Company, Contra Costa and Alameda counties. (2008). Completed a habitat assessment and suitability analysis for proposed pull site locations and other features, and conducted protocol-level rare plant surveys at potential impact locations. Wrote results memo; prepared photo appendix.

Sonoma-Marin Area Rail Transit Project, Sonoma and Marin counties (2006 to 2009, ongoing). Conducted reconnaissance surveys and mapped vegetation within the rail corridor and adjacent proposed bike and pedestrian pathway. Conducted two-year protocol-level rare plant surveys for federally and state-listed endangered vernal pool plants within the Santa Rosa Plain segment of the rail corridor, and mapped suitable habitat for these species. Contributed botanical sections to the Biological Assessment prepared for U.S. Fish and Wildlife Service.

Vegetation Management Plan, Marin Municipal Water District, Marin County. (2007-2008). Project area consisted of three watersheds, including the Mt. Tamalpais ecosystem. Wrote the Biodiversity Management Plan, which was part of the Vegetation Management Plan. Provided a status assessment of all rare plants, fish, amphibians, reptiles, birds and mammals. Identified and evaluated threats to all resources. Designed a biodiversity protection and management strategy that includes specific projects to address all types of threats, and improve habitat quality. Participated in a one-day biodiversity symposium on the Mt. Tamalpais ecosystem.

Habitat Conservation Plan/Natural Resources Conservation Plan, Mendocino Redwood Company, Mendocino County (2004-2007). Task leader for rare plant issues. Developed a comprehensive strategy to manage and monitor 40 species of rare plants in the context of active timber harvest operations. Prepared a Rare Plant Survey Handbook that describes acceptable survey and reporting methods.

State Highway 12 Shoulder Widening and Improvements, Caltrans, Solano County (2006-2008). Conducted protocol-level rare plant surveys for three separate phases of the project. Wrote rare plant reports and some botanical sections of the Biological Assessment.

State Highway 12 Laguna Bridge Replacement, Caltrans, Sonoma County (2006-2009, ongoing). Conducted habitat assessments, multiple-year protocol-level rare plant surveys, and a tree survey. Mapped suitable habitat for federally endangered plants. Wrote rare plant reports, and contributed sections for the Biological Assessment and other documents.

DECLARATION OF

Russell Huddleston

I, Russell Huddleston, declare as follows:

1. I am presently employed by CH2M HILL Incorporated as a wetland ecologist.

2. A copy of my professional qualifications and experience are attached hereto and incorporated herein by reference.

3. I prepared the attached testimony on wetlands and other waters for the Ivanpah Solar Electric Generating System project based on my independent analysis, supplements thereto, data from reliable sources, and my professional experience and knowledge.

4. It is my professional opinion that the prepared testimony is valid and accurate with respect to the issue(s) addressed herein.

5. I am personally familiar with the facts and conclusions related in the testimony and if called as a witness could testify competently thereto.

I declare under penalty of perjury that the foregoing is true and correct to the best of my knowledge and belief.

Dated: NOV 10, 2009 Signed: Russell Huddleston

At: Oakland, CA

Russell Huddleston
Wetland Ecologist

Education

M.S., Ecology, University of California, Davis, 2001
B.S., Biology, Southern Oregon University, 1998

Professional Registrations

Professional Wetland Scientist (PWS #1634)

Endangered Species Act Section 10 Scientific Take Permit for Threatened and Endangered Vernal Pool Crustaceans and Selected Rare Plant Species (Permit TE-054120-2)

California Department of Fish and Game Scientific Collectors Permit for Threatened and Endangered Vernal Pool Crustaceans (Permit No. 005934)

California Department of Fish and Game Scientific Collectors Permit for State-listed Threatened and Endangered Plants (Permit No. 08030.1)

Distinguishing Qualifications

- Specialized experience in wetland delineation and assessment
- Specialized experience in rare plant surveys and habitat characterization
- Specialized experience surveys for listed vernal pool invertebrates

Relevant Experience

Mr. Huddleston is a wetland ecologist/botanist in the Environmental Business Group in CH2M HILL's Bay Area office. He has more than 10 years of professional experience in wetland science, plant community classification, habitat assessment, and special-status species surveys. In addition, he has training and experience with global positioning system (GPS) technology used for habitat mapping, wetland delineation, and special-status species surveys.

Mr. Huddleston is a Certified Professional Wetland Scientist and has worked in a variety of wetland types throughout the western United States including Coastal and tundra wetlands in Alaska; vernal pools and seasonal wetlands in California and southern Oregon; mountain streams and seeps in Utah; and desert playas and washes in Arizona, Nevada and Southern California. Mr. Huddleston has also received specialized training in wetland delineation methodology, hydric soils and wetland plants. Mr. Huddleston is a member of the Society of Wetland Scientists and has been a volunteer docent at the Jepson Prairie vernal pool preserve for over 9 years.

Mr. Huddleston has conducted numerous botanical inventories, habitat assessment and characterization studies and surveys for rare, threatened and endangered plant species throughout California in a variety of habitats including coastal sage scrub, valley grasslands, montane forests and the Mojave deserts. He hold scientific collection permits for California State-listed threatened and endangered plants as well as selected federally listed plant species. Mr. Huddleston is an active member of the California Native Plant Society and other professional botanical organizations.

Mr. Huddleston has conducted protocol level surveys for federally-listed vernal pool crustaceans for a variety of clients, including Travis Air Force Base, Camp Pendleton Marine Corps Base, the California Department of Transportation and the Riverside County Transportation Commission. In addition, he has been involved in long-term population monitoring projects for vernal pool species in the Greater Jepson Prairie ecosystem in Solano County, California.

Representative Projects

Ivanpah Solar Electric Generating Project, San Bernardino County, California. Task lead for wetlands and waters delineation for approximately 4,272 acres on an alluvial fan east of the Clark Mountain Range and west of Ivanpah Dry Lake. The entire study area is dissected by numerous ephemeral washes ranging in size from small weakly expressed erosional features to broad drainages with defined bed and bank characteristics.

State Route 79 Realignment Project, Hemet, California. Task lead for wetland delineation surveys for an approximately15-mile highway realignment project. Wetland studies encompassed over 1,800 acres including multiple project alternatives. Wetlands included several problem areas due to seasonal hydrology, strongly alkaline soils and ongoing agricultural practices. Worked in coordination with the U.S. Army Corps of Engineers, the U.S. Environmental Protection Agency and hydric soil specialist to develop procedures to adequately characterized and determine wetlands in the project study area.

Update to Natural Resource Management Plan, Travis Air Force Base, Solano County, California. Conducted an assessment and evaluation of base wide natural resources, including vernal pool habitats, rare plants, and special-status species. Various projects for the Base included vernal pool habitat mapping and assessment, protocol-level surveys for federally-listed vernal pool crustaceans, rare plant surveys, and wetland habitat mitigation monitoring.

On-call Environmental Services, California Department of Transportation, District 4. Provide a variety of environmental support services for highway projects including wetland delineations, rare plant/endangered species surveys, mitigation planning, permitting, and agency coordination.

On-call Environmental Services, Sacramento Municipal Utility District, California. Provided a range of environmental services, including wetland delineations, special-status species surveys, habitat assessment and compliance monitoring as part of the on-call environmental services contract.

Forest Highway 114/Hyampom Road Reconstruction, U.S. Federal Highway Administration, Trinity County, California. As part of the environmental review process, consulted with federal resource agency staff, assisting with rare plant surveys and habitat mapping and classification. Habitat types included Douglas-fir forest, oak woodland and riparian ecosystems. The U.S. Federal Highway Administration in cooperation with the U.S. Forest Service and Trinity County proposed to reconstruct approximately 8.5 miles of Forest Highway in Trinity County, California.

Russell Huddleston

California-Oregon Border Power Plant, People's Energy Resources, Bonanza, Oregon. CH2M HILL was contracted by the California-Oregon Border Power Plant to prepare the Site Certificate Application for submittal to the Oregon Office of Energy. Project related facilities included a nominal 1,150-megawatt generating facility, a 7.2-mile electric transmission line, a 4.1-mile natural gas supply pipeline and a 2.8-mile water supply pipeline. Responsible for coordinating with state and federal resources agencies and conduction habitat mapping, rare plant surveys, and wetland delineations for the proposed project. Natural habitats included sagebrush steppe, juniper woodland, ponderosa pine forest and seasonal wetlands. Vegetation within each habitat was characterized and the habitat was evaluated based on the Oregon Department of Fish and Wildlife's Habitat Classification System.

Sierra Army Depot, U.S. Army Corps of Engineers, Sacramento, California. Conducted an assessment of jurisdictional waters of the U.S. (including wetlands) on an approximately 110-acre site at the Sacramento Army Depot in southern Sacramento County, California. This assessment includes lands to be transferred to the City of Sacramento as part of the Base Realignment and Closure Act.

State Route 153 Roadway Improvement Project, Federal Highway Administration, Beaver, Utah, September 2003. Conducted an assessment of jurisdictional waters of the U.S. (including wetlands for approximately 766 acres along Utah State Highway 153. Wetland delineation was conducted along 11.5 miles of roadway.

In-Delta Storage Project, California Department of Water Resources, Sacramento and Contra Costa Counties. Assisted DWR botanists with rare, threatened and endangered plant surveys in the Sacramento-San-Joaquin Delta. Habitat types included inter-tidal areas, annual grassland, riparian areas and agricultural lands.

Sacramento Municipal Utility District's Cosumnes Power Plant, California. Conducted a wetland delineation for the proposed energy facility site, laydown area, and 26-mile natural gas supply pipeline. Habitat types included annual grassland, seasonal wetlands, vernal pools, and riparian areas.

Proposed Sewer Alignment, Vallejo Flood and Sanitation District, California. Conducted preconstruction plant surveys for special status plant species along a proposed sewer pipeline alignment. Habitat types included inter-tidal marsh, annual grasslands, wet meadows, riparian areas, and wetlands.

Pacific Gas & Electric Line 401 Capacity Loops Project, Pacific Gas & Electric, California. Conducted biological resource surveys including rare, threatened and endangered plant species. Habitat types included mixed conifer forest, sagebrush steppe, seasonal wetlands and riparian areas.

Utah-Nevada Pipeline Project. Task lead for wetland delineation for an approximately 400-mile pipeline from Salt Lake City, Utah to Las Vegas, Nevada for Holly Energy Partners. Delineation included numerous wetlands and other waters including ephemeral washes, lakes, streams and emergent wetlands.

Alaska Department of Transportation Dalton Highway Maintenance Sites. Conducted habitat and wetland assessment of 24 gravel excavation areas for roadway maintenance of the Dalton Highway between Prudhoe Bay and Fairbanks, Alaska.

DECLARATION OF

MARK KUBIK

I, Mark Kubik, declare as follows:

1. I am presently employed by West Yost Associates, Inc. as a Principal Engineer.

2. A copy of my professional qualifications and experience are attached hereto and incorporated herein by reference.

3. I helped prepare the attached testimony on stormwater runoff for the Ivanpah Solar Electric Generating System project based on my independent analysis, supplements thereto, data from reliable sources, and my professional experience and knowledge.

4. It is my professional opinion that the prepared testimony is valid and accurate with respect to the issue(s) addressed herein.

5. I am personally familiar with the facts and conclusions related in the testimony and if called as a witness could testify competently thereto.

I declare under penalty of perjury that the foregoing is true and correct to the best of my knowledge and belief.

Dated: November 12, 2009 Signed: Mark Kubik

At: Davis, CA

Mark O. Kubik, P.E.

Professional Registration

Professional Civil Engineer, 1993
California No. C50963

Education

B.S., Civil Engineering, California Polytechnic State University, San Luis Obispo, 1988

Professional Affiliations

Floodplain Management Association

California Stormwater Quality Association

Mark Kubik has 21 years of experience as project manager and team member of surface water management projects. He has experience developing hydrologic and hydraulic models for storm drainage master plans, floodplain determinations, flood frequency analyses, probable maximum flood studies, dam failure analyses, and reservoir yield studies. He is familiar with many of the commonly used surface water computer models including HEC-RAS, HEC-HMS, SWMM, XPSWMM, UNET, HEC-DSS, HEC-FFA and others. He also has experience with two-dimensional hydraulic modeling. Other experience includes detention and pipeline design, storm water quality analysis, and infrastructure design for residential and commercial developments.

PROJECT EXPERIENCE

Storm Drainage Master Planning

Storm Drainage Master Plan. Project manager for the City of Elk Grove's City-wide Storm Drainage Master Plan. The City covers approximately 27,000 acres in Sacramento County and is served by a drainage system that includes 400 miles of underground pipelines, 60 miles of creeks or channels, and a number of detention basins. The City's rapid population growth triggered the need for a comprehensive storm drainage master plan to help the City protect its residents from flooding and to develop a Capital Improvement Plan. For this project, Mark performed and managed the analyses to identify existing system deficiencies, define floodplain limits, defined future facility needs, and determined facility costs for use in establishing development fees. The analyses were performed using HEC-1, HEC-RAS, XP-SWMM, and ArcGIS. *City of Elk Grove, California.*

City of Sacramento Storm Drainage Master Plans. Managed the preparation of a storm drainage master plan for Basins 22 and 108. The analysis included an evaluation of the existing drainage systems to determine the adequacy and reliability of the systems. Alternatives were developed to upgrade the performance of the systems economically. The hydraulic analysis was performed with the Sacramento Storm Water Management Model (SWMM), which performs unsteady-state flow calculations. Recently completed similar studies for City of Sacramento Basins 26, 67, 68, 69, and 139. *City of Sacramento, California.*

Drainage Master Planning. Performed the hydrologic and hydraulic analyses for the drainage master planning in the Sacramento areas listed below. The work included the analysis and design of drainage channels, flood control detention basins, storm water quality detention basins, culverts, and storm drainage pipe systems. The hydraulic analyses were performed with the use of HEC-1, HEC-2, SACPRE, HEC-RAS, HEC-DSS, and other computer programs. Also developed cost estimates for use in the Capital Improvement Programs. Locations included:

North Vineyard Station specific plan area (Elder & Gerber Creeks)

East Elk Grove specific plan area (Elk Grove Creek and tributaries to Laguna and Elk Grove Creeks)

East Antelope specific plan area (Tributary to Dry Creek)

Middle Branch of Strawberry Creek

North Natomas community plan area (Drainage sheds 1 and 2)

Basin 157 Storm Drainage Master Planning. Project manager for a storm drainage master plan for Basin 157, which covers nearly 2,800 acres in the City of Sacramento. The basin includes large areas of both developed and undeveloped land. For the developed areas, the master plan defined those portions of the existing drainage system that do not meet the City's drainage criteria and identified the required facility upgrades. For the undeveloped areas, the drainage facilities necessary to safely convey runoff for the anticipated buildout conditions were identified. Developed an unsteady-state Sacramento SWMM model of the drainage system and used it to establish existing drainage conditions within the watershed, predicted the potential effects of future development, and evaluated alternative drainage improvement projects. Prepared implementation cost estimates for each alternative and established the implementation priorities for the recommended improvements. *City of Sacramento, California.*

Reclamation District 784 Drainage Master Plan. Managed the preparation of a drainage master plan for Reclamation District 784. Flood hydrographs were calculated for existing and ultimate conditions using HEC-1 and water surface profiles were calculated using the unsteady state UNET computer model. Alternative flood control plans were developed to handle the increased runoff anticipated from future developed. Preferred alternatives were developed and estimates of construction, operation, and maintenance costs were prepared. A presentation of the master plan was delivered at a public meeting. *Reclamation District 784, Yuba County, California.*

Madison and Esparto Flood Control. Managed the evaluation of flood control alternatives for the communities of Madison and Esparto in Yolo County. Alternatives included channel improvements, flood walls, levees, flow diversions, and the raising of roads and buildings. The study also consisted of a peer review of a previously developed flood control plan for Madison. Hydraulic analyses of the alternatives were performed with HEC-1 and HEC-2. Cost estimates were prepared for each alternative. The results of the study were summarized in a report and were presented at a public meeting and a board of supervisors hearing. *Yolo County Public Works Department, Yolo County, California.*

Mather Office Campus Project. Project engineer for a redevelopment project in the Mather Field area of Sacramento County. Storm runoff in the redevelopment area is conveyed through a long box culvert under a roadway to a small channel that begins just downstream of the project area. A floodplain study for the area was originally performed in the late 1990's, and subsequent changes were made to the drainage facilities and surface elevations in the area. To determine the effects of the changes, WYA performed hydrologic and hydraulic modeling using HEC-1 and XP-SWMM. WYA used the models to define the existing floodplain limits and to determine if the Mather Office Campus Project was sufficiently protected from potential flooding. *Sacramento Housing & Redevelopment Agency, California.*

Hydrologic and Hydraulic Analyses

Napa Interior Drainage Study. Managed the preparation of updated interior flooding conditions behind proposed Corps of Engineers levee improvements along the Napa River. In coordination with the local Flood Control District and the Corps, WYA performed hydrologic and hydraulic analyses to insure that the final Corps project did not result in increased interior flooding at any location. WYA also prepared design plans for channel and culvert improvements to improve the interior drainage conditions. *Napa County Flood Control and Water Conservation District, Napa, California.*

Gasser Property Storm Drainage. Managed the preparation of design plans and specifications for a project to reduce stormwater ponding between the old and new alignments of the Napa Valley Wine Train near Imola Avenue in the City of Napa. WYA identified an open ditch and culvert project that would solve the problem for less cost than the original pipeline cost. For this project, WYA also performed a peer review of a storm drain pipeline design by others for the Gasser Property. *Napa County Flood Control and Water Conservation District, Napa, California.*

Peer Review of Drainage Plan for Napa Valley Wine Train Relocation. Performed a peer review of the proposed storm drainage facilities proposed with the Napa Valley Wine Train Relocation Project. After preparation of the 65 percent plans for the project, TranSystems received questions about whether the project would worsen the drainage on private properties in the area. Based on our experience on other projects in the area, WYA was able to assist TranSystems by providing data that facilitated the preparation of a detailed drainage study. WYA provided a peer review of the study and made recommendations that improved the efficiency of the proposed system. *TranSystems Corporation, Oakland, California.*

Ivanpah Solar Electric Generating Facility – Stormwater Runoff and Sediment Transport Analysis. Prepared a storm drainage study for a proposed 400 megawatt solar power project covering 4,000 acres in the Ivanpah Valley of the Mojave Desert in California. The project is situated on an alluvial fan located at the base of the Clark Mountain Range. Flood flows from the mountains are initially confined in incised channels but once arriving on the alluvial fan the flood flows are less confined and can take random paths across the fan. To analyze this complex problem, Mark prepared a two-dimensional flood model using Flo-2D. With this model, WYA was able to show that a low impact approach to the project design could be accomplished with minimal impacts on flood flows and sediment transport in the watershed. This approach allowed the client to significantly reduce the cost of the project by eliminating large detention basins and channels that had originally been proposed by others. *BrightSource Energy, Inc., Oakland, California.*

Alamo Creek LOMR. Currently preparing a two-dimensional hydraulic model to define the 100-year floodplain cause by spill out of Alamo Creek in the City of Vacaville. Current FEMA maps do not provide floodplain elevations and do appear to depict reasonable floodplain limits within a portion of the City that is subject to flooding from Alamo Creek. The two-dimensional model will be used as the basis of a Letter of Map Revision application to be submitted to FEMA.

Point Pleasant Flood Control. Managed the evaluation of flood control alternatives for the Point Pleasant community in southern Sacramento County. Alternatives included a dry dam on the Cosumnes River, conversion of an agricultural island into a flood storage basin, and construction of a ring levee around the community. Hydraulic analyses of the alternatives were performed with an unsteady-state HEC-RAS model covering the complex North Delta region. Cost estimates were prepared for each alternative. The results of the study were summarized in technical memoranda and presented at a public workshop. *County of Sacramento, California.*

Hazel Avenue. Managed the hydraulic analysis for the Hazel Avenue bridge-widening project over the American River in Sacramento County. The review consisted of hydraulic calculations with HEC-RAS to evaluate the potential increase in water surface elevations and velocities for several alternative bridge configurations. Potential scour at the bridge was estimated using the Federal Highway Administration's "Hydraulic Engineering Circular No. 18, Evaluating Scour at Bridges" as implemented in HEC-RAS. *Parsons Brinckerhoff, Sacramento, California.*

North Stockton Railroad Grade Separation Project. This active project includes construction of three new roadway overpasses, one new underpass, and replacement of two bridges in northern Stockton. Mark is managing the hydrologic and hydraulic analyses to define the required roadway drainage and stormwater quality treatment facilities. This includes design of a pump station for the underpass on Lower Sacramento Road. WYA is performing the hydraulic analysis of the proposed bridges to insure that they are appropriately designed so as not to increase the risk of flooding along the creeks. The hydraulic analysis will include an evaluation of the scour potential at the bridges to assist with bridge design. *Mark Thomas & Company, Sacramento, California.*

EIR Support

City of Chico Storm Drainage Master Plan. In support of an EIR for the City of Chico storm drainage master plan, performed a review of the hydrologic and hydraulic analyses used to develop the master plan. A flood frequency analysis of historic flow data was performed in order to verify the flows developed for the master plan. Inadequacies of the previous analyses were identified, and solutions were recommended. *City of Chico, California.*

FEMA Letter of Map Change Applications

Prepared and processed FEMA Letter of Map Change applications for the following projects:

McClellan Park (CLOMR for Magpie Creek, Sacramento County, California)

Ivywood Subdivision (LOMR for Ulatis Creek, City of Vacaville, California)

Chestnut Subdivision (CLOMR for Linda Drain, Yuba County, California)

The Price Club of South Sacramento (LOMR/Floodway Revision for Union House Creek, Sacramento County, California)

Tolman Acres (LOMA - Dry Creek, Sacramento County, California)

Oakcreek Cove (LOMA - Arcade Creek, Sacramento County, California)

Creekview (CLOMR – Laguna Creek, Sacramento County, California)

Park Meadows (CLOMR – Laguna Creek, Sacramento County, California)

Silver Springs North (CLOMR – Laguna Creek, Sacramento County, California)

Sanitary Sewer

Sanitary Sewer Master Plans. Prepared regional sanitary sewer master plans for the following planning areas within Sacramento County, California:

Elk Grove-West Vineyard Area

East Antelope Specific Plan Area

Infrastructure Improvement Plan Design

Performed improvement plan design, including the design of storm drainage pipe systems and channels, sanitary sewer systems, water distribution facilities, street lighting systems, and grading plans. Developed cost estimates and bid documents. The following is a list of specific projects, which are all located in Sacramento County, California:

Northbrook Units 3, 4, and 5

Larchmont Antelope Creek Units 1 and 2

Antelope Marketplace Shopping Center

The Cottages at Antelope Park

Oakcreek Cove

DECLARATION OF

Thomas A. Lae

I, Thomas A. Lae, declare as follows:

1. I am presently employed by CH2M HILL Incorporated as a Geologist.
2. A copy of my professional qualifications and experience are attached hereto and incorporated herein by reference.
3. I prepared the attached testimony on Geologic Hazards and Resources for the Ivanpah Solar Electric Generating System project based on my independent analysis, supplements thereto, data from reliable sources, and my professional experience and knowledge.
4. It is my professional opinion that the prepared testimony is valid and accurate with respect to the issue(s) addressed herein.
5. I am personally familiar with the facts and conclusions related in the testimony and if called as a witness could testify competently thereto.

I declare under penalty of perjury that the foregoing is true and correct to the best of my knowledge and belief.

Dated: 11-6-09 Signed: T. A. Lae

At: Sacramento, CA

Thomas Lae, P.G.
Geologic Hazards and Resources Task Lead

Education

B.S., Geology

Professional Registration

State of California Professional Geologist, License No. 7099

Relevant Experience

Mr. Lae has more than 18 years of experience in environmental geology and project management and is a California Professional Geologist. Mr. Lae works on numerous projects for a variety of private, federal, and municipal clients and has an extensive background in environmental field investigations. Projects include: geologic hazards and resources section preparer for numerous power plant licensing projects, Superfund site investigation oversight, remedial investigations/feasibility studies, underground storage tank/oil water separator closures, landfill groundwater monitoring, and phase II environmental assessments.

Representative Projects

BrightSource Energy, Ivanpah Solar Electric Generating System, California. Authored the Geologic Hazards and Resources section for the licensing of a 400 MW concentrating solar plant using a proprietary Distributed Power Tower (DPT) technology. The approximate 4,060-acre project is located in the Mojave Desert on land managed by the Bureau of Land Management (BLM). Due to BLM's involvement, the environmental analysis needed to comply with both CEQA and NEPA. An Application for Certification was prepared under the California Energy Commission (CEC) process.

Electrical Power Plant Application for Certification section preparer. Mr. Lae has prepared Geologic Hazards and Resources sections for 22 AFCs. These include East Altamont Energy Center (Calpine), Central Valley Energy Center (Calpine), Los Esteros Energy Center (Calpine), Cosumnes Power Plant (SMUD), Woodland II (Modesto Irrigation District), Modesto Electric Generation Station (Modesto Irrigation District), Walnut Energy Center (Turlock Irrigation District), San Francisco Electrical Reliability Project (San Francisco Public Utilities Commission), Highgrove (AES Pacific),Walnut Creek Energy Project (Edison Mission Energy), Sun Valley Energy Project (Edison Mission Energy), Eastshore Energy Project (Tierra), South Bay Energy Facility (Duke), Chevron Richmond Power Plant Replacement Project SPPE, Ivanpah Solar Electric Generating System (Bright Source Energy), Carlsbad Energy Center Project (NRG), Tracy Power Plant (GWF), Vacaville Energy Center (Competitive Power Ventures), Lodi Energy Center (NCPA), Contra Costa Generating Station (Radback Energy), and Mariposa Energy Project (DGC). Mr. Lae is well versed in the assessment of geologic resources and hazards relating to CEQA and NEPA requirements.

Thomas Lae, P.G.

California Energy Commission Hazardous Waste Remediation Oversight. A part of the PG&E Gateway Generating Station construction (Antioch, CA) and Colusa Generating Station (Colusa, CA), Mr. Lae served as the project's on-call Professional Geologist. His duties included the coordination of sampling, characterization, and remediation of hazardous waste materials (asbestos, PCBs, and/or TPH) encountered during plant excavation activities. Mr. Lae provided summary reports upon completion of remedial activities for submittal to the CEC.

Superfund Site Investigation Oversight. CH2M HILL provides oversight support to the USEPA for six task orders, with Mr. Lae serving as project manager. This project involves the review and comment of reports, white papers, technical memoranda, and studies that are submitted for regulatory review. This facility that has been impacted by solvent, fuel, propellant, and metals contamination in soil, soil gas, and groundwater.

Union Pacific Railroad. Mr. Lae serves as the project manager for four UPRR projects that include: a groundwater and soil TPH investigation at a former UST site (Donner Summit UST); and an arsenic in soil assessment at a Right of Way (Clyde, California). Mr. Lae successfully received regulatory closure including a TPH in soil site at Right of Way (Chico, California), and nitrogen contamination in onsite soils (Willows, California).

Groundwater Study/Well Decommissioning. Mr. Lae served as the project manager for TO 467 at Beale AFB. This project involved the installation of groundwater monitoring wells and the collection of groundwater samples to assess the effects of potential impact to the underlying groundwater from a retention pond that receives treated waste water. In addition, this project required the destruction of several former water/agricultural supply wells at the base per County and State destruction protocol.

Soil Vapor Extraction System Termination. Mr. Lae served as the project manager for the IC27 STOP project at the former McClellan AFB. This project involved the collection of soil gas samples and the preparation of report documentation to support the SVE system termination (closure). The project successfully met regulatory criteria and system termination was granted. The project also required the decommissioning of the system wells and conveyance pipelines.

Superfund Site Investigations. Mr. Lae serves as the project manager for the Lava Cap Mine site in Nevada City, California. This project is a site that has been affected by arsenic contamination from past gold mine processing and is undergoing Feasibility Study evaluations for remedial alternatives.

Oil /Water Separator Closure Investigation. Mr. Lae served as the project manager for three projects at Beale AFB in the evaluation for regulatory closure of 25 former oil/water separators across Beale. The project included the assessment of environmental impacts to underlying soil and groundwater from past releases and preparing closure documentation. Mr. Lae has successfully received closure of 23 OWSs. Two OWSs are undergoing biovent remediation prior to closure.

DECLARATION OF

Steven P. Long

I, Steven P. Long, declare as follows:

1. I am presently employed by CH2M HILL Incorporated as a Project Scientist.
2. A copy of my professional qualifications and experience are attached hereto and incorporated herein by reference.
3. I helped prepare the attached testimony on Soils for the Ivanpah Solar Electric Generating System project based on my independent analysis, supplements thereto, data from reliable sources, and my professional experience and knowledge.
4. It is my professional opinion that the prepared testimony is valid and accurate with respect to the issue(s) addressed herein.
5. I am personally familiar with the facts and conclusions related in the testimony and if called as a witness could testify competently thereto.

I declare under penalty of perjury that the foregoing is true and correct to the best of my knowledge and belief.

Dated: November 9, 2009 Signed: Steven P. Long

At: Sacramento, California

Steve Long
Soils Task Lead

Education

M.S., Soil Science
B.S., Forest Resources

Relevant Experience

With over 20 years of professional experience as an environmental scientist, Mr. Long is responsible for a wide range of tasks associated with natural resource and hydrogeologic environmental evaluations and permitting. Duties include permit planning, preparation, and consultation with resource agencies, as well as supervision of field data collection, interpretation, and report preparation for contaminant and environmental quality assessments. Has participated in numerous investigations and risk assessments for terrestrial, aquatic, and stormwater pathway impacts for Superfund and Department of Defense projects.

Hydrogeological investigations experience includes in-field testing of soil, soil gas and groundwater samples using portable gas chromatograph; in-situ aquifer permeability testing; and monitoring subsurface explorations and installations (monitoring wells, piezometers and vapor extraction systems). Remediation experience includes managing bioventing systems and supervising site clean-ups. Strong skills in environmental sampling and testing.

Natural resource experience includes evaluations of wetland, riparian, forest, and agricultural systems. Duties have included delineation and documentation of wetlands by federal and state criteria in California, Nevada, Washington, Connecticut, Massachusetts, New York, New Hampshire, and Maine; evaluation of project constraints and development of alternate strategies for local, state, and federal permitting. Strong skills in soil description and taxonomic classification, vegetation, and insects; permitting of wetland activities; and statistical data analyses.

Representative Projects

Ivanpah Solar Electric Generating System, San Bernardino County, California. Provided senior review for AFC section that assessed potential impacts to soil and agricultural resources for the proposed power plant project which encompassed approximately 3,800 acres in the Mojave Desert. Provided additional support for wind and water soil loss estimates used to estimate needs for construction water use and maintenance of detention pond facilities.

Chula Vista Energy Upgrade Project, MMC Energy, San Diego County, California. Prepared CEQA-equivalent documentation to support an Application for Certifications (AFC) for review by the California Energy Commission. Prepared AFC section that assessed potential impacts to soil and agricultural resources for the proposed power plant projects This documentation included a summary of applicable laws, ordinances, and regulations

(LORS), estimates of soil losses from wind and water erosion during construction, and agencies contacts.

Humboldt Bay Replacement Project, PG&E. Planned and executed the Phase II ESA using staff from a minority owned 'mentor-protégé' firm. Prepared the Phase II ESA cost proposal and work plan. Coordinated the field sampling activities and prepared the report. Met with client and regulator from the North Coast Regional Water Quality Board, where we garnered approval for our final recommended site investigation tasks to complete the Phase II ESA. Provided senior review for AFC section that assessed potential impacts to soil and agricultural resources for the proposed power plant project.

South Bay Replacement Project, LS Power. Prepared CEQA-equivalent documentation to support an Application for Certifications (AFC) for review by the California Energy Commission. Prepared AFC section that assessed potential impacts to soil and agricultural resources for the proposed power plant projects including all linear features (transmission lines, water supply and discharge lines, and natural gas supply lines). Also prepared section for waste management that described demolition, construction, and operation waste streams. This documentation included summaries of applicable laws, ordinances, and regulations (LORS) and agencies contacts. It also included estimates of soil losses from wind and water erosion during construction and mitigation and management strategies.

Eastshore Energy Center, Tierra. Prepared CEQA-equivalent documentation to support an Application for Certifications (AFC) for review by the California Energy Commission. Prepared AFC section that assessed potential impacts to soil and agricultural resources for the proposed power plant projects including all linear features (transmission lines, water supply and discharge lines, and natural gas supply lines). This documentation also included a summary of applicable laws, ordinances, and regulations (LORS), estimates of soil losses from water erosion during construction, and agencies contacts.

Application for Certification, Los Esteros Critical Energy Facility, Calpine C*Power, San Jose, California. Prepared Biological Resources Mitigation and Monitoring Plan (BRMIMP) for the Los Esteros Critical Energy Facility. Also documented the extent of jurisdictional waters of the U.S. at a stormwater outfall along Coyote Creek. Prepared a Low Effect Habitat Conservation Plan for the Phase II Facility. This plan was submitted for Section 10 consultation with the U.S. Fish and Wildlife Service to secure an incidental take permit for Bay Checkerspot butterfly and to offset potential impacts to four endemic serpentine plants under the Endangered Species.

Application for Certification, East Altamont Energy Center, Calpine Corp., Tracy, California. Prepared CEQA-equivalent documentation to support an Application for Certifications (AFC) for review by the California Energy Commission. Prepared AFC section that assessed potential impacts to soil and agricultural resources for the proposed power plant projects including all linear features (transmission lines, water supply and discharge lines, and natural gas supply lines). This documentation also included a summary of applicable laws, ordinances, and regulations (LORS), estimates of soil losses from wind and water erosion during construction, and agencies contacts. Additionally, conducted field investigations to assess wetlands in proximity to linear routes for the East Altamont Energy Center.

DECLARATION OF

Sarah Madams

I, Sarah Madams, declare as follows:

1. I am presently employed by CH2M HILL Incorporated as a Project Manager.
2. A copy of my professional qualifications and experience are attached hereto and incorporated herein by reference.
3. I prepared the attached testimony on Hazardous Materials and Waste Management for the Ivanpah Solar Electric Generating System project based on my independent analysis, supplements thereto, data from reliable sources, and my professional experience and knowledge.
4. It is my professional opinion that the prepared testimony is valid and accurate with respect to the issue(s) addressed herein.
5. I am personally familiar with the facts and conclusions related in the testimony and if called as a witness could testify competently thereto.

I declare under penalty of perjury that the foregoing is true and correct to the best of my knowledge and belief.

Dated: November 13, 2009 Signed: SMadams

At: Sacramento, CA

Sarah Madams
Hazardous Materials Management and Waste Management Task Lead

Education

B.S., Environmental Toxicology

Relevant Experience

Ms. Madams has more than 11 years of professional experience including project management, regulatory compliance, permitting, public involvement/community relations, data collection and analysis, database management, compliance audits, document preparation, and technical writing. For the last 6 years, Ms. Madams has served as the Deputy Project Manager for power plant licensing work performed by CH2M HILL, and is serving as the Project Manager for the Lodi Energy Center. Her expertise includes working with multidisciplinary teams to assess the environmental impacts of power plant projects on the environment. These assessments include impacts to air, biological and cultural resources, land uses, noise, socioeconomics, public health, water and visual resources, soils and geology, and paleontology.

Representative Projects

Hazardous Materials and Waste Management Task Leader; Ivanpah Solar Electric Generating System; Eastern San Bernardino County; California. Hazardous Materials and Waste Management Task Leader for preparation of the Hazardous Materials and Waste Management sections of the AFC to the CEC for a 400 MW solar energy power generation facility using heliostat fields to focus solar energy on power tower receivers. Responsible for evaluating hazardous materials used and stored at the facility, as well as waste management issues associated with the project site . AFC submitted to the CEC in September 2007 and currently going through CEC AFC processing.

Lodi Energy Center, NCPA, San Joaquin County, California. Project Manager for the licensing of this 255-MW combined cycle power plant. Managed a multidisciplinary team of scientists, planners, and engineers in preparing and filing the license application. Submitted FAA Form 7460s and notice criteria tools to FAA. Coordinated efforts between CEC project management, local and state agencies and CH2M HILL staff.

Chula Vista Energy Upgrade Project, MMC Energy, San Diego County, California. Deputy Project Manager for the AFC for a 100-MW power plant. Prepared and provided testimony on the waste management, alternatives, worker health & safety and hazardous waste sections of the AFC.

Russell City Energy Center Amendment, Calpine, Alameda County, California. Deputy Project Manager for the AFC for a 600-MW power plant. Prepared and provided written testimony for the waste management, alternatives, worker health & safety and hazardous waste sections of the AFC. Coordinated biological and cultural surveys of the project area. Submitted FAA Form 7460s and notice criteria tools to FAA. Addressed multidisciplinary issues received from state and local agencies. Attended public workshops and hearings.

Sarah Madams

Application for Certification, Los Esteros Critical Energy Facility, Calpine C*Power, San Jose, California. Project Coordinator for the AFC for a 180-MW power plant. The project required the preparation of numerous other studies/documents to satisfy the CEC staff request. These studies/documents included the preparation of a General Plan amendment and planned development zoning applications, archaeological and paleontological survey reports, and biological resource protection permits. Ms. Madams assisted with the development and implementation of biological, cultural, and paleontological resource monitoring programs; risk management plan; and traffic and transportation management plan. The plant is currently in operation.

Small Power Plant Exemption, MID Electric Generation Station (MEGS), Modesto Irrigation District, California. Project Coordinator for the SPPE for a 95-MW peaking plant. She reviewed applications, coordinated multidisciplinary data requests and responses, and served as liaison and coordinated efforts between CEC project management and staff.

Application for Certification, Walnut Energy Center, Turlock Irrigation District, California. Project Coordinator for the AFC for a 250-MW combined cycle power plant. She reviewed applications, coordinated multidisciplinary data requests and responses, and coordinated efforts between CEC project management and CH2M HILL staff. Ms. Madams assisted with the development of the security plan and emergency response plan. The plant is currently in operation.

Application for Certification, Salton Sea Unit 6 Geothermal Power Plant, Mid-American Energy Holding Company, Imperial County, California. Project Coordinator for the licensing of the 185-MW geothermal power plant. The power plant design was based on the flash geothermal power plant process, which produces both solid and liquid byproducts that required disposal. The project site was in a rural area of Imperial County, but was adjacent to a National Wildlife Refugee that supports significant populations of avian species. The licensing process involved the review of all environmental areas, and specifically focused on waste disposal, air quality, hazardous materials handling, and biological resources. Ms. Madams was responsible for the development and tracking of data response submittals requested by the CEC. The project was successfully completed, with a license issued by the CEC.

Various Power Plant Applications for Certification (AFCs) – Prepared or assisted on the Worker Health and Safety, Hazardous Materials, and Waste Management sections. In addition prepared Field Safety Instructions, Health and Safety Plans and served as the Site Safety Coordinator for the following power plant Applications for Certification:

- GWF Tracy Combined Cycle Power Plant
- Eastshore Energy Center
- Carlsbad Energy Center
- San Francisco Electric Reliability Project
- Walnut Creek Energy Park
- Sun Valley Energy Project
- Confidential Southern California Power Project

DECLARATION OF
THOMAS PRIESTLEY, Ph.D.

I, Dr. Thomas Priestley, declare as follows:

1. I am presently employed by CH2M HILL Incorporated as a Senior Technologist.
2. A copy of my professional qualifications and experience is attached hereto and incorporated by reference herein.
3. I am adopting the attached testimony on Visual Resources for the Ivanpah Solar Electric Generating System project based on my independent analysis and my professional experience and knowledge.
4. It is my professional opinion that the prepared testimony is valid and accurate with respect to the issue(s) addressed herein.
5. I am personally familiar with the facts and conclusions related in the testimony and if called as a witness could testify competently thereto.

I declare under penalty of perjury that the foregoing is true and correct to the best of my knowledge and belief.

Dated: November 11, 2009 Signed: Thomas J. Priestley

At: Oakland, CA

Thomas Priestley, Ph.D., AICP/ASLA
Senior Technologist

Education and Training

Ph.D., Environmental Planning, University of California, Berkeley, 1988
M.L.A., Environmental Planning, University of California, Berkeley, 1976
M.C.P., City Planning, University of California, Berkeley, 1974
B.U.P., Urban Planning, University of Illinois, 1969

Professional Affiliations

American Institute of Certified Planners
American Planning Association
American Society of Landscape Architects

Relevant Experience

Dr. Priestley serves as the leader of the firm's visual resources practice group. In this role, Dr. Priestley guides the company's visual resources work through issue scoping, development of study designs, mobilization of staff and technologies appropriate to the assignment, guidance of analysis activities, and senior review of final products. In addition, Dr. Priestley consults directly in cases that require special visual resources expertise and he provides expert witness testimony when requested.

Dr. Priestley has more than 30 years of professional experience in urban and environmental planning and project assessment. He is known nationwide for his expertise in evaluating aesthetic, land use, property value, and public acceptance issues related to infrastructure facilities. His experience includes projecting community land use development trends to determine facility needs and optimal location; assessing land use and visual effects of proposed facilities; and conducting studies of public perceptions of project visual effects. Dr. Priestley is skilled in scoping aesthetic and urban design issues related to projects and in developing and implementing the analyses appropriate to address them as part of project assessments.

Dr. Priestley is skilled in scoping aesthetic and urban design issues related to a wide range of large-scale projects and in developing and implementing the analyses appropriate to address them as part of project assessments. Dr. Priestley has led efforts to prepare environmental assessment documents in response to the requirements of the NEPA, CEQA, the Bureau of Land Management Visual Resource Management System, the U.S. Forest Service Scenery Management System, the Federal Energy Regulatory Commission, and the California Energy and Public Utilities Commissions.

Representative Projects

Visual Resource Impact Analyses of Gas-fired Power Plants, Various Clients, Various Locations, California. Evaluated potential visual resources impacts of more than 25 gas-fired power plants proposed for a variety of urban and rural settings in California. Identified visual issues, designed the analysis strategies, contributed to development of architectural and landscape treatments, prepared visual resources analyses for the Applications for Certification for submittal to the California Energy Commission, reviewed and critiqued relevant sections of the Energy Commission's analyses of the projects, and evaluated the visual issues associated with CEC-proposed alternative sites. As an expert witness on visual resources, prepared written testimony and provided oral testimony in hearings before the California Energy Commission.

Ivanpah Solar Electric Generating System, San Bernardino County, CA. Senior reviewer for the AFC visual resource analysis prepared for a solar thermal project proposed by Bright Source for development on 3,400 acres of Federal land managed by the BLM that are located in the desert region of eastern San Bernardino County, approximately 5 miles southwest of Primm, NV.

Silver State Photovoltaic Power Project, Clark County, NV. As the Senior Consultant, now preparing the Federal EIS visual resource assessment for a proposal by NextLight to develop a photovoltaic power plant on 7,840 acres of Federal land managed by the BLM that are located immediately east of Primm, NV.

Rice Solar Energy Project, Riverside County, CA. . Senior reviewer for the AFC visual resource analysis prepared by CH2M HILL's visual resources staff for a solar thermal project proposed by Solar Reserve for development on 3,325 acres of privately owned land on the site of the former Rice Army Airfield in the Mojave Desert region of eastern Riverside County.

AT&T Solar Pilot Initiative, Analysis of Potential Visual Effects, San Ramon, California. Analyzed the potential aesthetic effects of a 1.1 MW photovoltaic electric generation system proposed for installation on the roof of the AT&T headquarters building. Identified and photo documented views from sensitive viewing areas and directed production of visual simulations to depict the appearance of the installed PV system. Prepared a report that presented the simulations, evaluated the project's effects on the views and addressed concerns about the potential for the system to create glare effects.

Eldorado to Ivanpah 220 kV Transmission Line, Proponent's Environmental Assessment, San Bernardino County, CA and Clark County, NV. Provided senior support and review for the preparation of the PEA visual resources impact analysis of a proposal by SCE to develop a new 36-mile 220 kV transmission line between the Eldorado Substation and a new Ivanpah Substation located in eastern San Bernardino County, CA, 7 miles southwest of Primm, NV.

Tehachapi Renewables Transmission Project, Proponent's Environmental Assessment, Southern California. Technical lead for the analysis of a 190 mile, 500-kV transmission line being proposed by Southern California Edison. The route traversed a diverse and complex set of landscapes that include open desert lands in the Antelope Valley, National Forest lands in the San Gabriel Mountains valued for their recreational and scenic importance, and highly developed urban areas in the San Gabriel Valley. Designed the analysis strategy that was implemented by a team of five CH2M HILL visual resource specialists, who were supported by CH2M HILL planners and GIS, visual simulation, graphics, and report production staff.

DECLARATION OF
TOM REAGAN

I, Tom Reagan, declare as follows:

1. I am presently employed by Bright Source Energy, Incorporated as a Development Engineer.
2. A copy of my professional qualifications and experience is attached hereto and incorporated by reference herein.
3. I am adopting the attached testimony on Project Description, Engineering, Soils and Water Resources for the Ivanpah Solar Electric Generating System project based on my independent analysis and my professional experience and knowledge.
4. It is my professional opinion that the prepared testimony is valid and accurate with respect to the issue(s) addressed herein.
5. I am personally familiar with the facts and conclusions related in the testimony and if called as a witness could testify competently thereto.

I declare under penalty of perjury that the foregoing is true and correct to the best of my knowledge and belief.

Dated: November 16, 2009 Signed: Tom Reagan

At: Sacramento, CA

THOMAS M. REAGAN

6008 Auburn Avenue
Oakland, CA 94618
415.608.3291 cell
510.653.4514 home
thomasmichaelreagan@hotmail.com

OBJECTIVE: SENIOR INFRASTRUCTURE MANAGEMENT FOR LARGE SCALE DEVELOPMENTS

EXPERIENCE

DIRECTOR OF INFRASTRUCTURE, WILSON MEANY SULLIVAN LLP, JANUARY 2007-NOVEMBER 2008
REAL ESTATE DEVELOPER, SAN FRANCISCO, CA.

Director of infrastructure responsibilities: Negotiating and procuring project permits from government agencies, civil design and final mapping oversight, Public Agency approval of construction drawings, negotiate for delivery of utilities to site, consultant and construction contracts, pro forma budgets, schedules and all staffing necessary for the construction of project site civil improvements.

Bay Meadows Development

San Mateo, CA

Conversion of 85 acre former horse racing facility, this site is located between Silicon Valley and San Francisco. New mixed use, Transit Oriented Development, including a Caltrain station, 1,250 homes, 850,000 sf of office space

Accomplishments as Director of Infrastructure:

- Successfully procured infrastructure related entitlements and permits from City and County of San Mateo, State and Federal agencies.
- Coordinated civil engineering design team.
- Negotiated with utility agencies for power and other utilities to site.
- Led in the formation of Community Funding District (CFD) for public infrastructure bond financing.
- Drafted and issued RFP's and contracts for contractors and consultants.

Demolition started in September 2008. Subsequent infrastructure construction, which was to start in October 2008, has been delayed indefinitely due to difficulties obtaining financing.

Treasure Island Development

San Francisco, CA

This 400 acre development, which will be owned by the City of San Francisco, will require in excess of $700 mil of infrastructure improvements to support the 6,000 planned residences, 800,000 sf of retail and a 50 story hotel. Our firm was chosen, by City, to be the lead developer of the island.

Accomplishments as Director of Infrastructure:

- Coordinated with development team partners and consultants and led infrastructure team in procuring entitlements.
- Drafted overall schedule for entitlements deliverables and construction phasing.
- Created pro forma budgets for infrastructure and entitlement costs.
- Coordinated with consultants in the drafting of EIR.
- Coordinated with SF PUC, PG&E in coordinating the routing, design and delivery of power, communication and other utilities to island.
- Negotiated with Caltrans for design and route of entry and exit ramps for island from Interstate 80.

Design drawings and construction start date have been put on hold pending the land title transfer from US Navy to City of San Francisco.

AREA MANAGER- GRANITE CONSTRUCTION COMPANY, 2004-2007
Managed estimating and project management for San Francisco East Bay Region, staff included twelve managers and engineers and up to 200 craft workers.

Major Project: Stonebrae Country Club, Residential and Golf Community-Hayward, CA $80,000,000

1600 acre master planned community located in the East Bay Hills. Contract included relocation of existing power lines, installation of water, sanitary, storm drain, gas and electrical systems. Extensive grading was required for the construction of roads, 750 house pads, and PGA certified golf course, designed by David McKay Kidd.

- Estimated and negotiated contract for all infrastructure improvements.
- Managed project team.
- Extensive coordination with City of Hayward, County of Alameda, EBMUD and PG&E to achieve aggressive construction schedule.

SENIOR LEAD ESTIMATOR, GRANITE CONSTRUCTION COMPANY, 1997-2004
HEAVY CONSTRUCTION DIVISION (HCD), WATSONVILLE, CA.
Granite Construction is a national leader in heavy civil infrastructure construction. Majority of projects bid were design/build.

Lead Estimator Responsibilities:

- Managed team of internal estimators typically five to six, and the external engineering design consultants.
- Oversee writing, assembly and submission of bid proposal.
- Assist Owner Agency in procurement of any permits necessary for construction.

Successful Design/Build Proposals:

Reno RETRAC, Reno, NV, Design/Build $171,000,000 Relocation of Southern Pacific railway from at grade to 30 foot depressed trench. Required large scale utility and roadway relocations.

US 60 Improvements, Phoenix, AZ, Design/Build, $184,300,000. New fly over bridges, retaining walls, utility upgrades and road widening.

Hiawatha Light Rail Transit, Minneapolis, MN, Design/Build $310,000,000. 12 mile light rail line from Great Mall to downtown Minneapolis. Substantial roadway grading, utilities/communication relocation, bridges, and electrical upgrades.

I-17 Improvements, Phoenix, AZ, Design/Build, $79,750,000. Extensive widening and bridge construction.

PRIOR EXPERIENCE:
SENIOR/LEAD ESTIMATOR, GUY F. ATKINSON CONSTRUCTION COMPANY, 1994-1997
Guy F. Atkinson Construction Company was an international builder of heavy civil infrastructure projects.

- Successful low bidder on three large "Big Dig" projects in Boston,MA. Contract value $103,000,000, $84,000,000 and $72,000,000.

CONSTRUCTION MANAGEMENT & SENIOR ESTIMATOR, WASHINGTON CONSTRUCTION CO. (MORRISON-KNUDSEN) 1989-1994
- Estimated and worked onsite as asst. construction manager for Eastside Reservoir Project, Hemet, CA. Largest reservoir in Southern California. Contract value $454,000,000

ESTIMATOR, FIELD ENGINEER- GRADE-WAY (RGW) CONSTRUCTION, INC. 1982-1989
Public and private sector heavy/highway, excavation, and paving contractor

EDUCATION
San Diego State University. B.S. in Civil Engineering, B.S. in Geological Sciences, 1981

DECLARATION OF
KATHY ROSE

I, KATHY ROSE, declare as follows:

1. I am presently employed by CH2M HILL Incorporated as a SOIL AND WATER SCIENTIST.
2. A copy of my professional qualifications and experience are attached hereto and incorporated herein by reference.
3. I helped prepare the attached testimony on Soils and Water Resources for the Ivanpah Solar Electric Generating System project based on my independent analysis, supplements thereto, data from reliable sources, and my professional experience and knowledge.
4. It is my professional opinion that the prepared testimony is valid and accurate with respect to the issue(s) addressed herein.
5. I am personally familiar with the facts and conclusions related in the testimony and if called as a witness could testify competently thereto.

I declare under penalty of perjury that the foregoing is true and correct to the best of my knowledge and belief.

Dated: 11/6/2009 Signed: Kathy Rose

At: Sacramento, California

Kathy Rose, Ph.D.
Soil and Water Scientist

Education

Ph.D., Soil and Water Sciences
M.S., Soil Science
B.S., Soil Science

Licenses/Certifications

Certified Professional Soil Scientist, ARCPACS No. 36374

Relevant Experience

Dr. Rose has 20 years of combined experience in academic research related to soil-plant-water relationships; water quality planning with the California Regional Water Quality Control Board-Santa Ana Region; and environmental consulting. Her academic experience focused on the ecosystem function of soils in forest and chaparral environments. Water quality regulatory experience included development of total maximum daily loads (TMDLs), and construction and industrial stormwater permit compliance. Dr. Rose also has extensive experience in water quality regulatory compliance, including obtaining Clean Water Act Section 404/401 permits from the Army Corps of Engineers and Regional Water Quality Control Boards; Streambed Alteration Agreements with California Department of Fish and Game; and mitigation planning. Most recently, Dr. Rose's work includes developing applied solutions to soil and water management projects, including land application of municipal waste, land reclamation, nutrient management planning, and alternative landfill covers. Additionally, she has managed or collaborated on a number of CEQA/NEPA projects, including Initial Studies-Mitigated Negative Declarations, Environmental Impact Reports, and General Plan updates. Dr. Rose has worked on several solar energy projects, including preparation of Applications for Certification, development of restoration/revegetation plans, and evaluating beneficial use impacts to waters of the State.

Representative Projects

BrightSource Energy, Ivanpah Solar Electric Generating System, California. Dr. Rose prepared responses to comments from California Energy Commission related to soil and water resources. Specifically, she evaluated water quality impacts from heliostat wash water, was a contributing author to the Restoration and Revegetation Plan, and the Beneficial Use Impacts Evaluation related to dredge and fill within ephemeral drainages.

SolarReserve, Rice Solar Energy Project. Dr. Rose prepared the Soil Resources section of the Application for Certification for this solar energy project proposed for development in eastern Riverside County, California.

Iberdrola, Reconnaissance Surveys for Solar Energy Facility Siting. Dr. Rose prepared comprehensive evaluations of soils for several potential solar energy sites in California, Arizona, Nevada and New Mexico. Soil survey information was used, when available, to compare sites for relative soil erodibility via wind and water, and other physical and chemical properties that could affect feasibility for development.

Contra Costa Generating Station LLC, Contra Costa Generating Station Project. Dr. Rose prepared the Soils Resources section of the Application for Certification for a 624 MW, natural gas-fired power plant to be located in the city of Oakley, Contra Costa County, California.

Turlock Irrigation District, Almond 2 Power Plant. Dr. Rose provided senior review of the Soil Resources section of the Application for Certification for a 174 MW natural gas-fired combined cycle power plant to be developed in the city of Ceres, California.

Caltrans, Comprehensive Monitoring Program Guidance Manual –Statewide. Dr. Rose authored chapters and provided technical review on sections of the revised Caltrans Stormwater Monitoring Guidance Manual, which provides protocols for planning and implementing stormwater monitoring programs/projects conducted on Caltrans' facilities.

Regional Water Quality Control Board-Santa Ana Region, Water Quality Planning and Stormwater Compliance. While working with the Coastal Waters Planning Section of the Santa Ana Regional Water Quality Control Board, Dr. Rose developed TMDLs for organochlorine compounds (DDT, PCBs, chlordane, toxaphene) for San Diego Creek and Newport Bay, Orange County, California. Work included holding stakeholder and CEQA scoping meetings; analyzing data; organizing and participating in a Technical Advisory Committee; preparing technical staff reports and the Draft Basin Plan Amendment. Presented the TMDLs with Implementation Plan at the December 2006 meeting of the SARWQCB; and the TMDLs were adopted by the Board in 2007. Additionally, oversaw implementation of the sediment TMDLs for San Diego Creek and Newport Bay; participated in grant proposal review and selection; issued 401 Water Quality Certifications; and managed contracts. With the Coastal Waters Storm Water Section, inspected construction and industrial sites for compliance with stormwater general permits; wrote inspection reports; prepared enforcement actions when necessary (e.g., Administrative Civil Liability Complaints [ACLs]); and assisted with the MS4 permit update for Orange County.

University of California, Riverside, Academic Research related to Soil and Water Science. While working in the Soil and Environmental Sciences Department at UC Riverside, Dr. Rose designed and implemented complex field and laboratory studies that primarily focused on the role of weathered granitic bedrock in forest and chaparral ecosystems. Conducted literature reviews, installed field equipment; designed and built laboratory equipment to process samples; analyzed data; prepared manuscripts for journal publication. Directed staff; provided direction to undergraduate students; trained students and staff in the use of specialized equipment and laboratory methods; and taught the laboratory portion of a graduate-level class in Soil Mineralogy.

DECLARATION OF

Gary Rubenstein

I, Gary Rubenstein, declare as follows:

1. I am presently employed by Sierra Research as a Senior Partner.
2. A copy of my professional qualifications and experience are attached hereto and incorporated herein by reference.
3. I helped prepare the attached testimony on Air Quality, Public Health, and Alternatives for the Ivanpah Solar Electric Generating System project based on my independent analysis, supplements thereto, data from reliable sources, and my professional experience and knowledge.
4. It is my professional opinion that the prepared testimony is valid and accurate with respect to the issue(s) addressed herein.
5. I am personally familiar with the facts and conclusions related in the testimony and if called as a witness could testify competently thereto.

I declare under penalty of perjury that the foregoing is true and correct to the best of my knowledge and belief.

Dated: November 13, 2009 Signed: Gary G Rubenstein

At: Sacramento, California

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Résumé

Gary S. Rubenstein

Education

1973, B.S., Engineering, California Institute of Technology

Professional Experience

8/81 to present — Senior Partner
Sierra Research

As one of the founding partners of Sierra Research, responsibilities include project management and technical and strategy analysis in all aspects of air quality planning and strategy development; project licensing and impact analysis; emission control system design and evaluation; rulemaking development and analysis; vehicle inspection and maintenance program design and analysis; and automotive emission control design, from the initial design of control systems to the development of methods to assess their performance in customer service. As the Partner principally responsible for Sierra Research's activities related to stationary sources, he has supervised the preparation of control technology assessments, environmental impact reports and permit applications for numerous industrial and other development projects, including over 17,000 megawatts of electrical generating capacity, throughout the United States.

While with Sierra, Mr. Rubenstein has managed and worked on numerous projects, including preparation of nonattainment plans; preparation and review of emission inventories and control strategies; preparation of the air quality portions of environmental review documents for controversial transportation, energy, mineral industry and landfill projects; preparation of screening health risk assessments and supporting analyses; and the development of air quality mitigation programs. Mr. Rubenstein has managed the preparation of air quality licensing applications for over 13,000 megawatts of generating capacity before the California Energy Commission, and has managed air quality analyses for over 21,000 megawatts of generating capacity in a variety of jurisdictions.

Mr. Rubenstein has presented testimony and served as a technical expert witness before numerous state and local regulatory agencies, including the U.S. Environmental Protection Agency, California State Legislative Committees, the California Air Resources Board, the California Energy Commission, the California Public Utilities Commission, numerous California air pollution control districts, the Connecticut Department of Environmental Protection, the Hawaii Department of Health, and the

Alabama Department of Environmental Management. Mr. Rubenstein has also served as a technical expert on behalf of the California Attorney General and Alaska Department of Law, and has provided expert witness testimony in a variety of administrative and judicial proceedings.

6/79 to 7/81 — Deputy Executive Officer
California Air Resources Board

Responsibilities included policy management and oversight of the technical work of ARB divisions employing over 200 professional engineers and specialists; final review of technical reports and correspondence prepared by all ARB divisions prior to publication, covering such diverse areas as motor vehicle emission standards and test procedures, motor vehicle inspection and maintenance, and air pollution control techniques for sources such as oil refineries, power plants, gasoline service stations and dry cleaners; review of program budget and planning efforts of all technical divisions at ARB; policy-level negotiations with officials from other government agencies and private industry regarding technical, legal, and legislative issues before the Board; representing the California Air Resources Board in public meetings and hearings before the California State Legislature, the California Energy Commission, the California Public Utilities Commission, the Environmental Protection Agency, numerous local government agencies, and the news media on a broad range of technical and policy issues; and assisting in the supervision of over 500 full-time employees through the use of standard principles of personnel management and motivation, organization, and problem solving.

7/78 – 7/79 — Chief, Energy Project Evaluation Branch
Stationary Source Control Division
California Air Resources Board

Responsibilities included supervision of ten professional engineers and specialists, including the use of personnel management and motivation techniques; preparation of a major overhaul of ARB's industrial source siting policy; conduct of negotiations with local officials and project proponents on requirements and conditions for siting such diverse projects as offshore oil production platforms, coal-fired power plants, marine terminal facilities, and almond-hull burning boilers.

During this period, Mr. Rubenstein was responsible for the successful negotiation of California's first air pollution permit agreements governing a liquefied natural gas terminal, coal-fired power plant, and several offshore oil production facilities.

10/73 to 7/78 Staff Engineer
Vehicle Emissions Control Division
California Air Resources Board

Responsibilities included design and execution of test programs to evaluate the deterioration of emissions on new and low-mileage vehicles; detailed analysis of the effect of California emission standards on model availability and fuel economy; analysis of proposed federal emission control regulations and California legislation; evaluation of the cost-effectiveness of vehicle emission control strategies; evaluation of vehicle inspection and maintenance programs, and preparation of associated legislation, regulations and budgets; and preparation of detailed legal and technical regulations regarding all aspects of motor vehicle pollution control. Further duties included preparation and presentation of testimony before the California Legislature and the U.S. Environmental Protection Agency; preparation of division and project budgets; and creation and supervision of the Special Projects Section, a small group of highly trained and motivated individuals responsible for policy proposals and support in both technical and administrative areas (May 1976 to July 1978).

Credentials and Memberships

Air & Waste Management Association (Chair, Board of Directors, Golden West Section; Member, Board of Directors, Mother Lode Chapter)

American Society of Mechanical Engineers

Qualified Environmental Professional, Institute of Professional Environmental Practice, 1994

Selected Publications (Author or Co-Author)

"Evaluation of CTM-039 Dilution Method for Measuring $PM_{10}/PM_{2.5}$ Emissions from Gas-Fired Combustion Turbines," August 20, 2009.

"Dealing with the Scarcity of PM Offsets," presented to Law Seminars International: Air Quality Regulation in California on April 15, 2008, in Los Angeles, CA.

"Field Demonstration of a Dilution-Based Particulate Measurement System," presented to Stationary Source Sampling and Analysis for Air Pollutants on March 5, 2008, in San Diego, CA.

"The California Global Warming Solutions Act of 2006 – Implementation Considerations," presented to Law Seminars International: Energy in California 2007 on September 17, 2007, in San Francisco, CA.

“Preparing for and Conducting Air Quality Compliance Audits,” presentation to California Desert Air Working Group on October 19, 2006, in Big Bear Lake, CA.

“Test Results from Sugar Cane Bagasse and High Fiber Cane Co-fired with Fossil Fuels,” Biomass and Bioenergy, Vol. 30, Issue 6. pp. 565-574. June 2006.

“Gas Turbine Particulate Matter Emissions – Update,” Presentation to ASME/EIGTI Turbo Exp. on June 9, 2005 in Reno, NV.

“Gas Turbine Startup Emissions,” Presentation to ASME/IGTI Turbo Expo on June 9, 2005 in Reno, NV.

“Gas Turbine Particulate Matter Emissions – Update,” Presentation to ASME/IGTI Turbo Expo on June 18, 2003 in Atlanta, GA.

“Sources of Uncertainty When Measuring Particulate Matter Emissions from Natural Gas-Fired Combustion Turbines,” Presented to Air & Waste Management Association on March 30, 2001 in San Diego, CA.

“An Analysis of the Effect on Emissions of Allowing Drive-Thru Service Lanes,” Sierra Research Report No. SR97-11-01, prepared for California Business Properties Association, November 10, 1997.

“Searles Valley Air Quality Study (SVAQS) Final Report,” Sierra Research Report No. SR94-02-01, prepared for North American Chemical Company, February 1994.

“Regulatory Strategies for Reducing Emissions from Marine Vessels in California Waters,” Sierra Research Report No. SR91-10-01, prepared for the California Air Resources Board, October 4, 1991.

“An Analysis of the Effect on Emissions of Eliminating Drive-Thru Services Lanes,” Sierra Research Report No. SR91-07-03, prepared for California Restaurant Association, July 25, 1991.

“Development of the CALIMFAC California I/M Benefits Model,” Sierra Research Report No. SR-91-01-01, prepared for the California Air Resources Board, Agreement No. A6-173-64, January 1991.

“Criteria Pollutant Emission Inventory for the Coachella Valley Study Area,” Sierra Research Report No. SR90-11-01, prepared for South Coast Air Quality Management District, November 1990.

“User’s Guide to the CALIMFAC California I/M Benefits Model,” Prepared for the California Air Resources Board, May 1990.

"Potential Emissions and Air Quality Effects of Alternative Fuels – Final Report," Sierra Research Report No. SR89-03-04, prepared for Western States Petroleum Association, March 28, 1989.

"Interprecursor Offset Ratios for Ozone in the Searles Valley," Sierra Research Report No. SR89-03-02, prepared for Kerr-McGee Chemical Company, March 17, 1989.

"An Assessment of the Quality of California's Air Pollution Emissions Inventory," Sierra Research Report No. SR88-05-01, prepared for Western Oil and Gas Association, May 1988.

"Trends in Visibility-Related Emissions Affecting the R-2508 Restricted Airspace," Sierra Research Report No. SR88-05-02, prepared for Western Oil and Gas Association, May 1988.

"Volume I, Executive Summary: Impacts of Air Quality Regulations on Visibility-Related Emissions in the California R-2508 Restricted Airspace," Sierra Research Report No. SR88-03-02, prepared for Western Oil and Gas Association, March 1988.

"Volume II, Determination of California Air Basins Which Can Affect Visibility in the R-2508 Restricted Airspace," Sierra Research Report No. SR88-03-03, prepared for Western Oil and Gas Association, March 1988.

"Air Quality Impact Analysis for the Soledad Biomass Resource Recovery Project," Sierra Research Report No. SR87-10-01, prepared for Western Forest Power Corp., October 1987.

"Air Quality Impact Analysis for the Honey Lake Biomass Power Plant Project," Sierra Research Report No. SR87-05-01, prepared for GeoProducts-Zurn/NEPCO, May 22, 1987.

"1986 Update to the Kern County Nonattainment Area Plan," Sierra Research Report No. SR86-03-01, prepared for Kern County Air Pollution Control District and Kern Council of Governments, March 1986.

"An Analysis of Test Results on Grancor Pollution Control Devices for Automotive Retrofit Programs," Sierra Research Report No. SR85-09-01, prepared for Grancor, September 1985.

"Temperature Correction Factors for California's Motor Vehicle Emissions Model," Sierra Research Report No. SR85-06-01, prepared for the California Air Resources Board, June 1985.

“Critique of the EPA I/M Benefits Model for 1980 and Older Model Cars,” Sierra Research Report No. SR85-06-02, prepared for the California Air Resources Board, June 1985.

“Emission Factors for 1980 and Later Model Year California Passenger Cars and Light-Duty Trucks,” Sierra Research Report No. SR85-06-03, prepared for the California Air Resources Board, June 1985.

“Technology Assessment for Light-Duty Vehicle Compliance with a 0.4g/m NOx Standard,” Sierra Research Report No. SR85-06-04, prepared for the California Air Resources Board, June 1985.

“Development of California’s I/M Credits Model,” Sierra Research Report No. SR85-06-06, prepared for the California Air Resources Board, June 1985.

“Evaluation of Automotive CO Emissions Control Techniques at Low Temperatures (METFAC Report 2),” Sierra Research Report No. SR84-11-01, prepared for Alaska Department of Environmental Conservation, November 1984.

“Critical Metal Consumption in Automotive Catalysts – Trends and Alternatives,” Sierra Research Report No. SR83-12-01, prepared for Congress of the United States, Office of Technology Assessment, December 1983.

“Low Temperature Automotive Emissions (METFAC, Report 2),” Sierra Research Report No. SR83-11-01, prepared for Alaska Department of Environmental Conservation, November 1983.

“Proposed Emission Cutpoints for the Anchorage Inspection and Maintenance Program,” Sierra Research Report No. SR83-06-01, prepared for Municipality of Anchorage, Alaska, June 1983.

“A Study of Air Pollution Offsets for Cogeneration and Resource Recovery Technologies in Kern County – Interim Report: Project Inventory,” Sierra Research Report No. SR82-01-01, prepared for Kern County Air Pollution Control District and Kern County Council of Governments, January 1983.

“Automotive Retrofit Devices for Improving Cold Weather Emissions and Fuel Economy,” Sierra Research Report No. SR82-10-01, prepared for U.S. Army Cold Regions Research and Engineering Laboratory, October 1982.

“Carbon Monoxide Air Quality Trends in Fairbanks, Alaska,” Sierra Research Report No. SR82-09-01, prepared for Fairbanks North Star Borough, September 1982.

“Cogeneration and Resource Recovery in Kern County – Final Report,” Sierra Research Report No. SR82-06-01, prepared for Kern County Air Pollution Control District and Kern County Council of Governments, June 1982.

"Cold Weather CO Problems – An Analysis of Research Needs," Sierra Research Report No. SR82-04-01, prepared for Alaska Department of Environmental Conservation, April 1982.

"The Potential for the Use of Catalytic NOx Controls on Stationary Sources in California," Sierra Research Report No. SR82-02-01, February 1982.

"Staff Report - Cogeneration Technology and Resource Recovery Status Report," California Air Resources Board, November 1981.

"The Effect of Clean Air Act Amendments on High Altitude Passenger Cars," Sierra Research Report No. SR81-09-01, September 1981.

"Staff Report - Public Meeting to Discuss Proposed Guidelines for the Control of Emissions from Coal-Fired Power Plants (81-11-2)," California Air Resources Board, June 1981.

"Staff Report - Public Hearing to Consider Amendments to Title 13, Section 1960.1, CAC, Regarding Exhaust Emission Standards and Test Procedures for 1983 and Subsequent Model Passenger Cars, Light-Duty Trucks and Medium-Duty Vehicles," California Air Resources Board, May 1981.

"Staff Report - Suggested Control Measure for the Control of Hydrogen Sulfide Emissions from Geothermal Operations at the Geysers Known Geothermal Resources Area (81-6-1)," California Air Resources Board, April 1981.

"Staff Report - Proposed Methodology for Calculating a NOx Amelioration Factor for Light-Duty Diesel Vehicles," California Air Resources Board, April 1981.

"Staff Report - A Proposed Air Resources Board Policy Regarding Incineration as an Acceptable Technology for PCB Disposal," California Air Resources Board, March 1981.

"Staff Report - Public Meeting to Discuss a Proposed Air Resources Board Policy Regarding Incineration as an Acceptable Technology for PCB Disposal," California Air Resources Board, March 1981.

"Staff Report - Suggested Control Measure for the Control of Oxides of Nitrogen Emissions from Electric Utility Gas Turbines (81-4-2)," California Air Resources Board, March 1981.

"Staff Report - Public Hearing to Consider Amendments to Title 13, Section 1956.7, CAC, Regarding Exhaust Emission Standards and Test Procedures for 1984 and Subsequent Model Heavy Duty Engines (81-1-1)," California Air Resources Board, January 1981.

"Gasohol: Technical, Economic or Political Panacea?" SAE Paper No. 800891, 1980.

"Staff Reports Related to Public Hearing to Consider Amendments to Rule 475.1 of the South Coast Air Quality Management District and to Rule 59.1 of the Ventura County Air Pollution Control District, Which Control the Emissions of Oxides of Nitrogen from Power Plants," California Air Resources Board, January 1980; March 1980; November 1980; December 1980.

"Staff Report - Public Hearing to Consider Confirmation of Emergency Adoption of Section 1960.4, Title 13, CAC, Regarding Special NOx Standards for Small-Volume Manufacturers (80-25-1)," California Air Resources Board, December 1980.

"Staff Report - Public Hearing to Consider Adoption of California Assembly- Line Test Procedures for Certain 1982 Model Year Vehicles and Adoption of Section 2060, Title 13, CAC, Incorporating the Test Procedures (80-26-4)," California Air Resources Board, December 1980.

"Staff Report - Public Hearing to Consider Repeal of 1955-1965 Model Year Motor Vehicle Exhaust Retrofit Emission Control Requirements - Title 13, CAC Section 2007 (80-20-2)," California Air Resources Board, October 1980.

"Staff Report - Public Hearing to Consider Amendments to Rule 424 of the Kern County APCD Controlling Emissions of Sulfur Oxide from Steam Generators Used in Oil Field Operations," California Air Resources Board, October 1980.

"Staff Report - Proposed Amendments to Title 13, CAC, Sections 2035-42, Regarding Warranty of Emissions-Related Components of Vehicles (80-18-1)," California Air Resources Board, September 1980.

"Staff Report - Proposed Amendment to Title 13, CAC Regarding Standards and Test Procedures for Modified Vehicles - 1981 and Subsequent Passenger Cars, Light-Duty Trucks and Medium-Duty Vehicles," California Air Resources Board, September 1980.

"Staff Report - Public Meeting to Discuss Issues Related to Power Plant Siting," California Air Resources Board, September 1980.

"Staff Report - Emergency Public Hearing to Consider Amendments to Title 13, CAC, Regarding Exhaust Emission Standards for Oxides of Nitrogen (NOx) from Vehicles Produced by Small Manufacturers for the 1982-1986 Model Years of Passenger Cars, Light-Duty Trucks and Medium- Duty Vehicles," California Air Resources Board, August 1980.

"Staff Report - Emergency Public Hearing to Consider Adoption of a Particulate Exhaust Emission Standard for 1982 and Subsequent Model Year Light-Duty Diesel Vehicles and to Consider Amending the 1982 NOx Exhaust Emission Standard for Those Vehicles (80-15-2)," California Air Resources Board," August 1980.

"Cogeneration Technology and Resource Recovery Status Report," California Air Resources Board, August 1980.

"Staff Report - Response to the Motorcycle Manufacturers' Petition Requesting the Board Reevaluate the 1.0 Gram Per Kilometer Exhaust Emission Standard for 1982 and Subsequent Model Year Motorcycles (80-13-3)," California Air Resources Board, July 1980.

"Staff Report - Inventory of Potential Cogeneration Technology and Resource Recovery Projects Planned or Proposed to Be Constructed Before 1987," California Air Resources Board, July 1980.

"Staff Report - Public Hearing to Consider Proposed Amendments to Kern County APCD Rule 424 - Sulfur Compounds from Oil Field Steam Generators," California Air Resources Board, May 1980.

"Staff Report - Public Hearing to Consider Amending the Rules and Regulations of Imperial County Air Pollution Control District, Los Angeles County Air Pollution Control District and San Bernardino County Air Pollution Control District," California Air Resources Board, May 1980.

"Staff Report - Public Hearing to Consider Amendments to Title 13, CAC, Regarding the Extension of California's 1980 Heavy-Duty Engine Emission Standards through the 1983 Model Year," California Air Resources Board, May 1980.

"Staff Report - Public Hearing to Consider Amendments to the Rules and Regulations of the Kern County APCD Amendments to Rule 210.1, Standard for Authority to Construct, and Addition of Rule 425, Relating to Retrofit Control for Emissions of Oxides of Nitrogen from Oil Fired Steam Generators," California Air Resources Board, March 1980.

"Staff Report - Public Hearing to Consider Proposed Amendments to Title 13 of the Administrative Code and to the Exhaust and Evaporative Emission Standards and Test Procedures for 1981 and Subsequent Model year Passenger Cars, Light-Duty Trucks and Medium-Duty Vehicles," California Air Resources Board, March 1980.

"Air Pollution Aspects of Resource Recovery Facilities," California Air Resources Board, March 1980.

"Memorandum of Agreement - Hondo 'A' Development Santa Ynez Unit, Santa Barbara Channel between The State of California, County of Santa Barbara and Santa Barbara Air Pollution Control District and Exxon Company, U.S.A.," California Air Resources Board, February 1980.

"A Report on California's Certificate of Compliance Program prepared for the California Legislature Joint Legislative Budget Committee in accordance with the requirements of the Supplemental Report on Item 194 of the Committee of Conference on the Budget," California Air Resources Board, December 1979.

"Status Report on the Need for/and Feasibility of a 0.4 NOx Standard for Light Duty Motor Vehicles," California Air Resources Board, December 1979.

"Staff Report - Status of NOx Control for Steam Generators and Availability of NOx Trade-offs in Kern County (79-27-1b)," California Air Resources Board, November 1979.

"Staff Report - Public Meeting to Consider Model Rule for the Control of Oxides of Nitrogen Emissions from Stationary Internal Combustion Engines (79-28-2)," California Air Resources Board, November 1979.

"First Annual Report to the Legislature on the Mandatory Vehicle Inspection Program (MVIP)," California Air Resources Board, October 1979.

"Chapter 27, California Lead Control Strategy - Revision to the State of California Implementation Plan for the Attainment and Maintenance of Ambient Air Quality Standards," California Air Resources Board, September 1979.

"Staff Report - Public Hearing to Reconsider the Adoption by the Board into the Regulations of the Kern County Air Pollution Control District on March 23, 1979, of Rule 424, for the Control for Emissions of Sulfur Compounds from Steam Generators Used in Oil Field Operations," California Air Resources Board, August - September 1979.

"Staff Report - Public Hearing to Consider the Adoption of Chapter 27 as a Revision to the State of California Implementation Plan for the Attainment and Maintenance of the National Ambient Air Quality Standards for Lead," California Air Resources Board, August 1979.

"Staff Report - Public Hearing to Consider Amendment of the State Regulation Which Limits the Lead Content of Gasoline Sold in California (79-22-1)," California Air Resources Board, August 1979.

"Staff Report – Alcohols and Alcohol/Gasoline Blends as Motor Fuels," California Air Resources Board, August 1979.

"Centralized Vehicle Inspection/Maintenance in California," California Air Resources Board, May 1979.

"Staff Report - Public Hearing to Consider Changes to the Air Resources Board's Standards and Test Procedures for 1980 and Subsequent Model Passenger Cars, Light-Duty Trucks, and Medium-Duty Vehicles," California Air Resources Board, April 1979.

"Staff Report - Public Hearing to Consider Proposed Changes in the Regulations of the Air Resources Board Regarding Predelivery Inspection and Compliance Test Evaluation," California Air Resources Board, April 1979.

"An Evaluation of California's Private Garage Emissions Inspection Program," California Air Resources Board, March 1979.

"Staff Report - Proposed Rule For Control of Emissions of Sulfur Compounds From Steam Generators and Boilers Used in Oilfield Operations in the Kern County Air Pollution Control District," California Air Resources Board, March 1979.

"Staff Report - Public Hearing to Consider Adoption of a Regulation Controlling Emissions of Sulfur Compounds from Steam Generators Used in Oilfield Operations in the Kern County APCD," California Air Resources Board, March 1979.

"Staff Report - Revisions to the State of California Implementation Plan (SIP) for the Attainment and Maintenance of National Ambient Air Quality Standards - Kings County, Madera County, Merced County, and Tulare County Non-attainment Plans (NAPs)," California Air Resources Board, February 1979.

"Staff Report - Public Meeting to Consider a Proposed Model New Source Review Rule," California Air Resources Board, January 1979.

"Staff Report - Proposed ARB-CEC Joint Policy Statement of Compliance with Air Quality Laws by New Power Plants (79-1-3)," California Air Resources Board, January 1979.

"Staff Report - Public Hearing to Consider Exhaust Standards for the Mandatory Vehicle Inspection Program," California Air Resources Board, September 1978.

"Staff Report - Public Hearing to Consider Proposed Emissions Warranty Regulations (78-3-1)," California Air Resources Board, February 1978.

"Staff Report - Public Hearing to Consider Proposed Highway Cycle Emission Standard for Passenger Cars, Light Duty Trucks, and Medium- Duty Vehicles (78-1-2)," California Air Resources Board, January 1978.

"Staff Report - Public Hearing to Consider Proposed Changes to Motor Vehicle Emission Standards Test Procedures, and Enforcement Programs (77-20-2)," California Air Resources Board, September 1977.

"Staff Report - Surveillance Bibliography of Passenger Cars, Motorcycles, Heavy-Duty and Medium-Duty Vehicles," California Air Resources Board, July 1977.

"Staff Report - Public Hearing on Proposed Changes to Regulations Regarding California Exhaust Emission Standards and Test Procedures for 1980 and Subsequent Model Motor Vehicles (78-9-2)," California Air Resources Board, May 1977.

"Staff Report - Public Hearing on Proposed Changes to Regulations Regarding Allowable Maintenance During New Vehicle Certification of Light-Duty and Medium-Duty Vehicles (77-12-1)," California Air Resources Board, May 1977.

"Staff Report - Public Hearing on Proposed Changes to Regulations Regarding Allowable Maintenance During New Vehicle Certification of Light-Duty and Medium-Duty Vehicles (77-9-2)," California Air Resources Board, April 1977.

"Staff Report - Manganese Fuel Additive MMT (77-9-3)," California Air Resources Board, April 1977.

"Staff Report - Public Hearing to Consider Amendments to the Hydrocarbon Standards and Test Procedures Applicable to 1978 Through 1981 Production Year Motorcycles (77-6-2)," California Air Resources Board, March 1977.

"Staff Report - Status Report on the Mandatory Vehicle Inspection Program (MVIP) (77-4-2)," California Air Resources Board, February 1977.

"Staff Report - Control of Motorcycle Evaporative Emissions and Certification of Motorcycle Fuel Fill Pipes (77-63)," California Air Resources Board, March 1977.

"Staff Report - Public Hearing on Proposed Changes to Regulations Regarding Vehicle Evaporative Emission Standards for 1980 and Subsequent Model Motor Vehicles (76-22-2 c)," California Air Resources Board, November 1976.

"Staff Report - Public Hearing on Proposed Changes to Regulations Regarding Exhaust Emission Standards and Test Procedures for 1979 and Subsequent Model Passenger Cars, Light-Duty Trucks and Medium-Duty Vehicles (76-22-2 a)," California Air Resources Board, November 1976.

"Staff Report - Public Hearing on Proposed Changes to Regulations Regarding Allowable Maintenance During New Vehicle Certification of Light-Duty and Medium-Duty vehicles (76-22-2 b)," California Air Resources Board, November 1976.

"Staff Report - Evaluation of Mandatory Vehicle Inspection and Maintenance Programs," California Air Resources Board, May-August 1976.

"Staff Report - Public Hearing to Consider Proposed Changes to Regulations Regarding Approval of 1978 and Subsequent Model Light-Duty Trucks and Heavy-Duty Engines (76-6-2)," California Air Resources Board, March 1976.

"Staff Report - Public Hearing to Consider Amendments to California Fuel Evaporative Emissions Test Procedures for 1978 and Subsequent Model Gasoline-Powered Vehicles (76-6-3)," California Air Resources Board, March 1976.

"Staff Report - Public Hearing Regarding Amendment of Emission Standards and Test Procedures for Motorcycles (76-1-4)," California Air Resources Board, January 1976.

"Staff Report - Catalyst Service and Replacement Regulations (75-20-2)," California Air Resources Board, October 1975.

"Staff Report - Emergency Action to Amend the New Vehicle Approval Regulations Regarding Catalyst Change (75-18-2)," California Air Resources Board, September 1975.

"Staff Report - Progress Report on Technology to Control Sulfate Emissions from Catalyst-Equipped Vehicles (75-15-2)," California Air Resources Board, August 1975.

"Staff Report - Public Hearing to Consider 1978 Production Motorcycle Emission Standards (75-14-2)," California Air Resources Board, July 1975.

"Staff Report - Consideration of Regulation Change to Extend the Alternate Heavy-Duty Engine Standards for 1977 and Subsequent Years (75-14-3)," California Air Resources Board, July 1975.

"Staff Report - Motorcycle Emission Control Strategies (75-11-4)," California Air Resources Board, June 1975.

"Staff Report - Catalytic Converter Retrofit Program - Used Vehicles Retrofitted with Universal Oil Products Catalytic Converters Final Report," California Air Resources Board, May 1975.

"Staff Report - Estimate of Contribution of Motorcycles to California Air Pollution (75-9-5)," California Air Resources Board, May 1975.

"Staff Report - Public Hearing for Adoption of Proposed Changes to Vehicular Enforcement Regulations Including Recall Procedures (75-9-4)," California Air Resources Board, May 1975.

"Staff Report - Public Hearing to Consider Inspection Specification Regulations in Title 13 -- New Vehicles (continued) (75-9-3a)," California Air Resources Board, May 1975.

"Staff Report - Emergency Action to Delete High Altitude Test Provisions from the 1975 and Subsequent New Vehicle Approval Procedures (75-7-7)," California Air Resources Board, April 1975.

"Staff Report - Public Hearing to Consider Fuel Evaporative Emission Regulations for Light-Duty Vehicles (75-7-6)," California Air Resources Board, April 1975.

"Staff Report - Reconsideration of Exhaust Emission Standards for 1977 and Subsequent Model-Year Heavy-Duty Engines (75-7-2)," California Air Resources Board, April 1975.

"Staff Report - Exhaust Emission Standards for 1977 Model-Year Light-Duty Vehicles (75-5-2)," California Air Resources Board, March 1975.

"Smog: A Report to the People," Caltech Environmental Quality Lab, 1972.

DECLARATION OF
ANDREW C. SANDERS

I, Andrew C. Sanders, declare as follows:

1. I am presently employed by the University of California, Riverside as Senior Museum Scientist and curator of the Herbarium, Dept. of Botany & Plant Sciences.

2. A copy of my professional qualifications and experience are attached hereto and incorporated herein by reference.

3. I helped prepare the botany portion of the attached Biological Resources testimony for the Ivanpah Solar Electric Generating System project based on my independent analysis, supplements thereto, data from reliable sources, and my professional experience and knowledge.

4. It is my professional opinion that the prepared testimony is valid and accurate with respect to the issue(s) addressed herein.

5. I am personally familiar with the facts and conclusions related in the testimony and if called as a witness could testify competently thereto.

I declare under penalty of perjury that the foregoing is true and correct to the best of my knowledge and belief.

Dated: 10 NOV. 2009 Signed: Andrew C. Sanders

At: Riverside, CA

Andrew C. Sanders, Herbarium Curator
Department of Botany & Plant Sciences
University of California, Riverside, CA 92521-0124
(951) 827-3601

Education

B.Sc. in Biology, specializing in Botany;
University of California, Riverside. June I975.

Employment

1. U.S. Department of the Interior, Bureau of Land Management (Riverside and Bakersfield Districts and California Desert Plan Staff). Aug. 1975 to Apr. 1978

2. University of California, Riverside. Dept. of Biology. Staff Research Associate and resident biologist at the James Reserve in the San Jacinto Mountains of Riverside County California. April I978 to Sept. I979.

3. University of California, Riverside. Dept. of Botany & Plant Sciences. Since September I979 I have been Museum Scientist and curator of the Herbarium. I have identified well over 100,000 plant specimens from North America and have enlarged the UCR collection to over thirteen times its former size (200,000 specimens) making it the 5th largest CA herbarium. I have personally collected over 37,000 plant specimens in North Am., especially from southern CA. I am generally recognized as an authority on the flora of Southern California and am regularly contacted by the USFWS and CA DFG for information on the status and distribution of threatened & endangered plant species.

Representative Publications

Boyd, S. and A.C. Sanders. 1999. "Noteworthy Collections, California, *Madroño* 46 (2): 112.

Costea, M., A.C. Sanders & J. G. Waines. 2001. Preliminary results toward a revision of the Amaranthus hybridus species complex (Amaranthaceae) *Sida* 19 (4): 931-974

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Elvin, M. A. and A. C. Sanders. 2009. Nomenclatural Changes for Monardella (Lamiaceae) in California. *Novon* 19 (3):

Elvin, M.A., A. C. Sanders and M. S. Brunell. In press. *Monardella* in *The Jepson Manual: Higher Plants of California, 2nd ed.*

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Jones, C. E., A. C. Sanders, et al. 1979. "Noteworthy Collections, CA, *Madroño* 29 (2): 101.

Minnich, R. A. and A. C. Sanders, 2000, Sahara Mustard (*Brassica tournefortii*), in California's Wildland Weeds: Identification and Control, C. Bossard, J. Randall, & M. Hoshovsky, eds, University of CA Press

Provance, M.C. and A.C. Sanders. 2005. *Diospyros torresii* (Ebenaceae): a new black zapote from tropical Mexico. *Sida* 21:2045-2050.
Provance, M.C. and A.C. Sanders. 2006. More American black sapotes: new *Diospyros* (Ebenaceae) for Mexico and Central America. *Sida* 22:277-304.
Provance, M. C., I. García Ruiz, and A. C. Sanders. 2008. The *Diospyros salicifolia* complex (Ebenaceae) in Mesoamerica. *J. Bot. Res. Inst. Texas* 2(2): 1009-1100
Provance M. C. and A. C. Sanders. 2009. An overview of the *Diospyros campechiana* complex (Ebenaceae) and description of three new species. *J. Bot. Res. Inst. Texas* 3 (1): 85-1123(1): 85 –112.
Roberts, F., S. White, A. Sanders, D. Bramlet, S. Boyd 2004. The Vascular Plants of Western Riverside Co., CA, An Annotated Checklist, F.M. Roberts Pub., San Luis Rey, CA
Sanders, A. C., 1996. Noteworthy Collections, California, *Madroño* 43(4):524-532.
Sanders, A. C., 1997. Noteworthy Collections, California, *Madroño* 44: 203-206.
Sanders, A. C., 1997. "Noteworthy Collections, California, *Madroño* 44 (3) 306-307.
Sanders, A.C. 1998. Polygonaceae in Martin, P., et al. 1998. Gentry's Río Mayo Plants: the tropical deciduous forest & environs of northwest Mexico, U. of AZ Press.
Sanders, A.C. 1999. Invasive Exotics in California: a Perspective from Inland Southern California. In: M. Kelly, E. Wagner, & P. Warner (eds.). Proc. of the CA Exotic Pest Pl. Council Symp. Vol 4: 1998. Pp. 7-10.
Sanders, A. C., D. L. Banks & S. Boyd , 1997 "Rediscovery of Hemizonia mohavensis Keck (Asteraceae) and addition of two new localities", *Madroño* 44 (2): 203-210.
Sanders, A. and S. Boyd, 1996. "Noteworthy Collections, CA, *Madroño* 43(4):523-524.
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Soza, V., M. Provance, & A. Sanders. 2000. Noteworthy Collections, CA. *Madrono* 47: 142.
Van Devender, T., A. Sanders, R. Wilson, & S. Meyer. "Vegetation, Flora, and Seasons of the Rio Cuchujaqui, A Tropical Deciduous Forest near Alamos, Sonora, Mexico", in *The Tropical Deciduous Forest of the Alamos, Sonora, Region: Ecology and Conservation of a Threatened Ecosystem,* ed. by R. H. Robichaux.
Van Devender, T. , A. Sanders, V. Steinmann, et al., 1995. "Noteworthy Collections, Sonora, Mexico, *Madroño* 42 (3): 411-418.
Vasek, F. C. & A. C. Sanders, 1983. Distribution of *Polygala acanthoclada*, *Madroño* 30 (3): 193-194.
White, S. and A. C. Sanders, 1997. "Clarification of Three Camissonia Boothii Subspecies' Distributions in California", *Madroño* 44 (1): 106-112
White, S., A. C. Sanders & M. Wilcox 1996. "Noteworthy Collections, CA, *Madroño* 43 (2): 334-335.

DECLARATION OF

Jennifer Scholl

I, Jennifer Scholl, declare as follows:

1. I am presently employed by CH2M HILL Incorporated as a Senior Project Manager.
2. A copy of my professional qualifications and experience are attached hereto and incorporated herein by reference.
3. I prepared the attached testimony on Land Use for the Ivanpah Solar Electric Generating System project based on my independent analysis, supplements thereto, data from reliable sources, and my professional experience and knowledge.
4. It is my professional opinion that the prepared testimony is valid and accurate with respect to the issue(s) addressed herein.
5. I am personally familiar with the facts and conclusions related in the testimony and if called as a witness could testify competently thereto.

I declare under penalty of perjury that the foregoing is true and correct to the best of my knowledge and belief.

Dated: November 13, 2009 Signed: Jennifer L. Scholl

At: Santa Barbara, CA

Jennifer Scholl
Land Use Task Lead

Education

B.A. Environmental Studies and Political Science International Relations

Distinguishing Qualifications

- Project management
- NEPA/CEQA compliance
- Industrial facility siting studies
- Environmental planning and permitting
- Permit compliance management
- Land use planning/Regulatory policy consistency
- Socioeconomic evaluation
- Public participation and community involvement

Relevant Experience

Ms. Scholl has more than 23 years of experience in environmental planning and permitting of complex and controversial development projects. Specifically, Ms. Scholl has been involved with the permitting and construction compliance for power generation projects and ancillary facilities (i.e., transmission, gas, water, and sewer lines) and offshore oil and gas facilities with onshore processing and storage components in California. In addition to serving in a management capacity, Ms. Scholl's specific emphasis on these projects has been to conduct the land use and policy consistency analyses. Ms. Scholl also has extensive experience managing environmental review projects for local agencies, with significant experience with agencies in Santa Barbara County. She has provided permitting and environmental review support for the following types of projects: power generation (natural gas, wind, and solar), resort, residential, and roadways. She also has extensive experience in leading Public Participation Programs. Prior to her work in private consulting, Ms. Scholl managed the permitting and environmental review of major oil and gas development projects, resort and residential developments, and oversaw the implementation of mitigation monitoring plans for the Santa Barbara County Planning and Development Department.

Representative Projects

Land Use Task Leader; Ivanpah Solar Electric Generating System; Eastern San Bernardino County; California. Land Use Task Leader for preparation of the Land Use Section of an AFC to the CEC for a 400 MW solar energy power generation facility using heliostat fields to focus solar energy on power tower receivers. Responsible for evaluating land use compatibility related to jurisdictional issues associated with the San Bernardino County, Bureau of Land Management, and the CEC. AFC submitted to the CEC in September 2007 and currently going through CEC AFC processing.

Jennifer Scholl

Land Use Task Leader; South Bay Replacement Project; City of Chula Vista; San Diego County; California. Land Use Task Leader for preparation of the Land Use Section of an AFC to the CEC for a 500 MW combined-cycle replacement project for the existing South Bay Power Plant. Responsible for evaluating the land use compatibility issues related to jurisdictional issues associated with the City of Chula Vista, California Coastal Commission, CEC, and the Unified Port of San Diego. AFC submitted to the CEC in June 2006 and currently going through CEC AFC processing.

Project Manager; Lompoc Wind Energy Project Environmental Impact Report (EIR); Lompoc, Santa Barbara County; California. Project Manager for preparation of an EIR, under contract to the County of Santa Barbara, for the development of a 120 MW wind energy electrical generation project on private ranch land in the Lompoc Valley. Public Draft released in July 2007, commercial operation expected in 2008.

Assistant Project Manager; Eastshore Energy Center; City of Hayward; Alameda County; California. Assistant Project Manager for preparation of an Application for Certification (AFC) (CEQA EIR equivalent) to the California Energy Commission (CEC) for a 115 MW peaker power plant in the City of Hayward. Responsible for assisting the Project Manager with day-to-day coordination with the client, CEC, and City of Hayward staff for addressing agency requirements. AFC submitted to the CEC in September 2006 and is going through AFC processing and expected to be approved and in commercial operation in 2009.

Project Manager; Pastoria Energy Facility 160 MW Expansion Project; Calpine Corporation; Kern County. Project Manager for preparation of an AFC to the CEC for a 160-MW simple cycle addition to the existing Pastoria Energy Facility in southern Kern County. Responsible for day-to-day coordination with the client and CEC staff for addressing agency requirements. AFC submitted to the CEC in April 2005, received a license from the CEC in December 2006, and is expected to be and in commercial operation in 2008.

Land Use Task Leader; NRG Carlsbad Energy Center; City of Carlsbad; San Diego County; California. Land Use Task Leader for preparation of the Land Use Section of an AFC to the CEC for a 560 MW combined-cycle project within the existing Cabrillo Energy Center power station. Responsible for evaluating land use compatibility related to jurisdictional issues associated with the City of Carlsbad, California Coastal Commission, and the CEC. AFC submitted to the CEC in September 2007 and currently going through CEC AFC processing.

Regulatory Advisor/Siting Study Manager; Additional Support to Cogeneration Proposals in California; Numerous Confidential Clients; California. Currently supports numerous electrical power generation proposals for multiple clients in California with siting and issue screening, project development, agency coordination, land use permit reconnaissance and strategy for AFC filing. Previous development prospects were in the following areas in California: San Jose, Arcata, Los Banos, Fresno, Antelope Valley, and several sites in southern California.

DECLARATION OF

W. Geoffrey Spaulding, Ph.D.

I, W. Geoffrey Spaulding, declare as follows:

1. I am presently employed by CH2M HILL Incorporated as a Senior Scientist.
2. A copy of my professional qualifications and experience are attached hereto and incorporated herein by reference.
3. The attached testimony on Paleontological Resources, Biological Resources, and Cultural Resources for the Ivanpah Solar Electric Generating System project was prepared by me, or at my direction, based on my independent analysis, supplements thereto, data from reliable sources, and my professional experience and knowledge.
4. It is my professional opinion that the prepared testimony is valid and accurate with respect to the issue(s) addressed herein.
5. I am personally familiar with the facts and conclusions related in the testimony and if called as a witness could testify competently thereto.

I declare under penalty of perjury that the foregoing is true and correct to the best of my knowledge and belief.

Dated: November 12, 2009 Signed: W. Geoffrey Spaulding

At: Henderson, Nevada

W. Geoffrey Spaulding, Ph.D.
Biological Resources and Paleontological Resources

Education

Ph.D., Geology (Paleobiology)
M. S., Geology (Palynology & Vertebrate Paleobiology)
B. A., Anthropology

Certifications

- California State Bureau of Land Management Paleontological Resources Use Permit CA-07-17
- Nevada State Bureau of Land Management Paleontological Resources Use Permit N-82749
- Approved Paleontological Resources Specialist by the California Energy Commission, State of California
- Qualifications as Paleontological Resources Expert Witness accepted by the Attorney General of the State of Washington

Distinguishing Qualifications

- Specialist Arid Lands Phytogeography and Field Ecology
- Specialist Paleontological Resources Management
- Expert in Paleoecology of Western North America
- Specialist in Site Formation Processes, Quaternary Paleobiology, Geoarchaeology, Paleohydrology
- Captain, Signal Corps, U. S. Army Reserve (Retired)

Relevant Experience

Dr. Spaulding is a senior scientist and paleontologist with CH2M HILL with extensive experience in paleobiology, paleontology, and paleoecology. He also is accomplished in the study of site formation processes, and the Quaternary geology of the western United States. He has more than three decades of technical experience in the Earth and Life sciences focusing on the deserts of western North America and on California. Prior to joining private industry, he was on the faculty of the University of Washington, Seattle specializing in paleobiology and paleoecology.

Dr. Spaulding received his advanced degrees at the University of Arizona where for seven years he was a research assistant at the Desert Laboratories, the former Carnegie Institute Desert Botanical Laboratory on Tumamoc Hill, west of Tucson. While at Arizona, Dr. Spaulding engaged in field and laboratory studies that focused on the floristics, plant ecology, and paleoecology of the warm deserts of America (the Mojave, Sonoran, and Chihuahuan deserts). He spent several years studying the methods of Robert Whittaker and others in gradient analyses appropriate to understanding plant community responses to environmental change. His doctoral and postdoctoral research was supported by the National Science Foundation, the U.S. Geological Survey, the Department of Energy, and the State of Nevada, and focused on the phytogeography and paleoecology of the Mojave Desert. His major findings in arid-lands plant ecology and the phytogeographic history of the Mojave Desert are published in his 1985 Geological Survey Professional Paper, his 1990 synthesis in the journal *Quaternary Research*, and summarized by D. K. Grayson in *The Deserts Past (*1993, Smithsonian Press).

Representative Projects (Biology)

Ivanpah Solar Electric Generating System EIS/AFC, Eastern Mojave Desert. Perform senior review of plant ecology and botanical reports, and coauthor Weed Management Plan. Provide senior guidance for field methodology development, as well as bio-climatic characterizations of the project area. Senior team lead for the project Revegetation, Rehabilitation and Restoration Plan. Prepare appropriate revegetation plan sections including the succulent salvage plan for BLM EIS and California Energy Commission Application for Certification. Provide responses and plan additions in response to data requests.

Edison Mission Energy Habitat Inventory and Mapping, Western Mojave Desert. Provide senior guidance in the development of methodology for remote imagery mapping of natural habitats on seventeen large land parcels in the western Mojave Desert. Conducted field reconnaissance with staff biologists, and assisted in the implementation of field ground-truthing of habitat identifications. This work included both desert scrub plant ecology as well as physical habitat designations.

SolarReserve Habitat Mapping, vicinity of Quartzite, Eastern Mojave Desert of Arizona. Developed criteria and application techniques for the remote imagery analysis of sand-dune habitats on the La Posa Plain of western Arizona. Ground-truthed initial remote imagery interpretations and provided geomorphic model explaining the distribution of different habitats which, in turn, were used to predict the presence/absence of an endangered species.

City of Henderson Landfill Revegetation Plan, eastern Mojave Desert of Nevada. Senior team lead on the preparation an implementation of a revegetation plan for a large site in the Mojave Desert of southern Nevada. Identify plant species best adapted to xeric climate and soils conditions, and those likely to provide the greatest success rate during revegetation. Prepare weed control strategies as well as sensible revegetation strategies using native but nevertheless disturbance-adapted plant species.

Representative Projects (Paleontology)

Ivanpah Solar Electric Generating System EIS/AFC. Conduct records review and literature search, field reconnaissance and subsequent field survey of paleontologically sensitive areas, and recordation of Paleozoic and Quaternary paleontological sites in support of a large solar powered electrical generation facility. Model pluvial lake fluctuations and alluvial fan surface development to determine distribution of paleontologically and archaeologically sensitive sediments. Prepare appropriate paleontological resources sections for BLM EIS and California Energy Commission Application for Certification. Address site formation process in subsequent data request phase.

GWF Energy Tracy Combined Cycle Conversion Project. Performed the paleontological resources literature review and records search, conducted the field reconnaissance, and prepared the AFC Paleontological Resources section for the conversion of an existing peaking plant to a combined-cycle baseload facility consisting of two natural-gas-fired turbines, fired heat recovery steam generators, steam turbine generator, and associated equipment.

GWF Energy Hanford and Henrietta Combined Cycle Conversion Projects. Performed the paleontological resources literature review and records search, conducted the field reconnaissance, and prepared the AFC Paleontological Resources section for the conversion of two existing peaking plants to combined-cycle baseload facilities. The combined cycle facilities included two natural-gas-fired turbines, fired heat recovery steam generators, steam turbine generator, and associated equipment.

Power Plant Licensing and Permitting Program, Calpine Corporation. Paleontological Resources Specialist for several AFCs before the CEC for Calpine's Delta Energy Center in Contra Costa County, and Los Medanos Energy Facility in Santa Clara County as well as AFCs for three peaking power plants licensed under the CEC's emergency AB970 licensing process. Prepared Data Request Responses, attending workshops and providing expert testimony before the licensing hearings. Also prepared preconstruction monitoring plans and provided construction monitoring and compliance services.

AES Highgrove Power Project. Prepared the air quality permits and AFC for 300-megawatt peaking facility consisting of three natural-gas-fired turbines and associated equipment. The project will employ General Electric's LMS100 combustion turbine generators that integrate new technology to increase the combustion turbine's efficiency above existing turbine technologies.

City of Vernon Power Project. Performed the paleontological resources literature review and records search, conducted the field reconnaissance, and prepared the AFC Paleontological Resources section for 914-megawatt baseload facility consisting of three natural-gas-fired turbines and associated equipment.

Paleontological Resources Specialist, Construction-Phase Mitigation Implementation, Multiple Power Generation Projects, California. Develop and manage paleontological resources monitoring and mitigation programs for the construction of power generation projects including the **Humboldt Bay Repowering Project near Arcata**, **Walnut Energy Center south of Modesto**, the **Roseville Energy Park east of Sacramento**, and the **Gateway Generation Station near Antioch**. Prepare the Paleontological Resources Module of the worker education program and visual aids for worker education. Direct the recovery of discovered paleontological resources (Quaternary vertebrate and paleobotanical remains), and consult with client representatives and the California Energy Commission on the adequacy of mitigation efforts. Develop site-specific stratigraphic framework to identify paleontologically sensitive sediments, and to provide client and the CEC with guidance regarding what construction activities need and need not be monitored.

Selected Publications

2008 - A Late Holocene Record of Vegetation and Climate from a Small Wetland In Shasta County, California. (with R. S. Anderson, S. J. Smith, and R. B. Jass). *Madroño* 55(1): 15–25.

W. Geoffrey Spaulding, Ph.D.

2004 - Development of Vegetation in the Central Mojave Desert of California during the Late Quaternary. (with P. A. Koehler and R. S. Anderson). *Palaeogeography, Palaeoclimatology, Palaeoecology* 215:297-311.

2001 – Ploidy Race Distributions since the Last Glacial Maximum in the North American Desert Shrub, *Larrea tridentata* (with K.L. Hunter, J.L. Betancourt, B.R. Riddle, T.R. Van Devender, and K.L. Cole). *Global Ecology & Biogeography* 10: 521-533.

2000 – A Molecular Analysis of Ground Sloth Diet through the Last Glaciation (with M. Hofreiter, H. N. Poinar, K. Bauer, P.S. Martin, G. Possnert, and S. Paabo). *Molecular Ecology* 9: 1975-1984.

1995 - Environmental change, ecosystem responses, and the Late Quaternary development of the Mojave Desert. In *Quaternary Environments and Deep Time: Papers in Honor of Paul S. Martin* (D. S. Steadman and J. I. Mead, eds.), p 225-256. Fenske Printing, Inc., Rapid City, SD.

1990 - Vegetational and climatic development of the Mojave Desert. In *Packrat middens: The last 40,000 years of biotic change*, edited by J. L. Betancourt, P. S. Martin, and T. R. Van Devender, pp. 166-199. University of Arizona Press, Tucson.

1979 - Development of vegetation and climate in the western United States (with T. R. Van Devender). *Science* 204: 701-710.

DECLARATION OF
TODD A. STEWART

I, Todd A. Stewart, declare as follows:

1. I am presently employed by Bright Source Energy, Incorporated as a Director of Project Development and Project Development Manager for ISEGS.

2. A copy of my professional qualifications and experience is attached hereto and incorporated by reference herein.

3. I am adopting the attached testimony on Project Description, Engineering and Noise for the Ivanpah Solar Electric Generating System project based on my independent analysis and my professional experience and knowledge.

4. It is my professional opinion that the prepared testimony is valid and accurate with respect to the issue(s) addressed herein.

5. I am personally familiar with the facts and conclusions related in the testimony and if called as a witness could testify competently thereto.

I declare under penalty of perjury that the foregoing is true and correct to the best of my knowledge and belief.

Dated: November 16, 2009 Signed: Todd A. Stewart

At: Sacramento, CA

TODD A. STEWART, P.E. – BrightSource Energy, Inc., Director, Project Development

PROJECT DEVELOPMENT

Nearly 30 years experience with engineering, construction, business, project management and development, and organizational management consulting. Specific expertise in managing successful development, engineering, construction, maintenance, and operations teams at renewable, gas, oil, and solid fuel power plants and at gas compressor station facilities.

Consulted on environmental compliance, safety, and organizational issues within independent power plants, utility plants, gas pipeline, and corporate service organizations.

Pioneered state-of-the-art project planning and management tools, and has extensive experience in creating, justifying and implementing project plans.

Skilled at project scope definition and justification, financial analysis, schedule creation, maintenance, and forecasting, WBS, cost tracking/forecasting. Also excels at contract administration and negotiation, team creation, subcontractor control, client communication, and defining roles and responsibilities.

EXPERIENCE

DIRECTOR, PROJECT DEVELOPMENT, DEVELOPMENT MANAGER – IVANPAH PROJECT - BRIGHT SOURCE ENERGY, INC., OAKLAND, CALIFORNIA

Accomplishments as Project Development Manager:

Ivanpah Solar Electric Generating System, Mojave Desert California near Primm, NV , ($2.2 Billion)

Managed permitting and licensing processes for the 480 MW ISEGS project in the California Mojave desert, which was processed through a joint BLM/CEC environmental review process. This combined the California CEQA process, with the federal NEPA process and involved working with both CEC and BLM project managers to complete the permitting, licensing, and ROW grant activities.

Key issues encountered in this process included storm water management of the 4000 acre ISEGS site using a low impact development strategy, desert tortoise relocation, competing land uses with a high speed rail project, new electric transmission construction and temporary interconnections, natural gas support issues, and a number of environmental intervener groups.

PROJECT MANAGER and WALNUT CREEK OFFICE MANAGER - PROCESSES UNLIMITED INTERNATIONAL, INC., BAKERSFIELD, CALIFORNIA 2007 - 2009

INFRASTRUCTURE MARKET LEADER - AFFILIATED ENGINEERS, INC. WEST, 2006 – 2007

PRINCIPAL - **WRMS ENGINEERING** 2005 - 2006

Accomplishments

Genentech Central Boiler Plant, ($25M).
Managed detailed design from conceptual and feasibility studies through issue for construction drawings and specifications. Managed APCD permitting activities that included a new BACT determination for ultra low NOx burners and diesel burning in the BAAQMD jurisdictional area. Led the construction administration process assuring that RFI's and Submittals are reviewed and returned in a prompt manner and that the constructors are kept abreast of drawing and design changes.

PROJECT DEVELOPMENT MANAGER, COMPLIANCE MANAGER, BUSINESS DEVELOPER - CALPINE CORP./**WRMS ENGINEERING (CALPINE OWNED WRMS)** 2000 - 2004

Accomplishments as Project Development Manager:

Los Esteros Critical Energy Facility, San Jose, CA. ($320M) - Managed all permitting and licensing processes, including CEC licensing, land rezoning, linear and access easements, governmental relations and community development and relations for this 180 MW plant.

Developed specific compliance and mitigation plans in all areas of CEQA process licensing.

Served as primary witness for applicant during CEC hearings.

Developed preconstruction compliance matrix and managed activities through permitting to construction. Developed and nurtured a very positive working relationship with CEC Project Manager, Robert Worl; CEC Compliance Director, Chuck Najarian; and CEC Compliance Project Manager, Chris Huntley.

Accomplishments: as Project Compliance Manager:

Malburg Generating Facility, Vernon, CA. - Managed the permitting processes for the 134 MW combined cycle Malburg Generating Station (MGS) power plant, post receipt of the CEC decision approving the facility.

Responsible for developing a preconstruction compliance matrix and completing specific activities for the project site and all associated linears in order to achieve an Authority to Construct (ATC) from the CEC.

Managed the day-to-day compliance activities during construction and commissioning of the MGS. Implemented compliance and mitigation plans in all areas of licensing.

Developed and nurtured a very positive working relationship with CEC Compliance PM, Steve Munro; and Compliance Director, Chuck Najarian.

Accomplishments as Project Manager

Landfill Gas GT Cogeneration Project, Northern Power Systems/SC Johnson & Son (Racine, WI). ($15M) Managed the detailed engineering design and construction contract document preparation for a new facility comprising a landfill gas fueled 3.5 MW as turbine, HRSG, gas compressor system. The new cogeneration facility was integrated into the existing SC Johnson manufacturing plant systems mechanically and electrically. The central control system was built to stand alone but provide remote monitoring capability

PRINCIPAL - NPG ENGINEERING (PROJECT/CONSTRUCTION MANAGEMENT**)** 1996 - 2000

Accomplishments:

IC Engine based Gas Compressor Station, PG&E (Holt, CA). Project Manager/Construction Manager - Managed proposal development, all permitting, engineering design, construction, and startup for a four unit IC engine gas compressor station in the environmentally sensitive San Joaquin River delta region.

Gas Service Wells, PG&E (Holt, CA) - Project Manager,
Managed the design and construction of two new natural gas production wells. Project scope development, justification, project team development, permitting support, management of well pad and control system design, construction management.

Plant DCS Installation PG&E (San Francisco, CA) - Project Manager**,**
Installed a DCS system at client's 210 MW steam plant. Managed design engineering team, control of scope, and liaison to operations and maintenance groups at the plant.

Kettleman Gas Turbine Compressor Station Project, PG&E (Avenal, CA). ($33M)- Project Manager,
Managed the design and construction of a 21,000 HP Low NO_x Turbine Gas Compressor Station. Included proposal development for the facility, overall engineering, procurement, construction team management, start-up and commissioning plan development and coordination.

PROJECT/OPERATIONS MANAGER - SANTA CLARA FUEL CELL PROJECT - INDEPENDENT POWER SERVICES, 1995 - 1996

Accomplishments

Project and Operations Manager, Santa Clara Fuel Cell Demonstration Power Plant Project, Fuel Cell Engineering (Santa Clara, CA).

Performed outage management including planning, scheduling (CPM), resource leveling, subcontractor management and contingency planning. Managed implementation of a PC-based maintenance management system.

Interviewed, hired and managed the operators and maintenance technicians for the Fuel Cell Power Plant.

POWER PRODUCTION ENGINEER; PROJECT MANAGER; CONSTRUCTION MANAGER -
PACIFIC GAS & ELECTRIC1980 – 1995

Assignment Locations:

Contra Cost Power Plant 1980-1985
Power Production Engineer
San Francisco Bay Power Plants 1986-1991
Senior Power Production Engineer, Project Manager
Plant Engineer
Supv. Of Maintenance
Pittsburg Power Plant 1991-1993
Construction Site General Manager for Pittsburg and Contra Costa Plants
San Francisco Bay Power Plants 1993-1994
Plant Engineer
General Office 1994-1995
Project Manager – Asset Divestitures

EDUCATION

BS - Mechanical Engineering - South Dakota School of Mines & Technology

PROFESSIONAL AFFILIATIONS

President, California Society of Professional Engineers 2005-Present
House of Delegates Rep for California, National Society of Professional Engineers 2007-Present
Distingished Engineer Recipient, California Society of Professional Engineers, 2009
Member, International Society of Pharmaceutical Engineers

LICENSES

Professional Engineer: Mechanical, California #M23264

DECLARATION OF

John Woolard

I, John Woolard, declare as follows:

1. I am presently employed by BrightSource Energy, In. as the CEO.
2. A copy of my professional qualifications and experience are attached hereto and incorporated herein by reference.
3. I prepared the attached testimony on Project Description for the Ivanpah Solar Electric Generating System project based on my independent analysis, supplements thereto, data from reliable sources, and my professional experience and knowledge.
4. It is my professional opinion that the prepared testimony is valid and accurate with respect to the issue(s) addressed herein.
5. I am personally familiar with the facts and conclusions related in this testimony and if called as a witness could testify competently thereto.

I declare under penalty of perjury that the foregoing is true and correct to the best of my knowledge and belief.

Dated: Nov. 16, 2009 Signed: ______________

At: Oakland, CA

John M. Woolard

PROFESSIONAL EXPERIENCE

BrightSource Energy Inc., Oakland, California **2006-present**
President and CEO

Itron Inc, Alameda, California **2003 to 2005**
Vice President Strategy and Business Development
Vice President- Energy Management Solutions

Silicon Energy Corp, Alameda, California **1998 to 2003**
President, CEO, Chairman of the Board

PG&E Energy Services, San Francisco, California **1997**
Product Manager, Facility Information and Control Systems

Lawrence Berkeley Laboratory, Berkeley, California **1996**
Associate, Energy Analysis Program

Alpine Software Inc., Richmond, Virginia **1994 to 1995**
Vice President, Sales and Marketing

Passages Inc., Richmond, Virginia **1992 to 1993**
President

Institute for Environmental Negotiation, Charlottesville, Virginia **1991 to 1992**
Associate

International Whitewater Guide: **1987 - 1990**
Shearwater Inc., Victoria Falls, Zimbabwe
Steve Currey Expeditions, Santiago, Chile

EDUCATION

University of California at Berkeley, Haas School of Business
Master of Business Administration, May 1997
Price Entrepreneurship Fellowship
Exchange Program: Institut Superieur des Affaires, HEC, Paris France, 1996

University of Virginia
Master of Environmental Planning, May 1992
Bachelor of Arts, Economics, May 1988

Personal/Board Membership:
Executive Committee – Xcel Energy Strategic Advisory Board – Office of CIO (former)
Board of Directors, Treasurer, California Clean Energy Fund (former)
Board of Directors, Treasurer, Tuolumne River Trust Advisory Board
Board of Directors, East Bay Zoological Society (Oakland Zoo)
Aspen Institute Crown Fellow - Aspen Colorado
Sierra Club Lifetime Member

DECLARATION OF
DR. FATUMA I. YUSUF

I, Fatuma Yusuf, declare as follows:

1. I am presently employed by CH2M HILL Incorporated as a Project Consultant.

2. A copy of my professional qualifications and experience are attached hereto and incorporated herein by reference.

3. I prepared the attached testimony on Socioeconomics and Environmental Justice for the Ivanpah Solar Electric Generating System project based on my independent analysis, supplements thereto, data from reliable sources, and my professional experience and knowledge.

4. It is my professional opinion that the prepared testimony is valid and accurate with respect to the issue(s) addressed herein.

5. I am personally familiar with the facts and conclusions related in the testimony and if called as a witness could testify competently thereto.

I declare under penalty of perjury that the foregoing is true and correct to the best of my knowledge and belief.

Dated: ______________________ Signed: November 9, 2009

At: Sacramento, CA

Fatuma Yusuf, Ph.D.
Socioeconomics Task Lead

Education

Ph.D., Agricultural Economics
M.S., Statistics
M.A., Agricultural Economics
B.S., Range Management

Relevant Experience

Dr. Yusuf is an economist and statistician. She has conducted economic analyses for energy, water supply, water quality, agriculture, transportation, and recreation projects; evaluated project feasibility; and assessed economic impacts associated with project implementation. She has experience in preparing the socioeconomic analysis for power plant permitting and other environmental documents, regional economic impact analysis, cost-benefit analysis, and rate impact analysis.

Representative Projects

Ivanpah Solar Electric Generating System, San Bernardino County, California. Socioeconomics Task Lead. Prepared the socioeconomics analysis section of the AFC. Also, analyzed the regional economic impacts of the project on employment and income.

Economic Impact Analysis for the Teanaway Solar Reserve, Kittitas County, Washington. Economics Task Lead. Provided screening-level economic, socioeconomic and fiscal impact analyses of the construction and operation associated with the Teanaway Solar Reserve project in Kittitas County, Washington.

Lodi Energy Center, NCPA; Lodi, San Joaquin County, California. Socioeconomics Task Lead. Prepared the socioeconomics analysis section of the AFC. Also, analyzed the regional economic impacts of the project on employment and income.

Chula Vista Energy Upgrade Project, MMC Energy, San Diego County, California. Socioeconomics Task Lead. Prepared the socioeconomics analysis section of the AFC. Also, analyzed the regional economic impacts of the project on employment and income.

Application for Certification, Eastshore Energy Project, Hayward, California. Socioeconomics Task Lead. Prepared the socioeconomics analysis section of the AFC. Also, analyzed the regional economic impacts of the project on employment and income.

Application for Certification, South Bay Replacement Project, Chula Vista, California. Socioeconomics Task Lead. Prepared the socioeconomics analysis section of the AFC. Also, analyzed the regional economic impacts of the project on employment and income.

Application for Certification for a number of energy projects including the San Francisco Electric Reliability Project in San Francisco, California, and the Walnut Energy Facility in Turlock, California. Socioeconomics Task Lead. Prepared the socioeconomics analysis section of the AFC. Also, analyzed the regional economic impacts of the project on employment and income.

Economic Analysis for the Calpine LNG Facility and Power Plant in Eureka, California. Project Manager. Provided screening-level economic, socioeconomic and fiscal impact analyses of the construction and operation associated with the Calpine LNG and Power Plant Projects in Eureka, California.

Socioeconomic Study Plan for the SMUD Upper American River Project Iowa Hill Pumped Storage Development Project. Socioeconomic Task Lead. Prepared the socioeconomic study plan and evaluated the socioeconomic impacts associated with the Iowa Hill Pumped Storage Development Project as part of the SMUD Upper American River Project Hydroelectric relicensing application. Also, analyzed the regional economic impacts of the project on employment and income.

Revision of SMUD Upper American River Project Socioeconomic Impact Study Report. Socioeconomic Task Lead. Prepared Revision 1 of the SMUD UARP Socioeconomic Impact Study Report on the SMUD Upper American River Project Hydroelectric relicensing. Revision 1 involved the verification of the study conducted by CSUS. Also, analyzed the regional economic impacts of the project on employment and income.

Agricultural Impact Study of the PacifiCorp's Hydroelectric Power Project. Analyzed the socioeconomic and regional economic impacts associated with the increased energy costs faced by Klamath irrigators. Prepared the regional economic impact report.

Industrial Siting Application for a number of energy projects in Wyoming including the Medicine Bow Coal to Liquid Project, Wygen III Unit 5, Seven Mile Hill and Glenrock Wind Energy Projects. Analyzed the regional economic impacts of the projects on employment and income.

Lower Colorado River Authority (LCRA)-San Antonio Water System (SAWS) Water Project (LSWP). Regional Economics Task Lead. Ongoing project. The project aims to develop strategies that would conserve and develop water in the lower Colorado River basin for both regions (LCRA and San Antonio). Strategies include: reducing agricultural irrigation water demand, capturing and storing unused and excess river flows in off-channel storage facilities, and developing groundwater for limited use in agriculture when surface water isn't available. Task is to evaluate the economic impacts associated with changes brought about by the project to satisfy the required legislative finding that the water transfer will protect and benefit the economic well-being of the lower Colorado River watershed and the LCRA water service area. Economic analysis tools to be used include: benefit-cost analysis, input-output analysis, sector analysis, socioeconomic analysis, recreation benefit analysis, and net environmental benefit analysis.

SR 79 Realignment Project Community Impact Assessment (CIA) and EIR/EIS. Economics/Environmental Justice Task Lead. Prepared the socioeconomics and environmental justice analysis sections of the Draft CIA and EIR/EIS for the SR 79 Realignment Project Domenigoni Parkway to Gilman Springs Road.

BEFORE THE ENERGY RESOURCES CONSERVATION AND DEVELOPMENT COMMISSION OF THE STATE OF CALIFORNIA
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APPLICATION FOR CERTIFICATION
FOR THE *IVANPAH SOLAR ELECTRIC GENERATING SYSTEM*

DOCKET NO. 07-AFC-5
PROOF OF SERVICE
(Revised 7/20/09)

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DECLARATION OF SERVICE

I, Mary Finn, declare that on September 10, 2009, I served and filed copies of the attached, Applicants Testimony dated November 16, 2009. The original document, filed with the Docket Unit, is accompanied by a copy of the most recent Proof of Service list, located on the web page for this project at: [www.energy.ca.gov/sitingcases/ivanpah].

The documents have been sent to both the other parties in this proceeding (as shown on the Proof of Service list) and to the Commission's Docket Unit, in the following manner:

(*Check all that Apply*)

FOR SERVICE TO ALL OTHER PARTIES:

__x__ sent electronically to all email addresses on the Proof of Service list;

__x__ by personal delivery or by depositing in the United States mail at Sacramento** with first-class postage thereon fully prepaid and addressed as provided on the Proof of Service list above to those addresses NOT marked "email preferred."

AND

FOR FILING WITH THE ENERGY COMMISSION:

__x__ sending an original paper copy and one electronic copy, mailed and emailed respectively, to the address below (*preferred method*);

OR

______ depositing in the mail an original and 12 paper copies, as follows:

CALIFORNIA ENERGY COMMISSION
Attn: Docket No. 07-AFC-5
1516 Ninth Street, MS-4
Sacramento, CA 95814-5512
docket@energy.state.ca.us

I declare under penalty of perjury that the foregoing is true and correct.

Mary Finn

**or by other delivery service, e.g., Fed Ex, UPS, courier, etc.

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