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Pacific Gas and Electric Company®

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Eureka, CA 95503-6859

HBGS-CEC-100

June 1, 2015

Mr. Jonathan Fong
Compliance Project Manager
California Energy Commission
Siting, Transmission, and Environmental Protection Division
1516 Ninth Street, MS 2000
Sacramento, California 95814-5512

Subject: Humboldt Bay Generating Station – Submittal of HBPP Hydrology Report (Stormwater Management Plan) for the Site Boundary Petition for Project Modification (06-AFC-07C)

Dear Mr. Fong:

On May 7, 2015, PG&E submitted a petition for project modification for the Humboldt Bay Generating Station (HBGS) to modify the HBGS site boundary once decommissioning and restoration of the adjacent Humboldt Bay Power Plant (HBPP) are complete.

The petition referenced a stormwater management plan (attached), which will be implemented as part of restoration of the HBPP site. This plan also satisfies Condition 12 of the HBGS 401 Water Quality Certification and was submitted to the North Coast Regional Water Quality Control Board in compliance with that condition. Once HBPP site restoration is complete and the HBGS site boundary modification is enacted, HBGS will manage the stormwater management system described in the attached report.

In addition, the Final Site Restoration Plan (Coastal Resources Assessment) which was submitted to the California Coastal Commission as part of the Coastal Development Permit amendment application for HBPP restoration and included in the petition as Appendix A, did not include the supporting engineering drawings. These drawings were being revised based on changes to the HBPP Final Site Restoration Plan resulting from Coastal Commission feedback. These drawings are now complete and included herein as an attachment.

The revised engineering drawings include:

- Site Plan (Sheet 1 of 6)
- Grading & Drainage Plan (Sheet 2 of 6)
- Water & Sewer Plan (Sheet 3 of 6)
- Electrical Site Plan (Sheet 4 of 6)
- Traffic Flow Plan (Sheets 5 and 6)

Please note that these revised engineering drawings are consistent with the calculations of restored areas/mitigated areas that are included in the Final Site Restoration Plan.

Should you have any questions, please contact me at 707-269-1810.

Sincerely,

Scott Washington
Environmental Compliance Manager
Humboldt Bay Generating Station

attachments

WHITCHURCH ENGINEERING

Humboldt Bay Power Plant Final Site Restoration Hydrology Report

REGISTERED PROFESSIONAL ENGINEER DARREN P. TULLY No. 73755 CIVIL STATE OF CALIFORNIA

REV. A

Prepared by: Brad Wilson, P.E. ____ 5/28/15

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Approved by: Jeff Laikam, P.E. ____ 5-28-15

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1. Executive Summary

1.1. Regulatory Context

The final phase of the decommissioning effort at the Pacific Gas and Electric (PG&E) owned Humboldt Bay Power Plant (HBPP) is the implementation of the Final Site Restoration (FSR) plan. Completion of decommissioning and implementation of the FSR plan requires the permitting and approval from Federal, State and local agencies. The intention of this engineering report is to convey the analysis process employed to design the major aspects of the proposed storm water management system at HBPP in accordance with the Low Impact Development (LID) requirements of the North Coast Regional Water Quality Control Board (NCRWQCB).

In addition, this report was prepared to comply with Condition 12 of PG&E's Humboldt Bay Generating Station (HBGS) 401 Water Quality Certification issued in October 2008 by the NCRWQCB. This condition requires that a post-construction storm water management plan for the HBPP site be submitted to the NCRWQCB. The HBGS 401 Certification was issued prior to the development of HBPP decommissioning plans. The NCRWQCB wanted to ensure that HBPP storm water management was addressed as part of decommissioning so a permit condition regarding the HBPP site was included as part of the HBGS 401 Water Quality Certification.

1.2. Storm Water Treatment and LID Features

The proposed FSR storm water management system at HBPP complies with governing LID principles of the NCRWQCB by utilizing seven customized bio-detention basins to capture and treat runoff generated by the proposed FSR configuration. This unique basin design was developed in coordination with the NCRWQCB and represents a combination of LID Best Management Practices (BMPs) that will adequately treat all of the onsite storm water runoff for reuse in the adjacent wetland habitats that surround HBPP. A conceptual cross section and location of these proposed basins is shown on page 2 of Attachment 1 and Attatchment 2.

These basins have been designed to comply with NCRWQCB recommendations, and the conditions of the Industrial General Permit (IGP) which will govern the maintenance of these features into the future. In accordance with NCRWQCB methods, which account for the specific cover types of the proposed FSR site plan within each basin's watershed; these basins have been designed to capture 150% of the minimum required volume of storm water runoff generated by the recommended 85th percentile, 24-hour design storm. In addition to features shown in page 2 of Attachment 1 and Attatchment 2, the basins feature a unique LID water treatment system which utilizes layers of soil, sand, and drain rock, concealing a network of perforated pipes, to simultaneously physically filter and bio-remediate storm water as it passes through the filter media and out of the perforated pipes into adjacent wetlands. The implementation of this proposed bio-detention basin design provides all the treatment necessary for the HBPP site to adhere to LID principles and improve the quality and ecology of adjacent habitats.

To ensure a robust basin design for these unique features, this report includes the use of 62 years of local, historical, hourly precipitation data (~560,000 data points) to model basin performance under observed historical storms of interest which span days, weeks, and months. This allows time sensitive relationships relevant to the bio-detention basin design and

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performance, such as accumulated volume, to be investigated and assessed. Utilization of this additional data and analysis informs the future construction detailing effort of critical considerations to overall design and performance with much greater resolution than would be possible with application of simple traditional methods.

1.3. Peak Flows and Design of Storm Water Conveyance Systems

The proposed HBPP FSR grading plan is shown in page 2 of Attachment 1; it features extensive alterations to large portions of the HBPP site while other areas are unmodified from current topography. The alterations to surface topography necessitated the installation of a new network of surface flow features to convey storm water runoff from the upper reaches of watersheds to their respective bio-detention treatment basin. Portions of the existing storm water conveyance system that can be integrated into the proposed system will be retained. The proposed FSR storm water management system routes all storm water runoff through one of the seven proposed bio-detention basins. To work within the site constraints this route necessitates low slope surface features (0.25% - 0.5%) and excludes the use of subterranean pipes.

All aspects of the proposed storm water conveyance system presented on the HBPP FSR grading plan, page 2 of Attachment 1, can convey 100% of the runoff predicted from a 10-year design storm with no anticipated flooding. The substitution of larger magnitude design storms results in some minor areas of surface inundation and temporary flooding. All techniques used to generate estimates of peak design flows and size conveyance systems employ several levels of conservatism and produce conservative (high) values of anticipated runoff quantities.

1.4. Results, Conclusions and Recommendations

This report, documenting preliminary design and sizing of the storm water management system for the proposed HBPP FSR plan, has identified numerous critical design considerations and preliminary results for guiding the construction detailing process. Detailed discussions of the following conclusions are included in referenced sections of this report.

- The adoption of the proposed bio-detention basin design satisfies all the necessary storm water capture and treatment criteria to comply with NCRWQCB's recommendations, with no additional LID features required; see Sections 3.2.2 through 3.2.4 and Section 4.8.
- Overall performance of the proposed bio-detention basins is heavily influenced by the infiltration rate of the proposed soil/sand/rock filter media. Faster infiltration rates provide increased quantities of storm water being physically filtered, but may not provide adequate levels of bio-remediation. During construction, the constituents of concern within the storm water runoff should be identified and the infiltration rate specified to provide an appropriate balance between physical filtering and bio-remediation; see Section 4.11.
- Bounding the possible proposed FSR site grades between the finished floor elevation of preserved buildings, adjacent grades at HBGS, and the relatively high inlet elevations of treatment basins, necessitates the adoption of fairly flat (0.25% - 0.5%) surface swales and excludes the use of subterranean pipes; see Section 4.7.
- All proposed drainage conveyance features adequately transport 100% of the peak flows associated with the 10-year design storm. The same storm water conveyance

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system subjected to 25-, 50-, and 100-year design storms exhibits minor quantities of surface inundation and temporary flooding of localized areas; see Section 4.7.

- When compared to the maximum historic build out of HBPP, prior to starting decommissioning (2003), the proposed FSR plan achieves a net reduction in impervious surfaces of 0.80 acres. This net reduction accounts for deliberate overestimation of proposed impervious areas, including the full build out potential of the Unit 1/2/3 laydown area, and potential future expansion of the ISFSI truck turnaround; see Sections 4.4.1 and 4.4.3.
- Use of an expanded historical data set permitted detailed peak flow analysis and assessment of the proposed bio-detention basins' performance under observed historical precipitation events. Average results range from 93% to 32% of the inflow runoff being infiltrated through the filter media of the basins; additional details can be found in Section 4.11.

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2. Introduction and Project Setting

2.1. Project Description

PG&E is currently decommissioning the HBPP and will be requesting termination of their Nuclear Regulatory Commission (NRC) license to own and operate a nuclear reactor. PG&E operated the HBPP between 1956 and 2010. The power plant consisted of two steam generating units (Units 1 and 2) and a boiling water nuclear reactor (Unit 3). The two steam generating units began operation in 1956 and 1958, respectively, and were shut down in 2010. The nuclear unit operated between 1963 and 1976 and entered the SAFSTOR in 1985 and began the decommissioning and permitting process in 2005. In 2010, the Humboldt Bay Generating Station (HBGS), located on the same property, came on line to replace the former generation capacity of Units 1, 2, and 3. The decommissioning of Units 1 and 2 was completed in 2011, leaving Unit 3 as the final component for decommissioning. Most demolition work will be done in 2017, with a small portion into 2018, and site leveling and final status surveys completed by mid-2019.

After the completion of the decommissioning of Unit 3, the HBPP site will enter FSR to: return portions of the industrial site to predevelopment conditions; enhance adjacent habitat and ecology; implement mitigation measures which will compensate for the decommissioning process; and provide remaining lines of business with a site suitable for intended uses. The FSR project area is a working facility and industrial in nature and includes paved areas, numerous buildings, outbuildings, and associated industrial facilities required to support the generation of power. Since the project site is currently in the phase of active decommissioning, site cover and configuration are constantly changing; the primary facilities existing at HBPP at the end of 2013 are shown on Figure 1. The proposed site configuration after the implementation of FSR is shown in Attachment 1.

2.2. Project Location and Climate

The HBPP is located on the northern California coast in Humboldt County, approximately 4 miles southwest of Eureka. PG&E owns 143 acres in the King Salmon region; parcel splits and omission of unusable regions like Humboldt Bay and adjacent wetland preserves reduce the total project site acreage to 74.9 designated as the owner-controlled area relevant to FSR, Figure 1.

The City of Eureka, with a population of approximately 26,000, is the largest population center in Humboldt County. The Eureka-Arcata-Fortuna metropolitan area has a population of approximately 135,000. Several small, residential communities are located within 5 miles of HBPP including King Salmon, Humboldt Hill, Fields Landing, and the suburban communities surrounding Eureka. King Salmon is west of the site, adjacent to the site location, and Fields Landing is approximately 0.4 mile south.

The HBPP site is located on a small peninsula known as Buhne Point, nominally at 12 feet above mean lower low water (MLLW), and rising to a promontory about 64 feet above sea level. The site is above the surrounding floodplain and wetland areas of Humboldt Bay and lies between the North Coast Railroad Authority (formerly the Northwestern Pacific Railroad) tracks and the north shoreline of Buhne Point. The HBPP site is not traversed by a public highway or a railroad. The only access to the site is from the south through King Salmon Avenue, which also serves

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the King Salmon community on the western part of the peninsula. Several boat landings in King Salmon are just west of the entrance gate to the PG&E-controlled area. King Salmon serves frequent commercial and recreational boat traffic. A public access trail runs along the shoreline and along the fence to the northwest of the PG&E-controlled area.

The climate of the greater Humboldt Bay region, including Eureka and the immediate coastal strip where the project site is located, is characterized as Marine West Coast climate. The average annual temperature is 51 degrees Fahrenheit (°F), with the warmest months from July to September and the coldest months from December to February. The rainy season generally falls between November and March, with an average annual rainfall of 39.38 inches over the 62-year record at Eureka, and a maximum recorded annual rainfall of 81.87 inches. The wind is predominantly from the north to northwest, with a shift to the south to southeast during the winter months.

Several rivers and creeks drain the region around HBPP, including Mad River, which flows west approximately 15 miles northeast of the site, and Eel River, which discharges into the Pacific Ocean approximately 8 miles south of the site. The Elk River discharges to Humboldt Bay about 1 mile northeast of the site. Buhne Slough drains areas adjacent to and south of the HBPP site and runs through PG&E property and south of the HBPP, before draining into the Fisherman's Canal.

2.3. Regulatory Context and Special Conditions

The intention of this engineering report is threefold:

- To inform stakeholders of the proposed Final Site Restoration (FSR) with regard to the storm water runoff and treatment methods and relevant supporting hydrologic details;
- Comply with Condition 12 of the HBGS 401 Water Quality Certification; and
- Upon approval by the relevant agencies, this report will provide a critical base to inform future efforts of moving from a conceptual design and permitting phase into construction detailing and contract development.

This generic storm water management plan provides critical engineering and design information to the involved stakeholders from multiple agencies; this report represents the resolution to a hybridization of requests and permit conditions from the NCRWQCB, the United States Army Corps of Engineers (USACE), the California Coastal Commission (CCC), the California Energy Commission (CEC), and additional local agencies. Also portions of the HBPP site will be taken into HBGS ownership and will thereby come into the regulatory jurisdiction of the CEC and the HBGS CEC license. Portions of the property will remain outside of the HBGS boundary.

The CCC has issued numerous Coastal Development Permits (CDPs) authorizing the activities associated with demolition of power generating Units 1, 2 and 3; conducting site remediation activities; and terminating the NRC license. These CDPs include a provision for PG&E to prepare an FSR plan of the HBPP site, including where power generating Units 1, 2, and 3 and associated buildings, storage facilities, and appurtenant structures, once stood. These areas will be restored to repurpose the former HBPP area to support the operating HBGS and potential future power generation-related activities on the property. Areas already committed for other operational needs, such as the Independent Spent Fuel Storage Installation (ISFSI), will

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continue. PG&E has worked closely with the CCC and other regulatory agencies to ensure necessary permits and approvals are in place to meet the requirements of the overall decommissioning project.

2.4. Description of Adjacent Wetlands and Waters of the State

The PG&E HBPP site is bordered on two of three sides by salt marsh habitats, and by the waters of Humboldt Bay on the third side, Figure 1. Several Federal and State agencies are considered stakeholders for these waters and habitats and provide regulatory guidance for development which may directly or indirectly affect such waters and habitats.

Storm water discharges from the proposed HBPP FSR plan will outlet into the Buhne Point Wetland Preserve on the southern edge of the site, the former HBPP intake canal on the eastern edge, and the wetland areas to the northeast of the site. These include some freshwater wetlands with intertidal and brackish water influences. For further details regarding the jurisdictional status of these wetlands and exact extents and habitat qualities see the report titled *Preliminary Wetland Delineation for the PG&E Humboldt Bay Power Plant Final Site Restoration Plan* (Stillwater Sciences, 2015).

The captured and treated storm water runoff from the HBPP site will be reused to recharge the wetlands, thereby enhancing their ecological functions and values. Providing a means of metering and treating the storm water runoff will have several beneficial effects to the ecology of these wetlands:

- Storm water will be detained, filtered and treated through the proposed bio-detention basins;
- Peak flows entering the wetlands will be metered and dampened by the proposed bio-detention basins;
- Fresh water flushing directly into the adjacent saltwater ecology within the intake canal will be minimized by removing non-essential storm water discharges directly into the intake canal and re-routing them through the Buhne Point Wetland Preserve.

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3. Low Impact Development (LID) Design Goals and Features

3.1. LID Design Goal for Storm Water Treatment

The NCRWQCB provided Whitchurch Engineering Inc. (WEI) with the City of Santa Rosa LID Design Manual for design of the FSR at HBPP. This LID manual outlines numerous LID Best Management Practices (BMPs) and methodologies for promoting onsite infiltration and management of storm water runoff.

The manual includes methods for providing storm water management for a project's entire site, but also has provisions to account only for new impervious regions installed during implementation of a development plan. As discussed in Sections 4.4.1 and 4.4.3, the HBPP site will undergo a net reduction in impervious area by implementing the FSR plan. Therefore the proposed FSR project would have been a candidate for adopting a management strategy based on only offsetting the impervious area associated with implementation of the FSR plan. However, PG&E has voluntarily adopted the more stringent treatment strategy. The FSR storm water management plan provides capture and treatment for 100% of the final surface configuration, in accordance with the supplied LID manual.

Per the supplied LID manual, capture and treatment of 100% of the volume of storm water runoff generated by the 85th percentile, 24-hour storm event achieves treatment levels acceptable to the NCRWQCB. In accordance with the supplied LID design manual, fulfillment of this goal will provide all the required storm water treatment for 100% of the entire site, not just the regions affected by implementation of the FSR plan. In addition, as discussed in Sections 4.4.1 and 4.4.3, regions of impervious cover which may be expanded or added to in the future (i.e. expansion of the ISFSI truck turn around for cask transport), were assumed to occupy their maximum extents for the purposes of sizing the basins; this assumption ensures basins will be adequately sized for maximum potential development of the retained industrial portions of the site.

3.2. Identification of Desired LID Storm Water Treatment Features

The HBPP FSR plan proposes that 100% of the storm water runoff produced by the entire HBPP site will be routed through surface swales and subterranean piping systems before being captured and treated in appropriately sized bio-detention basins. These bio-detention basins, as detailed below, will provide all the detention and treatment required to comply with LID design principles and satisfy the NCRWQCB's expectation for storm water treatment and management. These basins are strategically located to intercept all of the runoff produced throughout the site, treat it through a combination of bio-remediation and infiltration, before releasing it to recharge the adjacent wetlands and water bodies.

3.2.1. Identification and Adoption of LID BMPs and Pollution Prevention Measures

The LID manual supplied to WEI by the NCRWQCB for the design of FSR at HBPP approaches the implementation of LID principles by dividing LID BMPs into six tiers of preferred implementation, with preferred BMPs occupying the lower tiers and less preferred BMPs occupying higher tiers. The HBPP site has a number of site restrictions which exclude adoption of some of the lowest tier options which emphasize 100% infiltration of storm water into the soils directly below the proposed project. 100%

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infiltration is neither feasible nor desired for the HBPP FSR plan for the following reasons:

- Slow infiltration rates of sub-soils within the project boundary and a high groundwater table make 100% infiltration unreliable, see Section 4.11.3;
- Infiltration of runoff, even if it would function efficiently, would not provide as much ecological benefit as routing it through the existing wetland surface features; and
- Performance requirements associated with maintaining status as an operating industrial power generation facility necessitate certain degrees of site connectivity and surface coverage that limit the extent of feasible infiltration area.

For these reasons, the adoption of second tier LID BMPs is proposed for the HBPP FSR plan. The HBPP FSR plan proposes the creation of a unique LID feature representing a combination of standard BMPs shown in the supplied manual. As indicated above, these proposed LID features take the form of bio-detention basins and combine the treatment features of the LID manual's second tier rain garden with the scale and capacity of the LID manual's second tier constructed wetland.

3.2.2. Conceptual Development of the Proposed Bio-detention Basins

The proposed bio-detention basins are on the scale of a constructed wetland, but feature a layer of sand/soil filter media above a network of perforated pipes to facilitate discharge of treated storm water into adjacent wetlands and water bodies, page 2 of Attachment 1. The proposed bio-detention basins will feature the desired capture volume, as determined in accordance with the LID manual, above the soil/sand filter media. Storm water runoff will enter and accumulate in a proposed bio-detention basin, before being passively filtered and treated through a combination of physical and biological processes, as it migrates through a layer of sand/soil filter media below.

This unique proposed bio-detention design was reviewed with representatives of the NCRWQCB to ensure the design was acceptable. This design is ideal for the HBPP FSR plan as it:

- Aligns with PG&E's environmental management goals by providing treatment of storm water discharges for the entire site;
- Complies with and exemplifies LID methods and the expectations of the NCRWQCB;
- Provides a single type of treatment features for implementation across the site, simplifying maintenance and avoiding impacts to the desired site uses; and
- Provides 100% of the onsite storm water capture and treatment, so no other LID BMPs are required; additional BMPs currently in use could continue to be used as deemed appropriate, see Section 5.
- Additionally, there are several vegetated swales that will be utilized to transport surface flows and will provide additional pre-treatment.

A total of seven bio-detention basins will provide the treatment for the entire site, Attachment 1, and Attachment 2. Three large basins provide the majority of treatment to the HBPP core region. Two of these basins are hydraulically connected in a manner

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which will enable them to function as a single large basin; these basins cannot be joined because there are existing utilities which cross this area and need to be preserved. There are four additional basins which are comparatively small; these basins provide treatment for water being released from the access roadways, Attachment 1 and Attachment 2.

3.2.3. Features of the Proposed Bio-detention Basins and Storm Water Management System

The basins will have the same essential features as specified in the LID manual and coordination with representatives of the NCRWQCB. As shown on page 2 of Attachment 1, the proposed bio-detention basins will feature:

- Storage volumes capable of capturing 150% of the 85th percentile 24-hour storm event for their respective catchment areas;
- A one- to two-foot-thick layer of soil/sand/rock filtration media that occupies the entire base course of the basin and filters the storm water prior to discharge;
- A system of interconnected perforated pipes below the layer of filtration media that collects filtered/treated storm water and discharges it into the adjacent wetlands and/or waterbodies;
- An adjustable flow control structure at the discharge location capable of refining the outflows observed into the adjacent wetlands and/or waterbodies;
- Native plants and micro-biology to support the bio-remediation of storm water pollutants;
- A stage-storage stick calibrated to each basin capable of indicating total accumulated volumes and volumes of seasonally accumulated sediment;
- 10-foot-wide grassed maintenance access roads around the perimeter and into the bottom of each basin to facilitate maintenance. (The 4 small basins do not provide access as they are adjacent to proposed roadways and small enough to be easily maintained from a single point);
- A means to completely drain each basin to facilitate maintenance; and
- An appropriately sized high flow bypass to ensure large storm events do not cause flooding of upstream regions or compromise the basin structure.

Several of the noted features are preliminarily designed and described in this report, however, further construction detailing and specification will identify these features in greater detail and define their operational bounds and parameters; these features include:

- The high flow bypass - For this cursory phase of design the magnitude of the peak flows utilizing the bypass within each basin have been predicted and will be discussed further in Section 4.11.1 of this report;
- The variable metered outflow - The specification and development of the precise metered outflows and their associated: range, flow rates, and extent of variability; and
- The precise planting plan within each basin to achieve adequate storm water treatment.

There are no other storm water treatment systems proposed for implementation of the HBPP FSR plan; BMPs currently in use on the HBPP site will continue after the

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completion of FSR, as applicable, and no new BMPs will be required beyond those required for the management of the bio-detention basins, see Section 3.3. A few existing portions of the storm drain system will be retained and rerouted into the proposed basins; however the majority of the existing drainage infrastructure will be removed and replaced with surface flow features. Surface flow features are being adopted because the relatively flat portions of the site only have approximately two feet of fall to the required inlet elevation of the bio-detention basins, Section 4.7. This excludes the use of subterranean pipes or other means of storm water conveyance that would require additional depth. In addition, surface flow features have beneficial properties which include: slowing runoff down, creating opportunities for infiltration to occur, and providing ease of visual inspection. Detailed discussion of the design of these proposed surface flow features can be found in Section 4.7 of this report.

3.2.4. Sizing of the Proposed Bio-detention Basins

Per the supplied LID manual, the basins are sized to intercept at least 100% of the volume produced by the 85th percentile 24-hr storm event to achieve the desired LID. As discussed in Section 4.1, 0.68” was identified as the total precipitation associated with the 85th percentile 24-hr storm event. As described in Sections 4.8, this precipitation value was used with the prescribed NRCS curve method outlined in the manual to determine the minimum volume required for capture and treatment. Detailed discussion of the design and sizing of these proposed bio-detention basins can be found in Sections 3.2 and 4.8 of this report.

In addition, to support the future adoption of the proposed HBPP FSR storm water treatment systems into the HBGS IGP/SWPPP (Storm Water Pollution Prevention Plan) upon termination of the HBPP construction SWPPP, the minimum required volumes determined in accordance with the LID manual have been increased by 150%. This increase to the minimum LID requirements will provide the factor of safety necessary for transition onto HBGS’s IGP. HBGS’s IGP will eventually be the governing storm water management procedure for these proposed storm water management features.

3.3. SWPPP BMPs and Pollution Prevention Measures

Governing BMPs and Pollution Prevention Measures will be implemented in accordance with the governing SSWPPP and IGP in effect at HBGS at the time of FSR completion; BMPs currently employed at HBPP will be carried into the transition and continue after FSR, as deemed appropriate by qualified individuals executing revisions.

As part of a state wide mandate, HBGS is adopting a revised site specific SWPPP in July of 2015; this SWPPP includes BMPs which encompass all of the generic site features and activities anticipated at an industrial power generation site. Although this revision to the SWPPP makes no specific accommodations for BMPs at HBPP, nor does it facilitate the incorporation of the HBPP site after FSR, it is anticipated that due to the similar site uses and storm water management features, the BMPs applied at HBGS will encompass the drainage features at HBPP after restoration (e.g. DI inspection, good housekeeping, etc.). The only unique storm water management and treatment system proposed as part of implementing the HBPP FSR plan are the bio-detention basins which will be maintained as outlined in Section 5. These unique drainage features will depart from typical BMPs; Section 5 provides the details

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necessary to incorporate these features into the HBGS SWPPP at the completion of FSR. Other generic BMPs like spill prevention and good housekeeping will be adopted in accordance with the governing HBGS SWPPP and are not detailed herein.

There are no known or anticipated sources of storm water pollution that will be present after the completion of FSR which are not already described in the HBGS SWPPP; therefore the set of BMPs contained in the current HBGS SWPPP will be appropriate for expansion to the regions of HBPP turned over to HBGS control at the completion of FSR. Due to file size, a copy of the SWPPP currently in use at HBGS, and the BMPs therein, is available upon request.

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4. Calculations

4.1. Identification of the 85th Percentile, 24-hour Precipitation Depth

To establish a robust model of the watersheds proposed in the HBPP FSR plan the largest available historical data set was evaluated. Multiple sources provide the critical value for bio-detention basin sizing in Humboldt County; these sources list the average depth of the 85th percentile 24-hour storm event as 0.65 inches. This number represents the average depth of this critical design storm event over the entire region of Humboldt County which is large, 4,052 square miles (USCB, 2015), and highly variable in both topography and observed precipitation quantities and intensities, as indicated in Figure 2 (NOAA, 2014). Due to the variability and uncertainty introduced by use of a generic average depth of the 85th percentile 24-hour storm event; Whitchurch Engineering Inc. (WEI) conducted an independent analysis of the available data for a weather station nearby HBPP. The adopted National Oceanic and Atmospheric Administration (NOAA) weather station is the Eureka Weather Forecast Office on Woodley Island, CA (Network ID: COOP_042910); Located approximately 5.4 miles to the North of HBPP, Figure 3. This weather station is similar to HBPP in exposure, elevation, and proximity to the ocean, making it an ideal source of data representative of precipitation events at the HBPP site.

The longest duration of available data for the Eureka weather station was downloaded from the National Climatic Data Center (NCDC). The raw unsorted data begins in 1893 and runs to the end of 2013. Data was purged to represent the 64 water years from 1949 to 2012 (10/1/1948 – 9/30/2012); this represents the longest span of available data where the use of a tipping bucket rain gauge gave reliable hourly total precipitation values reported in hundredths of an inch. The data set was purged again to remove the error values, represented by 99999 values in the precipitation column, and expanded to populate a continuous hourly calendar spanning the full date range, this represents approximately 561,000 hours. This complete data set represents the actual hourly rainfall experienced at the Eureka weather station on a continuous hourly timeline for the full duration of 64 water years, Figure 4; due to file size the data set is available upon request.

To identify the relevant design criteria, this historical data set was further modified to purge zero values of no recorded precipitation and isolate the desired percentile design storms. The 49,805 hourly data points were ranked from highest observed value (5.03 in.) to the lowest observed value (0.01 in.) and graphed according to exceedance probability, Figure 5. This allows the relevant precipitation values to be identified according to their exceedance probability; the relevant hourly design values are shown in Table 1.

To identify the desired 24-hour, 85th percentile storm depth, the hourly data for each 24-hour day was summed and reported as the 24-hour rainfall total. All 24-hour periods with at least 0.01 inches of rain were retained in the data set; this method is consistent with the recommendations of the EPA for determining 24-hour design storm events (EPA, 2009).

This consolidation of the available data produced 7,651 data points representing accumulated 24-hour precipitation totals (in). Similar to the hourly data, the 24-hour precipitation totals were ranked from the highest observed value (6.73 in.) to the lowest observed value (0.01 in.) and graphed according to exceedance probability, Figure 6. This allows the relevant daily

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precipitation values to be identified according to their exceedance probability; the relevant daily (24-hour duration) design values for various percentile thresholds are shown in Table 2.

The critical design value of the 85th percentile, 24-hour design storm was identified as 0.68 inches; this value closely confirms the published 85th percentile, 24-hour design storm value of 0.65 inches. For the purpose of preserving conservative designs of adequate capacity the slightly large 0.68 inch value of the 85th percentile, 24-hour storm will be adopted for this analysis of the proposed HBPP FSR storm management system.

4.2. Identification of Historical Monthly Design Storm Events of Interest

Similar to the process used to identify and rank the hourly and daily precipitation total described above, the data set was further consolidated to represent monthly precipitation totals. The daily precipitation values were added according to their month of occurrence; these monthly totals were ranked from the highest (23.31 in.) to the lowest (0.01 in.) observed value and graphed according to exceedance probability, Figure 7. This allows the relevant monthly precipitation values to be identified according to their exceedance probability. The relevant monthly design values for various percentile thresholds are shown in Table 3. These design months represent the actual variable precipitation observed at the data collection point; they can be used to model the proposed bio-detention basin's performance given variable precipitation values through time, not just a single value and moment as in standard methodology. Their percentile threshold is based on total accumulated precipitation and does not account for variation in storm intensities; therefore lower percentile months can exhibit shorter periods of more intense precipitation than higher percentile periods. It is assumed that this variation in possible intensity is accounted for by analyzing multiple percentile events, as the culmination of all these events provides a fair range of possible intensities, Section 4.9 provides additional discussion.

4.3. Advantages of Additional Historical Data Analysis

The further consolidation and manipulation of the historical data set allows for detailed modeling of the performance of the watersheds and their respective bio-detention basins through time as if they had been subjected to actual historical storms of interest. This is a significant expansion of the typical design methodology for storm water management systems that rely on singular design values for critical flows at only a single point in time. This expansion of design capability is ideal for the design of the proposed bio-detention basins because it will enable the theoretical accumulated volume within each basin to be determined at any point during an actual historical storm event while accounting for the upstream characteristics of the storm water conveyance system and changes in the design parameters at the proposed bio-detention basins.

Tracking the accumulated volume within each bio-detention basin enables the basin's performance to be evaluated when subjected to variability of inputs such as:

- Fluctuating intensities and durations of storm events;
- Adjustments to the cover types within each watershed;
- The characteristics and response times of upstream drainage features;
- Identification of total basin capacity;
- Potential infiltration rates of the proposed soil/sand filter media; and
- Average treatment and detention times.

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This approach to drainage and basin design provides additional verification that the drainage features and basins are appropriately sized to perform as desired given subsequent storms and actual historical distributions of precipitation over multiple days, weeks, and months, not just a single event or design value. These details and findings of these analysis variables are discussed further in Section 4.9.

4.4. Determination of Peak flows

Determination of peak flows (Qp) within and leaving each watershed, and sub-shed therein, contributing to each basin was executed using methods outlined in the TR-55 Urban Hydrology for Small Watersheds Manual. This standard methodology, generically referred to as the CIA method, generates estimates of Qp (cfs) by combining the overall area (A) and associated cover types (C) of a specific watershed with site specific precipitation Intensity (I) values in a simple multiplicative relationship. To utilize this methodology, a detailed hydrology map and routing schematic, Attachment 2, of all the watersheds within the region of analysis is required.

The Hydrology map and schematic represents several critical components to implementing the CIA method which include:

- The basic geographic and spatial distribution of watershed locations and relative sizes;
- The boundary of each watershed and sub-shed created by the proposed FSR grading plan;
- The total area within each watershed covered by a respective surface;
- The proposed or existing drainage structures conveying runoff;
- The critical design inputs for each of the proposed drainage structures;
- The routing of runoff from the upper reaches of watersheds through intermediate sheds down to the respective bio-detention basin; and
- The compositing of watersheds which results in accumulated flows as runoff approaches a bio-detention basin.

The methods employed for determining each of the variable inputs to the CIA calculation are outlined below for both individual watersheds and composite watersheds (those watersheds which represent a conglomeration of individual watersheds). The respective values of: C, I, A, Time of Concentration (TOC), discrete travel times for various flow types within watersheds, areas of specific cover types, and predicted Qp values for each return period analyzed for all of the watersheds within the analysis region are shown in Table 4 through Table 12.

4.4.1. Comparison of Existing and Proposed Impervious Areas

The HBPP site has undergone extensive incremental changes to site cover and use over the past 60 years providing for some subjectivity in the identification of the “existing” conditions. For the purposes of this report the “existing” impervious area for this project will be adopted as the site cover present in 2003. This year represents the maximum site build out associated with the historic site prior to the installation of HBGS, the ISFSI and the numerous temporary structures installed to facilitate decommissioning of units 1, 2, and 3. The appropriateness of this choice was verified with representatives of the NCRWQCB.

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For the purposes of this analysis of cover types, the outer PG&E parcel boundary was adopted as the boundary of comparison, 101.96 acres; since the outer boundary encompasses the full extent of site configurations in both 2003 and 2018 its adoption provides a baseline for standard comparison. It should be noted, that the curve numbers produced by this assumption do not typify the site as a whole, as the boundary includes regions which do not generate runoff like existing wetlands and even portions of Humboldt Bay; the adoption of this boundary is only valid for the purposes of comparison.

In addition, the region occupied by the HBGS and 60kV Switchyard (6.63 acres) was deducted from the overall area (101.96 acres) as those facilities have their own governing permits and storm water management systems. Any of the site covers present in 2003 which occurred within the boundary of HBGS and the 60kV Switchyard were ignored for the purposes of this analysis, as it is assumed these regions were credited during the pre and post development calculations for those facilities. This deduction reduces the total compared area to 95.33 acres; this deduction was applied for both the 2003 and 2018 cover type calculations to preserve a valid comparison.

In addition to simply comparing the overall area occupied by each cover type in each configuration the standard method of curve numbers (CN) was applied. This method is similar to the weighted average methods described in Section 4.4.3 for developing composite C values, however it features different values. The application of the CN method is useful for determining how variations affect the overall composite cover of the site; this indicates how changes in several cover types affect the overall runoff potential, not just a change in impervious area. The comparison of the CN from 2003 and the CN generated from the proposed 2018 FSR site plan will provide information for closing out the construction SWPPP currently governing HBPP. Table 13 compares these values and indicates a net reduction in CN from 2003 to 2018, from 80.1 to 80.0. A reduction in overall CN, when comparing post development to pre development, indicates that the runoff potential of the site is reduced.

The site aerial photo from 2003 is shown in page 9 of Attachment 2; this image was utilized to delineate the extent of three basic cover types on site: impervious (buildings, tanks, and roads), gravel, and grass (includes all trees, brush and maintained grass regions). These cover types were delineated and their total areas compiled into Table 13.

The proposed site configuration for the end state of HBPP is indicated in Attatchment 1; this figure was modified and simplified to produce page 10 of Attachment 2 which represents the basic surface cover types proposed for FSR at HBPP. As above, the same three surface cover types were adopted: impervious (buildings, tanks, and roads), gravel, and grass (includes all trees, brush and maintained grass regions). These cover types were delineated and their total areas compiled into Table 13.

A review and comparison of relative areas in 2003 to the proposed 2018 end state configuration after FSR indicate that the:

- Total impervious area will decrease by 0.80 acres;

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- Total graveled areas will increase by 0.68 acres; and
- Total grassed-vegetated areas will increase by 0.12 acres.

This reduction is achieved even under the worst case assumption of full build out potential. As discussed in Section 4.4.3: all proposed FSR paved regions were expanded by multipliers ranging from 110% to 125%, the laydown pad occupying the areas of the decommissioned units 1, 2, and 3 is assumed to be totally impervious, and the proposed ISFSI truck turnaround is assumed to be expanded to its full potential. These additions to the impervious area add 2.29 acres to the 2018 total shown in Table 13. Were these conservative assumptions accounting for future potential activities not included, the overall reduction in impervious area and resulting curve number would be more favorable indicating larger reduction in impervious areas and associated curve numbers.

While relevant to governing permits, these values do not affect the overall design or proposed storm water treatment features of the FSR because 100% of the runoff is being routed and treated through the storm water basins regardless of whether it is being produced by proposed or previously existing portions of the site. All proposed treatment and conveyance systems are sized to treat and convey runoff from the entire site, not just newly proposed elements.

4.4.2. Determination of the A Coefficients

The hydrology map, Attatchment 2, display the contours of the entire site as generated by an aerial topographic survey conducted in November of 2013. This base topographic map was adjusted according to proposed grades and topographical alterations to be implemented during FSR, producing a composite final site topographic map which captures existing topography of unaltered regions of the site and planned FSR grading activities. This final composite topographical surface was used to delineate the boundaries of all 71 of the watersheds within the boundary of the HBPP area of analysis. These watersheds range from tenths of an acre to several acres and represent the site specific drainage pattern within the bounds of the proposed storm water management system created by implementing the FSR grading and drainage plan, page 2 of Attachment 1 and Attachment 2.

These 71 watersheds were grouped according to which one of the 7 bio detention basins it contributes. A naming convention for identifying each of the watersheds was adopted which indicates its relative position to its basin and the flow path from the watersheds' location to the basin. All 7 major watersheds, with the applicable sub-sheds, are shown in Attachment 2. The total areas (acres) associated with each sub-shed are shown in Table 4 through Table 6; this value serves to supply the required (A) value of the CIA relationship used to calculate Qp.

4.4.3. Determination of the C Coefficients

The (C) component of the CIA relationship represents the cover type associated with each watershed and/or sub-shed. Typically a single watershed is composed of several cover types; each cover type has an associated standard coefficient representing the percentage of runoff produced by that cover type. Impervious areas like buildings, concrete and asphalt all have high C coefficient values, while pervious areas with cover

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indicative of grass or forest have low C coefficients, representing a low degree of runoff. To remain conservative, the variable site covers were reduced to three basic types all representing the more conservative published values, thereby generating the higher estimates of Qp.

All impervious areas of the site were classified under the same conservative coefficient value of C = 0.95; this consolidation includes regions of cover associated with buildings (0.75 - 0.95), concrete (0.80 - 0.95), and asphalt (0.70 – 0.95) (Lindeburg, 2014); all gravel regions were given the conservative coefficient of C = 0.80 (Lindeburg, 2014); all regions which are not captured under either impervious or gravel cover types are assumed to be grass, which is given the conservative coefficient of C = 0.30 (Lindeburg, 2014); this consolidation captures regions of forest and dense underbrush which all have lower coefficient values.

The application of weighted averages permits the various cover types within each watershed to be combined to produce a composite C coefficient, which represents the respective watershed's runoff potential. Since conservative values for independent cover types were used, the resulting composite C coefficient from their combination will generate estimates of Qp values which are likely higher than the actual flows anticipated; thereby retaining overall conservatism and ensuring storm water conveyance structures are adequately sized. The hydrology map in Attachment 2 display the cover types within each watershed, and Table 4 through Table 6 shows the relative areas of each cover types of each watershed and the resulting composite coefficient (C). The composite coefficient represents all of the combined cover types within a watershed of interest and is utilized to calculate the Qp associated with each watershed.

In addition to adopting relatively high values for representative cover types, additional conservative assumptions were made to ensure predictions of Qp values represent a worst case. The FSR Plan will undergo additional refinement upon entering the construction detailing phase of the project. To ensure that the values of Qp predicted with the current FSR plan are not invalidated by potential alterations in the future, proposed impervious areas were increased by 110%-250%, depending on the expansion potential of the specific area. The added area of the impervious regions was deducted from the area of grass cover within the same watershed; thereby increasing higher runoff coefficients while reducing lower runoff coefficients. This expansion of impervious area allows for some fluctuation in total impervious area during construction detailing without concerns of exceeding the capacity of storm water conveyance structures; the level of expansion used for each watershed is shown in Table 4 through Table 6.

In addition, the region of the FSR plan which repurposes the Unit 1, 2, and 3 footprint is shown in gravel on the FSR plan set, Attachment 1, as that is the anticipated cover to be installed; however, that region of the site may eventually be converted to a building pad or other impervious cover. To account for this potential site alteration, this entire region was assumed to be impervious cover not gravel for the purposes of calculating peak flows. A similar strategy was adopted for the truck turn around region proposed for the top of Bayview heights adjacent to the ISFSI. The FSR plan set, Attachment 1, shows a

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minimal truck turnaround; however if the ISFSI casks require transport and disposal in the future this region will need to be expanded by ~250%; therefore this region was assumed to be impervious cover to the full build out potential for the purposes of calculation.

4.4.4. Determination of the I Coefficient

The final variable required to calculate peak flows (Qp) with the CIA method is derived from the applicable set of IDF curves specific to the region of analysis. A specific IDF curve set has been previously derived specifically for the HBPP region of analysis, Figure 8, and adopted for this investigation. To permit the use of an automated spreadsheet, each IDF curve presented in Figure 8 was converted into a continuous function using the Excel spreadsheet program and standard methods of regression. The continuous functions took the form of power functions and were increased by 110% to ensure they always overestimate, or at least equal, the intensity at any given time thereby preserving conservatism. The comparison of the original unmodified IDF curve and the derived functional IDF curve are presented in Figure 9 through Figure 12 for each of the applicable return periods: 10 year, 25 year, 50 year, and 100 year.

The utilization of these functional IDF curves allows for the Intensity (I) of precipitation (in./hr) to be returned for any applicable Time of Concentration (TOC) without returning to a graph to manually re-evaluate TOCs as they vary with alternatives and proposed geometry of drainage features. TOCs are a critical input to determining (I) values and subsequent Qp values; determination of TOCs is complex and a detailed discussion is presented in Section 4.5. The adoption of functionally derived IDF curves allows for expedited development of alternative drainage systems, permitting a wide array of alternatives to be compared easily and efficiently.

As can be observed in Figure 9 through Figure 12, the agreement between the functionally derived IDF curve and the original unmodified IDF curve is relatively poor for times of concentration less than 7 minutes; this is mitigated by the following model parameters:

- As shown in Table 7 through Table 9, no watersheds have TOCs less than five minutes, as at least five minutes is required for initial abstraction of precipitation to build up a critical volume of water to produce overland flows; therefore TOCs between 0 and 5 minutes are not applicable.
- As shown in Table 7 through Table 9, approximately a third of the watersheds have TOCs between five and seven minutes, however nearly all of those watersheds have times of concentration near seven minutes. At a TOC of approximately seven minutes, all the functionally derived IDF curves feature an intercept with their respective original unmodified IDF curves, indicating that there is no error between the two functions at that point; therefore the magnitude of the error introduced to the intensity values between five and seven minutes is negligible. In addition, the error produced always results in an over estimate of the intensity, so conservatism of the model is preserved.
- Lastly, watersheds which feature TOCs less than seven minutes are relatively small; their small size contributes less to the overall QP values, so the slight overestimate error has relatively little effect on the overall model results.

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4.4.5. Determination of CIA Values for Composite Watersheds

The methods described in Section 4.4 describe how the peak flow was calculated for each individual watershed independent of all surrounding watersheds. This permits the estimation and design of flow control structures within the watershed of interest, but does not take into account the interconnectedness or specific routing sequence of watersheds which all contribute to the same point of concentration. To determine the peak flow within a drainage feature that is receiving runoff from multiple watersheds, those watersheds must be composited and new composite values of C, I, and A must be determined.

The composite watersheds are identified in Table 4 through Table 12; they are indicated by the letter "C" occurring at the end of their respective ID numbers. These watersheds are applicable to drainage control structures receiving run off from at least two upstream watersheds. For example two adjacent watersheds contribute runoff via their respective swales into a shared pipe. The peak flow within each watershed, governing the design of their respective swales, can be determined with the methods described in Section 4.4, but the peak flow observed in the shared pipe is not just the summation of the two independent flows. To determine the peak flow applicable to the shared pipe the two independent watersheds must be composited utilizing the methods outlined below.

The determination of the composite C value for the composite watershed is the same as the method for individual sheds; all the respective cover types from the individual watersheds which contribute to a shared composite drainage feature are added together and compared to the total area, the summation of these weighted averages represents a combined C value for the composite watershed. The determination of the composite A value for the composite watershed is simply the summation of the total areas of all the contributing water sheds.

As described previously, the (I) value used in the CIA calculation of peak flow is dependent on the TOC for the watershed of interest. For the purposes of this section of the report TOCs will be discussed as pre-determined values, Section 4.5 contains a detailed discussion of the determination of TOCs.

A composite watershed is comprised of multiple watersheds which could contain multiple parallel paths through numerous adjacent watersheds to the same point of concentration. The applicable TOC for the composite watershed is the TOC which permits the entire composite watershed to contribute to the same point of concentration; this TOC is equivalent to the longest possible travel time down any one parallel path through the multiple contributing watersheds. The spreadsheet was used to map out all of the possible parallel paths above a particular point of concentration within a composite watershed and compare the accumulated TOCs down each respective flow path. The longest travel time was adopted as being representative of the TOC for the entire composite watershed; this TOC was used to determine the appropriate (I) value for the CIA computation of peak flow within a composite watershed.

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The calculated Qp value was used to design the portions of the drainage system which receives runoff from multiple upstream watersheds. The relevant values of the: Areas (A); Composite Cover coefficients (C); Precipitation Intensities (I); and applicable TOC for all of the composite watersheds can be found in Table 4 through Table 12.

4.5. Determination of Times of Concentration (TOC)

The TOC governs the Intensity (I) value used in the CIA computation and is therefore an important variable in determining the peak flow (Qp) of a watershed. The TOC represents the total response time of the watershed; it is the duration of elapsed time it takes for water which began falling on the farthest reaches of a watershed to reach the point of concentration at the bottom of the watershed. After the total TOC has elapsed the entire area (A) of the watershed is contributing to flows observed at the point of concentration; because the area (A) variable of the CIA calculation is maximized, the observed runoff is considered to be the peak flow for that watershed. TOCs are a critical variable in determining the peak flow of a watershed because the elapsed duration in minutes serves as the input to the applicable IDF curve; ultimately determining the value of the Intensity used for analysis of a given watershed. Watersheds with shorter TOCs will have higher corresponding values of Intensity; while watersheds with longer times of concentration will have lower corresponding values of Intensity, Table 7 through Table 9, shows the total TOC values for all the watersheds.

Determining the TOC for a given watershed involves many variables which govern the velocity of the water on its way from the farthest reaches of the watershed to the point of concentration. The types of flows exhibited by runoff traversing a watershed can be composed of one, two, or all three of the following classifications: Overland Sheet Flow, Shallow Concentrated Flow, and/or Open Channel Flow; these types of flows and the computational methods employed to estimate the travel time associated with each one is discussed below.

4.5.1. Determination of Overland Sheet Flow Travel Times

The velocity and resulting total travel times for sheet flow were calculated in accordance with the methods outlined in the USDA NRCS TR-55, Urban Hydrology for Small Watersheds (USDA 1986). Sheet flow develops at the farthest reaches of the watershed as water collects and begins to run down gradient. Per TR-55, sheet flow often takes approximately 5 minutes to accumulate and is used for shallow flows (<0.1 foot) for up to 300 feet of traversed distance. The travel time for sheet flow is calculated with Manning’s kinematic solution in accordance with Equation 1 below. The resulting travel time is a function of the:

- Roughness coefficient of traversed surfaces;
- Total watercourse length;
- 2 year, 24-hour rainfall total (3.12 inches (NOAA, 2014)); and
- Watercourse slope.

Equation 1: Manning’s Kinematic Solution

$$T_t = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5} s^{0.4}}$$

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Where,

T_t	=	Travel time (*hr*).
n	=	Manning's roughness coefficient (-).
		n =0.011 for Impervious surfaces (concrete, asphalt)
		n =0.040 for gravel surfaces
		n =0.240 for grass surfaces
L	=	Flow length (*ft*).
P_2	=	2-year, 24-hour rainfall (*in*).
s	=	Slope of hydraulic grade line (*ft/ft*).

The relevant values of these variables and the resulting sheet flow travel times for each watershed are shown in Table 7 through Table 9.

4.5.2. Determination of Shallow Concentrated Flow Travel Times

After runoff has been traveling for more than 300 feet as sheet flow it typically turns into shallow concentrated flow (USDA, 1986). The average velocity for this type of flow is determined in accordance with Figure 3-1 of TR-55; which represents the average velocity of runoff for varying slopes over paved and unpaved surfaces, Figure 13. After the applicable velocity value is determined the travel time for shallow concentrated flow is simply determined by dividing the watercourse distance by the velocity value and correcting for units.

To permit the use of an automated spreadsheet, the manual process of using the TR-55 chart for each instance of shallow concentrated flow was eliminated by using the supplied TR-55 chart, Figure 13, to generate functional relationships which solve for the run off velocity on paved and unpaved surfaces at any given slope, Figure 14 shows this transformation and resulting equations. The functional relationships were best represented by power functions which achieved high R^2 values of more than 0.999; indicating very little variation between values returned by completing the manual method outlined in TR-55 and the automated methods employed by the spreadsheet. The relevant values of the: flow lengths; watercourse slopes; cover types; and resulting shallow concentrated flow travel times for all of the watersheds can be found in Table 7 through Table 9.

4.5.3. Determination of Open Channel Flow Travel Times

After runoff has made its way down the watershed, via sheet flow and shallow concentrated flow, it is often collected in swales, gutters, pipes or similar flow conveyance systems. The velocity, and subsequent travel time, of runoff within these flow control structures are governed by Manning's Equation for Open Channel Flow, Equation 2.

The velocity of flows within open channels is a function of the: hydraulic radius; the channel slope; and roughness coefficient. The hydraulic radius is a function of the cross sectional flow area and the wetted perimeter; both of these variables depend on specific channel geometry and the associated peak flow within the structure. A circular reference is introduced into the calculation methodology because variables which

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depend on the peak flow (cross sectional flow area and wetted perimeter) are also inputs to determine the travel time, time of concentration and resulting peak flows; this indicates that an iterative computational method is required to achieve convergence on a singular solution. To overcome this computational detail, while preserving overall efficiency and versatility of the automated spreadsheet, the following method was employed.

The third party computational package called Flow Master was utilized to obtain rating curves relating cross sectional flow area and wetted perimeter to a wide range of values for both channel slope and discharge for numerous proposed swale and pipe geometries. These rating curves discretize the continuous functions for both variables, as they relate to flow and slope, into singular values for every combination of channel slope and discharge within the limits of the rating curve.

This discretization is as follows:

- Discharge values between 0.01cfs and 10cfs were plotted on the curve in 0.1cfs increments; and
- Channel slopes values between 0.0025 and 0.5 were plotted on the curve in increments of 0.0025.

Therefore values of both the variables (cross sectional flow area and wetted perimeter) are available for any combination of discharge and slope within the limits described above for numerous proposed swale and pipe geometries. These rating curves are utilized to calibrate the cross sectional flow area and wetted perimeter variables when calculating the Manning's equation for flow velocity.

The values of the two variables used in the Manning's equation are set equal to the values observed in the rating curve data for a respective flow quantity; this ensures that for any calculated quantity of discharge the cross sectional flow area and wetted perimeter values equal the Flow Master rating curve values for that same quantity of discharge; thereby calibrating the Manning's equation in the spreadsheet and permitting the calculation of an accurate velocity value for use in the travel time calculation for open channel flow.

Furthermore, due to the non-continuous discretization of available Flow Master values for the variables of flow area and wetted perimeter in each rating curve, the values adopted for use in the spreadsheet are subject to slight error, as the Qp value calculated with the spreadsheet may not exactly equal the value of Qp returned by Flow Master. The introduction of this source of error was mitigated by keeping the discretization increment of each rating curve small (0.1 cfs), and always rounding up the spreadsheet value of Qp when referencing the corresponding values of the desired variables in the Flow Master rating curves. This accommodation results in the values of the flow area and wetted perimeter used in the spreadsheet calculation to correspond to a slightly higher flow value in Flow Master, thereby maintaining conservatism in the model.

Flow master was used to create rating curves for over 20 variations of swale and pipe geometry describing both existing conditions and proposed drainage structures. This

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permits the calculation of open channel flow travel times for a wide array of possible swale and pipe alternatives for any combination of slope and predicted discharge within the limits described above. After the applicable velocity value is determined, the travel time for open channel flow is simply determined by dividing the watercourse distance by the velocity value and correcting for units. The automated spreadsheet allows for the incorporation of two open channel flow segments within each watershed, typically this is a surface swale into a subterranean storm drain pipe, but could be any combination of pipe and swale. The relevant values of the: flow lengths; watercourse slopes; channel types; and open channel flow travel times for all of the watersheds can be found in Table 7 through Table 9.

Equation 2: Manning's Equation for Open Channel Flow

$$v = \frac{1.49 r^{2/3} s^{1/2}}{n}$$

Where,

v = Average velocity of fluid flow (*ft/s*).

r = Hydraulic radius, equal to a/p_w

- a = Cross-sectional flow area (ft^2).
- p_w = Wetted perimeter (*ft*).

s = Slope of hydraulic grade line (*ft/ft*).

n = Manning's roughness coefficient (-).

- n =0.011 for Impervious surfaces (concrete, asphalt)
- n =0.040 for gravel surfaces
- n =0.240 for grass surfaces
- n =0.010 for HDPE surfaces

4.6. Storm Water Routing

As discussed earlier in this report, the routing of runoff through the various paths and conveyance systems within each watershed or composite watershed becomes an important consideration in determining relevant values of the resulting TOC. Storm water runoff routing took two forms in the analysis: upstream routing used to determine peak flows of composite features, and downstream routing to determine applicable lag times for basin modeling; the methods used to determine each are described below.

4.6.1. Upstream Routing of Storm Water Runoff

Upstream routing of storm water runoff is used to determine the longest possible travel time from the upper reaches of a contributing watershed down through intermediate watersheds to a given point of concentration of a composite drainage feature receiving flows from multiple watersheds simultaneously. There could be multiple flow paths above a given point of concentration; each flow path would traverse a unique sequence of watersheds before reaching the same given point. As discussed in Sections 4.4.5 and 4.5, identifying the travel time associated with each of these parallel paths is an important detail to properly quantifying the peak flow within the receiving drainage structure.

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As indicated in Section 4.4.2, the longest possible travel time will correspond to the largest peak flow value as it indicates the point when the entire upstream watershed is contributing flows to the same location. The spreadsheet was used to map out each of the parallel flow paths above a singular point of concentration and compare the total travel time of each unique route. The longest route, in terms of elapsed time, was adopted for the CIA computation for that given point of concentration and associated drainage structure. This method was used to size all the composite features of the proposed HBPP FSR plan.

4.6.2. Downstream Routing of Storm Water Runoff

Downstream routing of storm water runoff is used to determine the total travel time from the upper reaches of a given watershed of interest, through all of the downstream drainage features, into the receiving basin. The spreadsheet was used to map out the route that water flowing from a given watershed of interest would take on its way to its associated basin. All the unique travel times it takes for runoff to traverse the intermediate watersheds were added together to determine a total lag time. This travel time accounts for shortened travel times through composite drainage features due to the combined flow and increased velocity created by consolidating run off; therefore the predicted travel time represents the travel time associated with the combined peak runoff down each of the water courses leading to a basin. This lag time represents the time it takes for runoff associated with given watershed to enter a receiving basin. These values are shown in Table 7 through Table 9 and are used to execute the detailed basin analysis described in Sections 4.9 and 4.10.

4.7. Design of Flow Conveyance Structures Using the Peak Flow Method

With all of the variables required to determine the critical peak flow values associated with all the watersheds and composite watersheds identified, as described in the preceding sections of this report, the CIA calculation can be completed. The spreadsheet was used to calculate and track the peak flow (Qp) values associated with 4 IDF curves: 10-, 25-, 50- and 100-year.

The automated spreadsheet allowed for the efficient comparison and evaluation of an array of available swale and pipe alternatives for various portions of the storm water conveyance system. Variations in swale and pipe geometry and their corresponding performance could be modeled with each of the IDF curves to determine which structures worked best for various locations. All of the proposed drainage structures were designed to convey flows generated by the 10-year IDF curve, so all proposed structures will convey at least peak flows associated with a 1 in 10 year event.

The remaining IDF curves (25-, 50-, and 100-year) were used to model the proposed structures' performance in higher intensity, less frequent storms. The depth of runoff predicted in each structure was used to indicate how close to capacity that given storm water conveyance structure was; Table 10 through Table 12 shows the percent capacity of each drainage structure in each watershed. Values in Table 10 through Table 12 in excess of 100% indicate that the feature is inadequate for the predicted peak flow value and temporary flooding will result in the region of that component. It can be observed in Table 10 through Table 12 that flooding is predicted for several drainage structures during 25-, 50-, and 100-year events,

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however; the maximum observed flooding is less than 20% greater than the full flow capacity of any single feature. This maximum predicted flooding is relatively small and is not expected to persist in any one location for long after the precipitation intensity lessens. For general design purposes it is not feasible to size drainage structures to convey storms as large as the 1 in 100 year event.

Table 10 through Table 12 indicates that all proposed drainage structures are less than 100% full during a 1 in 10 year event as predicted with flows derived by the CIA method. There are existing drainage features with are predicted to be inadequate in even the 1 in 10 year event; the slight flooding associated with these existing structures in the context of their location do not warrant replacement; therefore there are no proposed alterations to these existing structures.

Additionally, the intensity values shown on the adopted HBPP specific IDF curves are often up to 300% greater than any of the hourly intensity values observed in the 64 years of historical precipitation data. This confirms that the CIA method is inherently conservative; furthermore, as noted in applicable sections above, several conservative assumptions were made when calculating the peak flows with the CIA method. Accounting for these levels of conservatism throughout indicates that peak flows which result in a drainage feature being between 100% and 120% of capacity may be a product of conservatism; therefore predictions of flooding may not be as high as values indicate.

As noted in previous sections of this report, the grading of the site is constrained by retention of several buildings, matching existing grades along the border of HBGS and HBPP, relatively high inlet elevations to the bio-detention basins, and the predominantly flat profile of the lower regions of the site. These constraints excluded the use of conventional DIs and subterranean piping systems. The flat profile of the site requires the surface flow features responsible for conveying runoff to exhibit relatively flat slopes (0.25% to 0.5%).

4.8. Bio Detention Basin Sizing

As shown in Attatchment 1 and Attachment 2, the HBPP site contains seven bio-detention basins. There are three large basins which receive ~90% of storm water runoff produced on site. The four remaining smaller basins receive storm water runoff generated by portions of the Alpha and Bravo access roads only. All storm water generated on the HBPP site, as indicated in Attachment 2, passes through one of these basins prior to entering adjacent habitats. All bio-detention basins provide the minimum required volume to receive and detain 100% of the 85th percentile, 24-hour storm event. This satisfies the capture and treatment criteria in the City of Santa Rosa LID Design Manual provided to WEI by NCRWQCB for the planning of FSR at HBPP.

4.8.1. Bio-detention Basin Sizing

The basin sizing method outlined in the Santa Rosa LID Manual was utilized to size these basins. This method utilizes the TR-55 Manual, “Urban Hydrology for Small Watersheds” (USDA, 1986) Curve Number (CN) method. The equations used in this method are presented below.

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Equation 3: Curve Method Determination of Maximum Retention

$$S = \frac{1000}{CN_{POST}} - 10$$

Where,

S = Potential maximum retention after runoff (*in*).

CN = Curve number for the developed condition associated with the tributary area.

CN = 98 for impervious regions

CN = 91 for gravel regions

CN = 78 for grass regions

A weighted average, based on each cover type area, is used to determine the composite potential maximum retention after runoff for all three cover types,

Equation 4: Generation of Composite S Value

$$S_{COMPOSITE} = \frac{S_1 A_1 + S_2 A_2 + S_3 A_3}{A_1 + A_2 + A_3}$$

Where,

S_i = Potential maximum retention after runoff for area i (*in*).

A_i = Tributary area associated with cover type i (ft^2).

Equation 5: Run off Depth

$$Q = \frac{((P \cdot K) - (0.2 \cdot S))^2}{((P \cdot K) + (0.8 \cdot S))} \cdot \frac{1ft}{12in}$$

Where,

Q = Runoff depth (*ft*).

P = Precipitation for the 85th percentile, 24-hour storm event (*in*).

K = Seasonal precipitation factor.

S = Potential maximum retention after runoff (*in*).

Note: P =0.68in (Section 4.1), K =1 for conditions in Humboldt County, CA.

Equation 6: Minimum Required Capture Volume

$$V = Q \cdot A$$

Where,

V = Volume of storm water to be retained (ft^3).

Q = Runoff depth (*ft*).

A = Tributary area (ft^2).

Several levels of conservatism were adopted during this analysis to ensure the basins are adequately sized. The TR-55 cover types present on the HBPP site along with their associated curve numbers include the following: Paved (98), Gravel (91), Woods (77), Meadow (78), and Brush (73). To remain conservative, the three worst case

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combinations of cover types and curve numbers were adopted for analysis: Paved (98), Gravel (91), and Meadow (78); even though substantial portions of the site will include the woods and brush cover type, their exclusion increases the quantity of anticipated runoff and produce a larger basin size. In addition, available soil reports indicate the site is predominantly of soil type C; the curve numbers utilized in the calculations for minimum required capture volumes are associated with the slightly more conservative soil type D; thereby producing larger basins.

The paved and gravel areas were measured using Auto CAD and the meadow area was taken as the difference of the summation of the paved and gravel areas and the total applicable area under analysis. As detailed in Section 4.4.3, to account for the conceptual nature of the FSR plans, all proposed impervious areas have been increased by 110%-250%, and regions of the plant which may feature expanded impervious areas in the future were assumed to be impervious to their maximum extents to ensure the bio-detention basins are adequately sized for long term application. Another conservative measure was to adopt the slightly larger 85^{th} percentile, 24-hour storm event value for accumulated precipitation of 0.68 inches derived from the process described in Section 4.1 as opposed to the published value of 0.65 inches.

Lastly, all calculated minimum storage volumes were increased by 150%. As discussed in Sections 3.2.4 and 4.8 of this report, these basins will be inducted into the IGP for HBGS. The basins have been designed to comply with applicable portions of both the IGP and LID design criteria. The IGP required the inclusion of a safety factor in calculating detention basins sizes; therefore the storage volumes calculated in accordance with the LID manual have been increased by 150%.

Although Basins A and C are calculated and sized as individual basins, they are effectively connected via an equalization pipe, which allows unrestricted transmission of water from one basin to the other; this creates a single composite basin which receives water from both water sheds. The final entry in Table 14 indicates the combined volumes and areas of discrete cover types applicable to this combined basin. These basins could not be fully connected as there are existing utility lines which need to be preserved in their current configuration.

As discussed in Section 5 of this report, the basins will be monitored for sediment accumulation on an annual basis. The inclusion of the 150% factor of safety in overall design volume provides additional volume for sediment accumulation without violating the minimum required volume for treatment and capture. Up to 20% of the total design volume will be allowed to be occupied with accumulated sediment at which time the basin will be excavated and returned to the original design volume. This ensures the basins will maintain a minimum of 120% of the minimum required storage volume.

Attachment 2 and Table 14, respectively, shows the: regions of HBPP contributing to each of the seven basins; the areas occupied by the relative cover type; the calculated minimum volume required to achieve 100% capture and treatment; and design volume which includes the 1.5 factor of safety.

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4.9. Basin Modeling and Application of Historical Data

As described in Section 4, to best quantify the effective treatment of storm water within the proposed bio-detention basins, a robust hydrology model was developed which utilizes 62 years of local historical hourly precipitation data to assess how the proposed basins would have performed, if subjected to historical storms of interest. The processing and consolidation of this data set is described in Sections 4.1 through 4.3; as described therein, hourly data was consolidated into daily and monthly precipitation values and ranked by accumulated precipitation totals to identify specific historical days and months that represent thresholds of exceedance probability. As adopted by IDF convention these design thresholds of exceedance probability are:

- 1 in 10 Year Event (90th Percentile Event (0.900));
- 1 in 25 Year Event (96th Percentile Event (0.960));
- 1 in 50 Year Event (98th Percentile Event (0.980)); and
- 1 in 100 Year Event (99th Percentile Event (0.990))

The application of these thresholds to the monthly historic data set identified the four months shown in Table 3. These months have total precipitation values which place them at their respective percentile values when compared to the total available data set. The hourly data of each respective month of interest was retrieved from the historic hourly data set and is shown in Figure 15 through Figure 18. These precipitation events represent continuous historical hourly data of actual precipitation events for the duration of a month (720 data points per month).

This data allows for the investigation of basin performance over the course of 30 days when subjected to:

- Variable storm profiles over a wide range of hourly intensities;
- Multiple storms with varying degrees of separation; and
- Storms exceeding 24 hours in duration.

The performance of the required minimum basin sizes, as determined by the prescribed methods above, can be modeled as if historical precipitation events of interest were experienced by the proposed FSR site plan, producing varying quantities of predicted inflows. This allows for the determination of the detained volume within each basin as it varies over time. The incorporation of infiltration rates and some basic basin geometry, allows for the determination of outflows of treated (infiltrated water) and untreated (bypassed water). Comparing the total inflow to the treated outflow, the basin performance can be expressed as a simple percentage.

4.10. Determination of Variable Historical Flows into the Basins

To create a model which accurately describes the variable inflow to a basin that results from variable precipitation intensities over time; the respective times of concentration are used to lag the individual response and runoff contribution of all the watersheds contributing to the basin of interest, Section 4.6.2. To achieve this each independent water shed above a basin was routed through the flow path it would travel to reach the basin. All the respective TOCs of the traversed watersheds were tracked and summed; this total travel time for a specific watershed represents the lag time from the farthest reaches of the watershed down to the basin. The lag times for each watershed to reach its respective basin are indicated in Table 7 through Table 9.

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To find the inflow to a basin at any given minute during the analysis period the historical hourly precipitation values (in/hr) were transformed into continuous minute precipitation values (in/min). The lag times determined as described above were rounded to the nearest whole minute. Converting the units of the CIA equation to permit the use of (in/min) and the calculation of peak flows in (cfm) allows the previously determined CA values to be utilized with the historical (in/min) data to produce peak flow estimates for every elapsed minute. This method produces peak flow estimates (cfm) for every minute of an entire month of historical precipitation data (in/min) (43,200 data points).

The determined lag times are used to delay the inflow contribution of their respective watersheds by the applicable number of minutes, as indicated by the total lag time. Once the required number of minutes has elapsed, the entire uninterrupted data set of precipitation intensities (in/min) is applied to the respective watershed and the resulting runoff contribution (cfm) is added to the basin in a continuous flow. The entire storm is delayed from entering the basin by the respective lag time of each watershed. This variation means the inflow contributed to the basins by each watershed is occurring at a different time when compared to adjacent watersheds. For each elapsed minute at the basin: some sheds will be contributing inflows associated with intensities (in/min) which occur earlier in the storm; some sheds will be contributing inflows associated with intensities (in/min) which occur later during a storm; and some watersheds may not even be contributing to the basin yet, as their respective lag time has not elapsed. The inflow experienced at the basin is simply the summation of all the lagged contributions from all the applicable watersheds for each discrete minute that elapses. The graphed continuous inflow (cfm) to each basin that results from a historical precipitation data set being applied to the proposed FSR site grading and drainage plan, for all the minutes elapsed during a month, represents the inflow hydrograph for that bio-detention basin.

To determine the accuracy of this model, the singular Intensity (in/hr) value used in the conventional CIA determination of peak flow (Qp) entering a respective basin, Section 4.4, was substituted into the first hour of the historical storm being analyzed with the described lagging methods, Section 4.6.2. A comparison of the peak inflow value for a specific basin predicted by the conventional CIA methods, and the peak inflow value for a specific basin predicted by the lagging method, employed by the model, would provide an indication of model validity. Table 16 shows the peak inflow values predicted by the conventional CIA method and the lagging method for all basins and all percentiles of interest, along with their respective percent error.

As indicated by Table 16, the model performs well, exhibiting close estimations to the conventional CIA method. No percent error is observed on smaller watersheds and only a low percent error (~5%) is observed on larger watersheds; this indicates the hydrographs produced by employing the lagging method of the model would strongly agree with conventional CIA methods if those methods were applied 43,200 times to produce an equivalent data set. The small percent error introduced on the larger watersheds is likely the result of cumulative rounding of TOC inputs. It should be noted that the intensities (in/hr) produced by the utilization of the IDF curves are several times larger than any of the intensities (in/hr) observed in the historical data set, so the peak flow produced by each method, without substitution of the Intensity values, reflect that disparity.

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4.11. Determination of Percent Effectiveness of each Bio-Detention Basin

With the inflow hydrographs to each of the basins determined, for monthly historical precipitation events of interests, the outflow hydrographs can be calculated. To transform the basin inflow hydrograph to an outflow hydrograph several basin variables need to be identified or assumed as described below.

4.11.1. Basins Size and Bypass Flows

To determine the outflow hydrograph the total basin volume needs to be known, these minimum volumes have been identified utilizing the prescribed methods of the LID manual. As described in Section 4.8, these volumes were increased by 150% to provide a safety factor. This minimum volume, plus the 50% overage, represents the maximum possible volume of water which could be detained in each basin, Table 14; inflows which would require additional storage are bypassed and assumed to not pass through the filtration media and represent untreated storm water.

The high flow bypass predicted by the application of the CIA method informs the construction detailing effort of the maximum flows entering a basin at any given time and as such also determine the minimum capacity of high flow bypass features. Assuming the worst case combination of a full basin and 100-year event the recommended bypass capacities are indicated in Table 15.

4.11.2. Infiltration Rates through Bio-Detention Basin Filter Media

The infiltration rates through the bottom of the basins are also an important variable to determination of the outflow hydrograph and overall performance. Based on the total basin volume described above, the approximate area (sqft) at the bottom of each basin which would facilitate infiltration is shown in Table 14. This area is directly proportional to the total infiltration of water which occurs each minute in each basin.

The infiltration rate also depends on numerous independent variables including: soil/material type, antecedent saturation levels, performance over time, increased infiltration due to head pressure built up in a column of stored water, total area where infiltration can occur, etc.; quantification of all these variables are beyond the scope of this report, so the following assumptions have been made:

- Saturated antecedent conditions exists within the basins;
- Each basin exhibits steady state flow through the filtration media, and is unaffected by changes in head pressure; and
- Degradation of infiltration performance is negligible over the 1 month period of analysis.

Adoption of these assumptions allows for a single infiltration rate (in/hr) to be combined with the infiltration area (sqft), Table 14, to produce an estimate of total infiltrated volume each minute. This infiltrated volume is assumed to represent treated storm water; a comparison of treated (infiltrated) and untreated (bypassed) storm water provide a simple estimate of percent effectiveness of each basin. Subtleties of contaminates existing in runoff earlier in storm events, or the dilution of contaminates due to sustained precipitation is not part of this simple metric; the percent effectiveness

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discussed is only a prediction of water which will infiltrate through the filtration media vs. water that will be bypassed.

This report is intended to inform the construction detailing effort during final planning, so the basins were modeled under a range of possible infiltration rates; this allows for a desired infiltration rate to be identified which produces a certain percent effectiveness. The material/soil media used for infiltration can be developed to produce the desired infiltration rates. All basins were subjected to all the precipitation events of interest with varying rates of infiltration from 0.5 in/hr to 4.0 in/hr; their respective percent effectiveness was compared, Table 17.

In addition to the actual infiltration rate of the filter/soil media, the percent effectiveness of each basin could be increased by increasing the area where infiltration occurs. The area quantities representing the infiltration footprint used for each basin, Table 14, is based on a 3:1 (H:V) side slope in the large basins and a 2:1 (H:V) side slope in the smaller basins; if during construction detailing a higher infiltration rate is desired the side slopes could be steepened to expand the infiltration footprint and accelerate infiltration.

4.11.3. Groundwater and Infiltration Rates through Sub Soils below Bio-Detention Basins

In accordance with the supplied LID manual, WEI executed infiltration testing of the existing on site soils in the regions of the proposed bio-detention basins. Infiltration testing utilized a dual ring infiltrometer and was executed per ASTM D-3385 as recommended. The exact location and depth of the bottom of the proposed bio-detention basins were not available on site due to the active and ever changing decommissioning landscape. The infiltrometer tests were executed as close as possible to the proposed basin locations and at an elevation representative of the anticipated strata which would be encountered upon excavation of the basin, Figure 19.

Groundwater elevations at HBPP fluctuate with seasonal variations in precipitation and tides, but are typically encountered at an elevation of 6 feet; nominal surface grades across the site, before and after FSR, vary between 11 and 12 feet. The excavated extents of the bottom of the proposed bio-detention basins (below filtration media layers) vary between 7 and 8 feet; soils at this elevation consist of dense silty clays. These soils exhibit poor infiltration rates due to their composition and proximity to groundwater. The results of the infiltrometer tests are shown in Table 18 rates vary from less than 0.25 in/hr and 1.0 in/hr depending on the location of the test.

The test which resulted in infiltration rates of 1 in/hr is likely inaccurate as it was conducted at the least represenative location and likely does not accurately represent infiltration rates which will be observed at the actual basin excavation. The other tests which conclude infiltration rates of up to 0.5 in/hr are likely a better representation of anticipated conditions as their location and soil strata available for testing are more indicative of anticipated soil types.

In terms of design implications the predicted infiltration rates were neglected in calculations and predictions of basin performance. Due to the seasonal variation in groundwater and unknowns revolving around the hydraulic connectivity and equilibrium that will be established with a functioning saturated bio-detention basin, the resulting infiltration rate could vary from poor (~0.5 in/hr) to nonexistent. The additional minor quantity of infiltration which may occur at times during the rainy season will not be detrimental to the overall functioning of the basin or have ecological implications for the adjacent wetlands. In addition, exclusion of these infiltration rates from calculations and analysis preserve conservatism when determining overall bio-detention basin performance.

4.11.4. Bio-detention Basin Performance and Percent Effectiveness

Table 17 shows how the percent effectiveness of each basin fluctuates with varying infiltration rates. The infiltration rates used to produce Table 17 vary from 0.5 in/hr, associated with saturated loam soils (Hillel, 1982), 0.75in/hr, associated with saturated sands (Hillel, 1982), and higher infiltration rates associated with dry conditions in varying soil mixtures. A review of Table 17 clearly shows high percent effectiveness levels are achieved with higher infiltration rates.

The bio-detention basins' paramount purpose is to capture and treat the "first flush" of runoff associated with precipitation events, as it is this portion of the runoff which carries the majority of possible pollutants. Infiltration rates during the first flush portion of storm events are likely to occur prior to complete saturation of the filter media within the bottom of each basin; this indicates that infiltration rates will be faster during this portion of storm runoff. As observed in Table 17, increasing overall infiltration rates to values greater than 1 in/hr increases the percent effectiveness. After a period of runoff has saturated the basin filtration media, and presumably the first flush portion of the storm is complete, infiltration rates through the bottom of the basin will likely be closer to values of less than 1 in/hr. The range of overall performance for each basin when subjected to each historical storm period of interest is indicated by compositing the values in Table 17.

Percent effectiveness was assumed to be indicated by the ratio of water infiltrated through the bottom of the basin compared to the water bypassed through the basin during high flows when the infiltration capacity is exceeded. The validity of this assumption is linked to the ability of the soil/sand infiltration media within each basin to adequately treat the water through a combination of bio-remediation and physical filtration. It should be noted that higher rates of infiltration provide higher degrees of physical filtration due to an increased flow rate through the media; however this rapid infiltration will diminish contact time of storm water passing through the media, likely diminishing the effectiveness of potential bio-remediation. As stated previously, the intention of this report is to inform construction detailing processes conducted in the future. During this future specification process, constituents of concern should be identified and linked to a preferred treatment process (physical vs. bio-remediation). A balance of physical and biological treatment methods representative of the anticipated

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potential pollutants should be determined and guide the identification of desired infiltration rates and subsequent bypass flows.

During sensitivity analysis of basin performance, it was observed that in addition to the specific critical design variables indicated above, the quantity of bypassed water and the resulting percent effectiveness, is heavily influenced by the peak inflows to the basin. Storm events which have relatively high precipitation intensities and produce relatively large peak inflows have a higher probability of being bypassed. As discussed in Section 4.2, the method of ranking the monthly precipitation data by total accumulated precipitation values does not provide any indication of variable storm intensity within the given month; therefore the 90th percentile month produces slightly lower overall percent effectiveness than the 96th percentile month, because the 90th percentile month features storms with higher short term intensities. This detail is worth noting so that the reader is aware that the percent effectiveness values in Table 17 are likely best represented by the average values at the base of the tables rather than any one particular percentile month.

4.12. Comparison of Existing and Proposed Hydrographs

The compilation of the infiltrated (treated) and the bypassed (un-treated) storm water over time constitutes the outflow hydrograph for each basin. The proposed inflow and outflow hydrographs produced by the above methods and representing the runoff profiles generated by FSR site conditions are presented in Figure 20 through Figure 25 for each bio-detention basin. These figures represent the inflow hydrograph of each basin compared to the resulting outflow hydrograph for both treated and bypassed flows. Figure 20 through Figure 25 assume a steady state infiltration rate associated with saturated sand of 0.75 in/hr and shows the respective hydrographs resulting from application of the 90th percentile month of precipitation; this combination of saturated conditions and percentile design storm is not an extreme worst case, but certainly is a coincident of unfavorable conditions. Utilization of the spreadsheet can produce hydrographs for all combinations of basin, percentile design storms, and infiltration rates, but for the sake of brevity the full set of 144 hydrographs were omitted from this report and are available upon request.

Runoff hydrographs for the existing site conditions, prior to FSR, are not available; however the report assessing the condition of the existing drainage infrastructure composed by Whitchurch Engineering Inc. (WEI 2014) does indicate peak flow values from various outfalls across the site. The proposed FSR drainage plan alters surface flow patterns throughout the site and consolidates outfalls into the bio-detention basins, so the peak flow values indicated in the existing report cannot be directly compared to the proposed FSR outfalls as they are physically different. This fundamental difference precludes a quantitative comparison of existing and proposed hydrographs; however, several features of the proposed FSR drainage plan enable a qualitative comparison.

The existing storm conveyance systems employed onsite do not feature anything beyond incidental detention. Existing storm water runoff is consolidated into conventional conveyance systems and allowed to leave the site through outfalls which feature no metering or attributes allowing control of runoff rates into adjacent wetlands and waterbodies. This feature of the existing storm water conveyance system indicates that the outflow hydrographs will closely mimic the profiles and intensities of the precipitation events producing the runoff; with lagging only introduced by watersheds' respective times of concentration from the upper reaches to the outfalls.

By comparison, the proposed FSR storm water conveyance system routes all storm water runoff through bio-detention basins. These basins provide lagging because water is required to infiltrate through the infiltration media at the bottom of the basin prior to being discharged to adjacent wetlands. As described above, this infiltration rate is a design variable to be identified during construction detailing and is subject to several independent variables; however adoption of an assumed infiltration rate can indicate anticipated lag times. Under the current design, all FSR bio-detention basins are anticipated to have approximately 12" to 24" of soil/sand/rock infiltration media in their base. If it is assumed that in a saturated state, this media would have infiltration rates of approximately 0.75 in/hr, as would be the case with sand, it would take approximately 16 to 24 hours to infiltrate through the filter media.

This lagging effect provided by the basins indicates that runoff entering a basin would not exit the basin until it has been treated via infiltration many hours later; this lagging is significantly more metering than is provided by the current storm water conveyance system and therefore by inspection, would significantly dampen outflow hydrographs of the proposed FSR storm water management plan when compared to the existing infrastructure.

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5. Maintenance and BMPs

5.1. Description of Maintenance Procedures and Funding

Currently storm water conveyance and treatment systems at the PG&E site as a whole are governed by three permits for three separate entities; HBPP complies with a Construction SWPPP, HBGS complies with an Operating SWPPP and an IGP, and the 60kV switchyard complies with a Spill Prevention, Control, and Countermeasure (SPCC) Plan.

The HBPP Construction SWPPP, which encompasses the entire HBPP portion of the PG&E site, governs storm water management through the completion of the decommissioning and final site restoration scopes of work. This SWPPP is maintained and funded through the decommissioning budget, which will provide for SWPPP implementation until the anticipated completion of final site restoration at the end of 2018.

When site restoration is complete, construction activities will be complete, and the governing Construction SWPPP at HBPP will need to be terminated. HBGS, the operating PG&E power plant, will take control and ownership of the storm water drainage and treatment systems installed at HBPP during restoration; the existing IGP and SWPPP governing HBGS will be revised and expanded to include the proposed storm water conveyance and treatment systems serving HBPP. Funding for the maintenance, inspections, and implementation of governing procedures for the storm water management features at HBPP will be assumed by HBGS. The 60kV switchyard will be unaffected by the completion of restoration at HBPP and the governing SPCC will be unaltered.

Since HBGS will ultimately take ownership of the storm water management systems at HBPP, the methods, inspections, maintenance procedures, and record keeping practices currently used to preserve compliance with the governing SWPPP and IGP at HBGS will be applied to the storm water treatment and conveyance systems inherited upon the completion of HBPP site restoration. Sections 4, 5, 6, and 7 of the Operational Drainage, Erosion, and Sedimentation Control Plan (DESCP) / Industrial SWPPP at HBGS identify specific monitoring, sampling, and reporting requirements.

The following is a consolidation of the ***additional*** relevant requirements which would be applied to the ***inherited*** storm water systems, excluding portions of the procedures governing features currently being monitored by HBGS and/or generic site wide BMPs. Further specifications of activities currently implemented at HBGS can be found in the governing SWPPP/IGP documents for HBGS available upon request.

5.2. Monthly Observations for Proposed Bio-Detention Basins (*not during* storm events)

Visual observation of the condition of the proposed bio detention basins must be made once per month.

Observations must be made:

- By the Senior Environmental Field Specialist or designee.
- During daylight hours and during scheduled facility operating hours.
- When the basins are completely drained of water or water levels within the basin are low enough to permit assessment of the condition of the basin bottom and sediment

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level. The basins should be drained, if required, for as short a period as possible to preserve the ecology within.

Points of observation for each basin during the monthly inspections are:

- Significant debris, floatables or trash which may interfere with the function of the basins outfalls or bypasses.
- Evidence of erosion at the basin inlets and outfalls.
- Inspect for the presence or indications of prior, current, or potential unauthorized non-storm water discharges and their sources.
- Inspect for authorized non-storm water discharges, sources, and associated BMPs.
- Inspect outdoor industrial equipment and storage areas, outdoor industrial activities areas, BMPs, and all other potential sources of industrial pollutants.

5.3. Monthly Storm Water Discharge Observations for Proposed Bio-Detention Basins (*during* storm events)

Visual observation of a Storm Water discharge from the proposed bio detention basins must be made once per month during the period from October 1st to May 30th.

Observations must be made:

- By the Senior Environmental Field Specialist or designee.
- During the first four hours of discharge from a Qualifying Storm Event (QSE), or at the start of scheduled facility hours if the QSE occurred in the previous 12 hours, at all discharge locations, when feasible and safe to do so, observations are to be conducted at the time that the discharge is sampled.
- During daylight hours and during scheduled facility operating hours.
- When the storm event is preceded by 48 hours without storm water discharges from any drainage area.

Storm water discharges to be observed during the monthly inspections are:

- Visually observe and record the presence or absence of floating and suspended materials, oil and grease, discolorations, turbidity, odors, trash/debris, and source(s) of any discharged pollutants.
- Storm water discharged from the proposed bio detention basins into the adjacent wetlands and intake canal; and
- Storm water that bypassed the storm water treatment system due to high flows and discharges offsite.

- In the event that a discharge location is not visually observed during the sampling event, the Discharger shall record which discharge locations were not observed during sampling or that there was no discharge from the discharge location.

5.4. Sampling and Analysis

Storm water samples will be collected within four hours of discharge from a QSE, or at the start of scheduled facility hours if the QSE occurred in the previous 12 hours. Storm water samples shall be collected and analyzed from:

- Two (2) QSEs within the first half of each reporting year (July 1 to December 31), and
- Two (2) QSEs within the second half of each reporting year (January 1 through June 30).

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Storm water from the following locations will be sampled:

- Storm water discharged from each of the proposed bio-detention basins into the adjacent wetlands and intake canal; and
- Storm water that bypassed the storm water treatment system due to high flows and discharges offsite.

Only storm events that occur during scheduled facility operating hours and are preceded by at least 48 hours without storm water discharges will be sampled. Samples will not be collected in dangerous weather conditions such as flooding, electrical storm, etc.

Immediately following collection, sample bottles for laboratory analytical testing will be capped, labeled, and documented on a chain-of-custody form provided by the analytical laboratory; sealed in a re-sealable storage bag; placed in an ice-chilled cooler, as close to 4°C (39.2°F) as practicable; and delivered within 48 hours to the California-certified laboratory (unless otherwise required by the laboratory). Only the sample containers provided by the laboratory to collect and store samples will be used.

Samples will be analyzed for total suspended solids, pH, oil and grease, and total iron. The pH analysis will be performed onsite within 15 minutes of collection using a calibrated field instrument. Acceptable sample methods and reporting limits are listed in Table 19.

5.5. Annual Observations of the Proposed Bio Detention Basins (*during* the dry season)

Visual observation of the condition of the proposed bio-detention basins must be made once during the period between June 1st and August 31st. Observations should be conducted with enough lead time that, should the inspections reveal issues requiring correction, sufficient time exists before October 1st to complete the required work prior to entering the wet season.

Observations must be made:

- By the Senior Environmental Field Specialist or designee.
- During daylight hours and during scheduled facility operating hours.
- When the basins are completely drained of water or water levels within the basin are low enough to permit assessment of the condition of the basin bottom.

Points of observation for each basin during the annual inspections are:

- Each basin shall be fully drained, as required, to permit access as needed and facilitate unobstructed views of the entire basin bottom. Basins should be drained for as short a period as possible to preserve the ecology within.
- Determination of total and annual accumulation of sediment within the basin as indicated by a stage storage stick calibrated to the basin of interest. The observed depth of sediment within the basin will be compared to the previous year's observation and the base elevation of the as built basin to determine the total volume of sediment accumulated during the preceding rainy season and all previous rainy seasons experienced by the basin. All basins have been designed to accommodate at least 150% of the minimum required volume to satisfy the LID and IGP criteria for volume based treatment; this feature allows for accumulation of sediment within the basin. When the total accumulated sediment within the basin is observed to be approaching 20% of the original as built total basin volume, the sediment shall be excavated to restore the

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basin to the original as built configuration. This will ensure the basin retains at least 120% of the minimum required volume.

- Each basin should be inspected for significant debris, floatables or trash which may interfere with the function of the basins outfalls or bypasses. All foreign objects shall be removed and deposited in an appropriate facility.
- Inspection for evidence of invasive/undesirable species within the basins. Undesirable species shall be removed to the extent feasible to preserve optimal treatment of detained water.
- Inspection for evidence of discharge pipes exhibiting decreased performance and/or flow due to invasive root systems and/or sediment deposits. Suspect discharge pipes shall be cleared with conventional "roto-rooting" techniques to restore pipe performance on an as needed basis given qualitative observation of overall performance.
- Inspection for evidence of erosion at the basin inlets and outfalls. Any erosion observed shall be repaired prior to entering the next rainy season.

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6. Results, Conclusions and Recommendations

This report, documenting preliminary design and sizing of the storm water management system for the proposed HBPP FSR plan, has identified numerous critical design considerations and preliminary results for guiding the construction detailing process.

- The adoption of the proposed bio-detention basin design satisfies all the necessary storm water capture and treatment criteria to comply with NCRWQCB's recommendations, with no additional LID features required.
- The proposed bio-detention design allows for compliance with applicable storm water treatment methods while minimizing maintenance; a single generic passive treatment system will decrease long term costs associated with maintenance.
- The proposed capture volume of the bio-detention basins was increased by 150% over the minimum required by the LID methodology to support the future induction of these storm water treatment features into the governing Industrial General Permit.
- Overall performance of the proposed bio-detention basins is heavily influenced by the infiltration rate of the soil/sand/rock filter media. Faster infiltration rates provide increased quantities of storm water being physically filtered, but may not provide adequate levels of bio-remediation. During construction detailing the constituents of concern within the storm water runoff should be identified and the infiltration rate specified to provide an appropriate balance between physical filtering and bio-remediation.
- If increased levels of infiltration are desired, in addition to specifying a filtration media within the bio-detention basins that exhibits faster infiltration rates; steepening the sides slopes of the basins will expand the available surface area for infiltration and increase infiltration rates.
- Although a direct comparison of pre- and post-development hydrographs at outfalls is not possible, by inspection the hydrographs at the outfalls of the proposed bio-detention basins will significantly dampen flows entering the adjacent wetlands due to delays caused by infiltration times and use of the proposed flow control structures.
- Several vegetated swales with outlets into the proposed bio-detention basins will provide some level of pretreatment; these swales are not included in the overall determination of effectiveness and will only act to improve the treatment of storm water over anticipated levels.
- All proposed drainage conveyance features adequately transport 100% of the peak flows associated with the 10-year design storm. The same storm water conveyance system subjected to 25-, 50-, and 100-year design storms exhibits minor quantities of surface inundation and temporary flooding in localized areas.
- Bounding the possible proposed FSR site grades between the finished floor elevation of preserved buildings, adjacent grades at HBGS, and the relatively high inlet elevations of treatment basins, necessitates the adoption of fairly flat (0.25% - 0.5%) surface swales and excludes the use of subterranean pipes for the majority of the system.
- The proposed low slope swales with enough capacity to adequately convey 100% of the anticipated peak flows resulting from the application of a 10-year design storm have widths ranging from 2 feet to 8 feet depending on the area contributing to the runoff.
- When compared to the maximum build out of HBPP, prior to starting decommissioning, the proposed FSR plan achieves a net reduction in both impervious and gravel surfaces,

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0.8 acres and 0.68 acres respectively. This net reduction accounts for deliberate overestimation of proposed impervious areas including the full build out potential of the Unit 1/2/3 laydown area and potential expansion of the ISFSI truck turnaround.

- Utilization of an expanded historical data set permitted detailed peak flow analysis and assessment of the proposed bio-detention basins' performance under observed historical precipitation events. Average results for the 3 large bio-detention basins indicate that when proposed infiltration rates are equal to 4 in/hr, 93% of the inflow runoff is infiltrated; when proposed infiltration rates are equal to 0.5 in/hr, 66% of the inflow runoff is infiltrated through the filtration media. Average results for the 4 remaining smaller bio-detention basins indicate that when proposed infiltration rates are equal to 4 in/hr, 87% of the inflow runoff is infiltrated; when proposed infiltration rates are equal to 0.5 in/hr, 32% of the inflow runoff is infiltrated through the filtration media.
- Utilization of the historic data set permitted identification and adoption of the slightly more conservative value of the 85th percentile, 24-hour design storm depth of 0.68 inches compared to the published value of 0.65 inches; thereby producing slightly larger bio-detention basins.

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7. Appendix of Figures

Figure 1: HBPP Site Configuration - 2013

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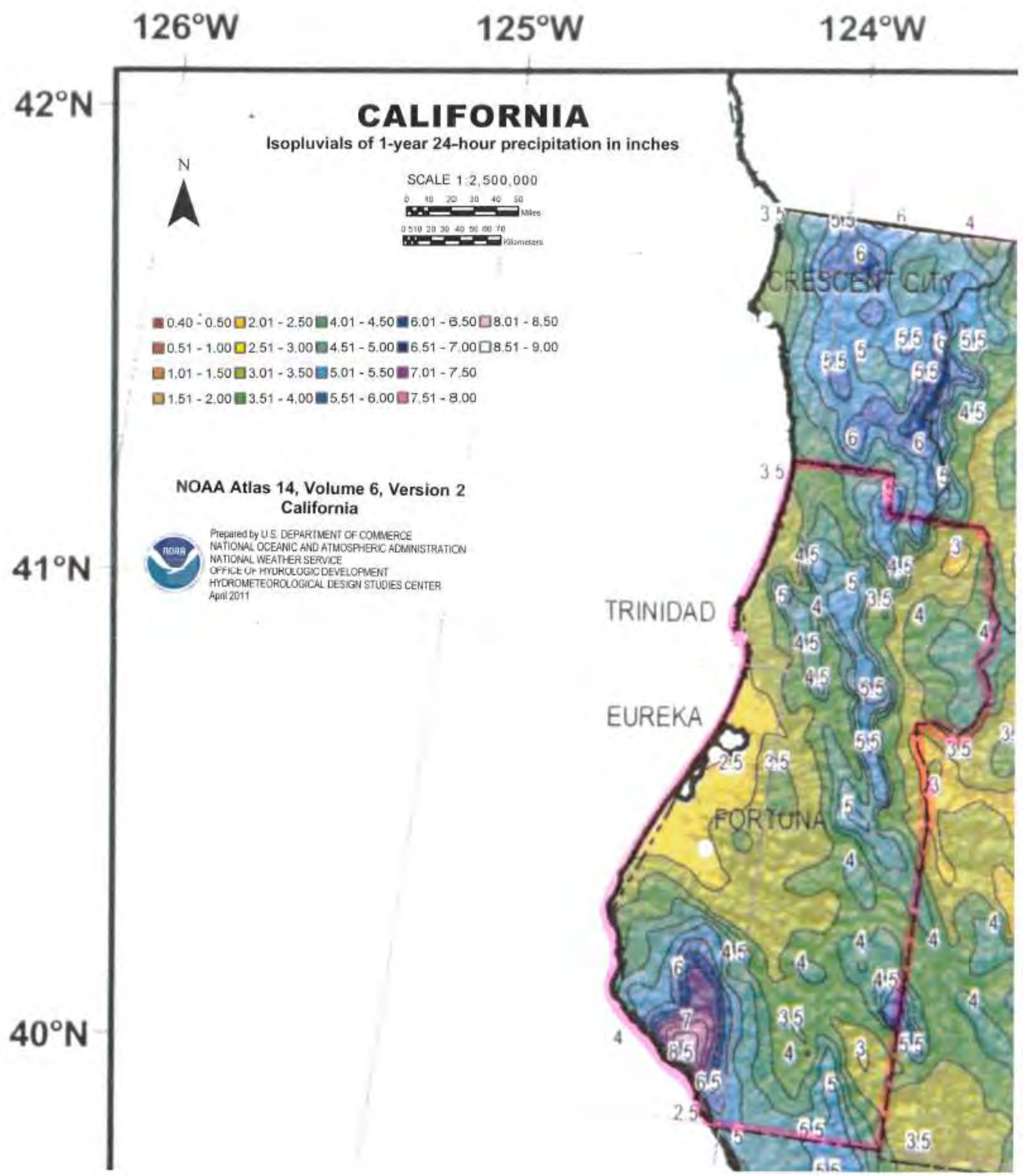

Figure 2: 1 Year, 24-Hour Isopulvials within Humboldt County (NOAA 2014)

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Figure 3: Location of Weather Data Collection vs. Location of the HBPP Project Site

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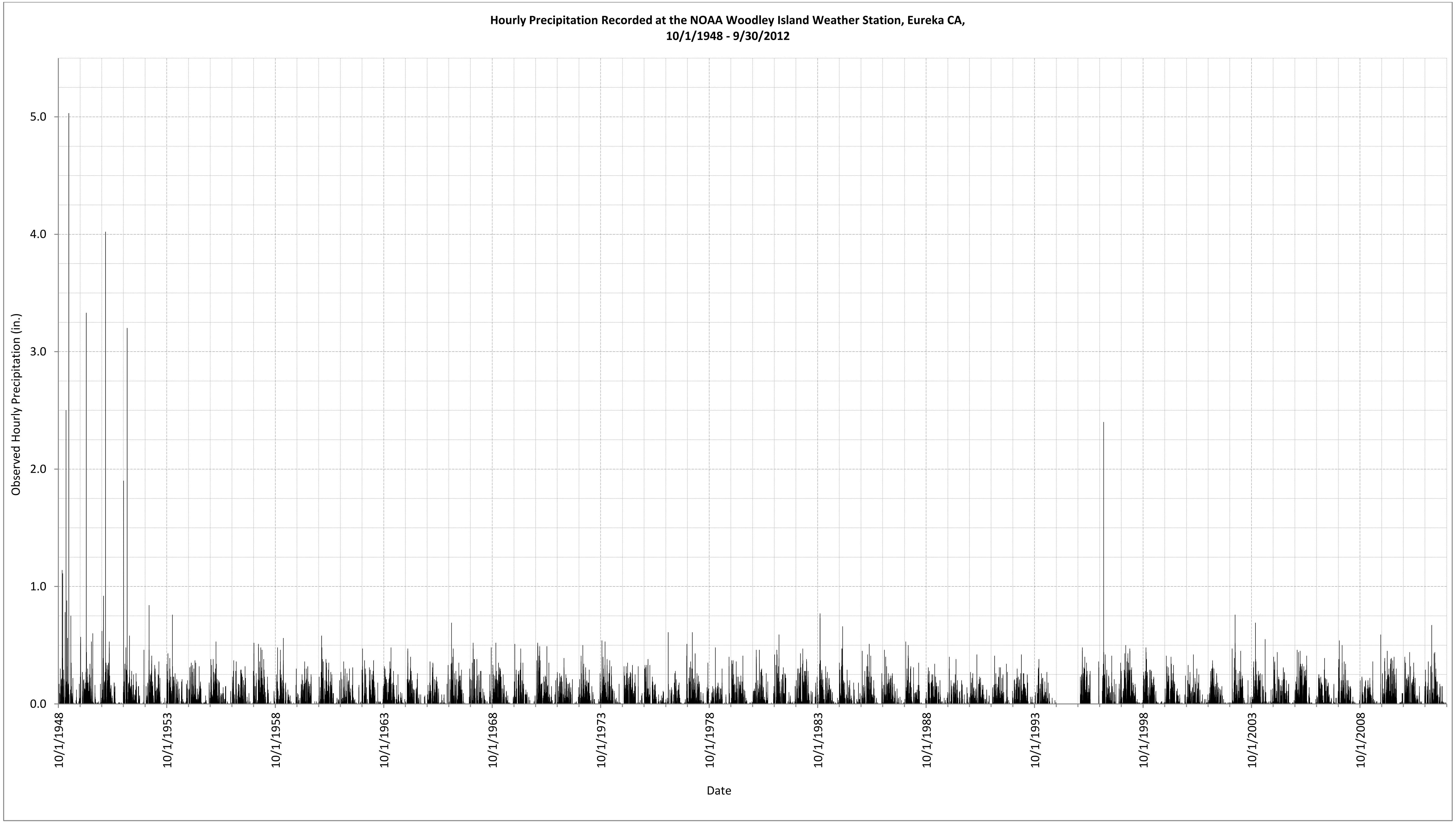

Figure 4: Observed Hourly Precipitation Values for the 64 Water Years Constituting the Entire Historical Data Set; Presented as Continuous Timeline

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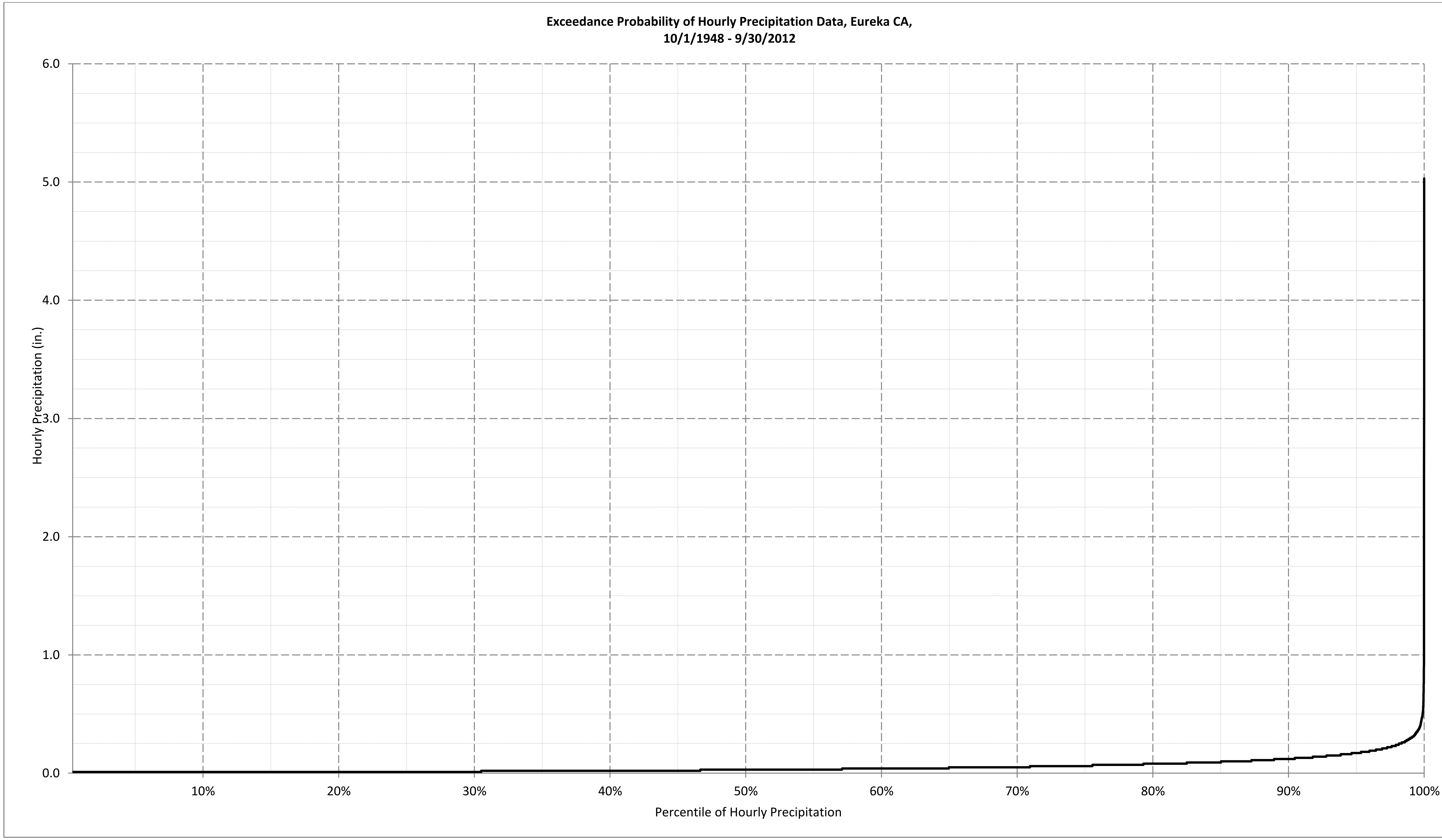

Figure 5: Observed Hourly Precipitation Values for the 64 Water Years Constituting the Entire Historical Data Set; Presented as Probability Curve

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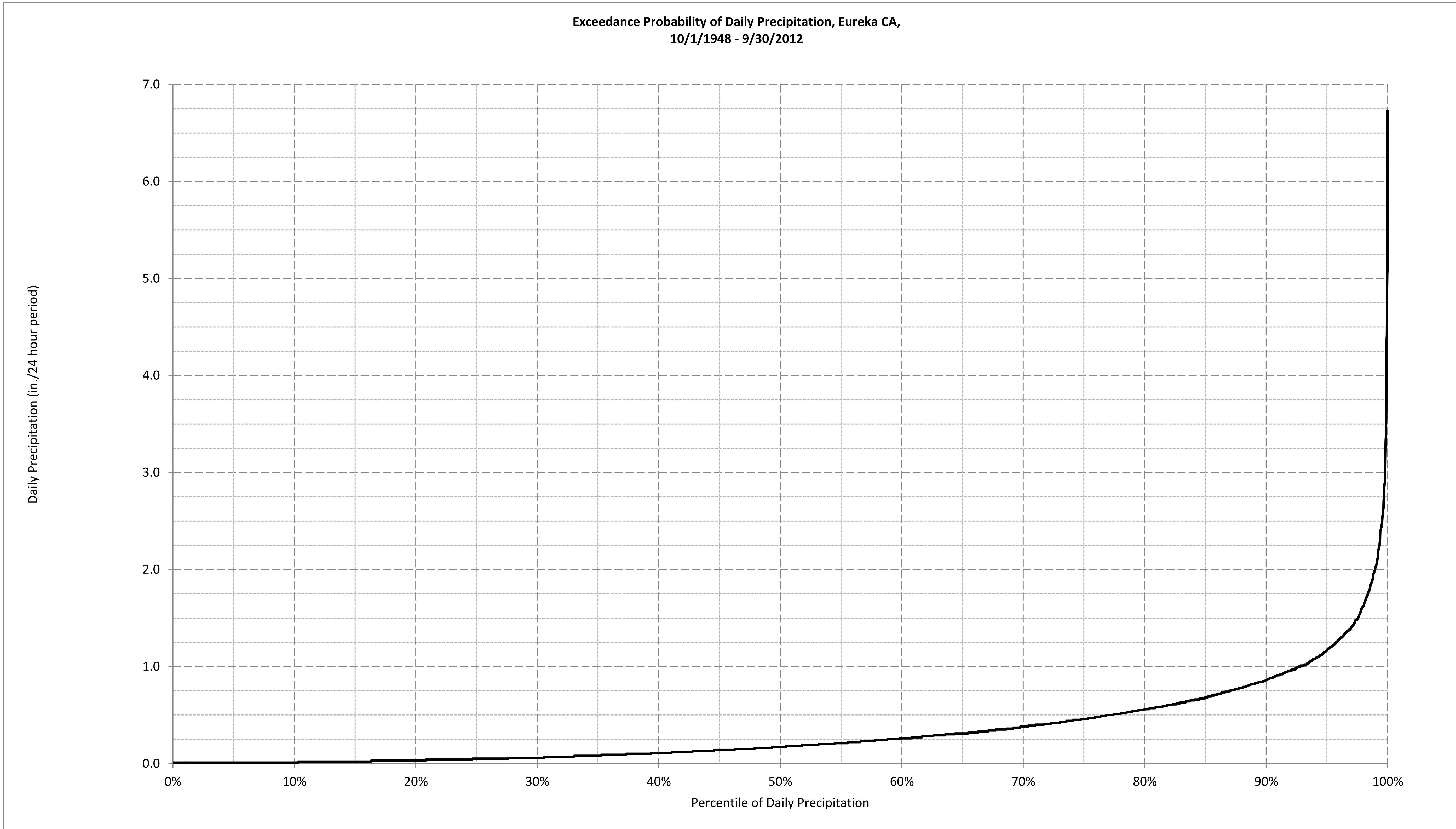

Figure 6: Observed Daily Precipitation Values for the 64 Water Years Constituting the Entire Historical Data Set; Presented as Probability Curve

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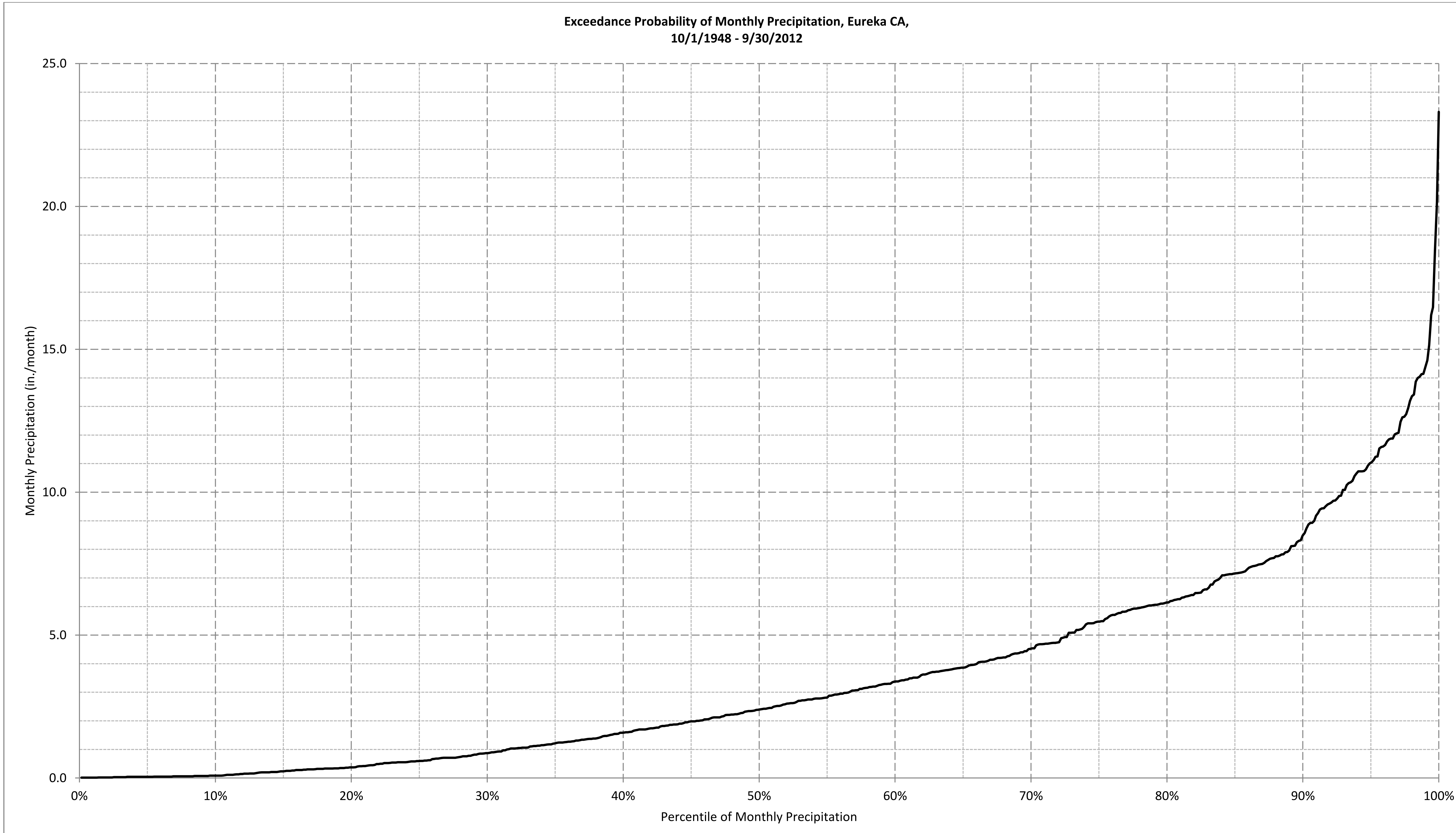

Figure 7: Observed Monthly Precipitation Values for the 64 Water Years Constituting the Entire Historical Data Set; Presented as Probability Curve

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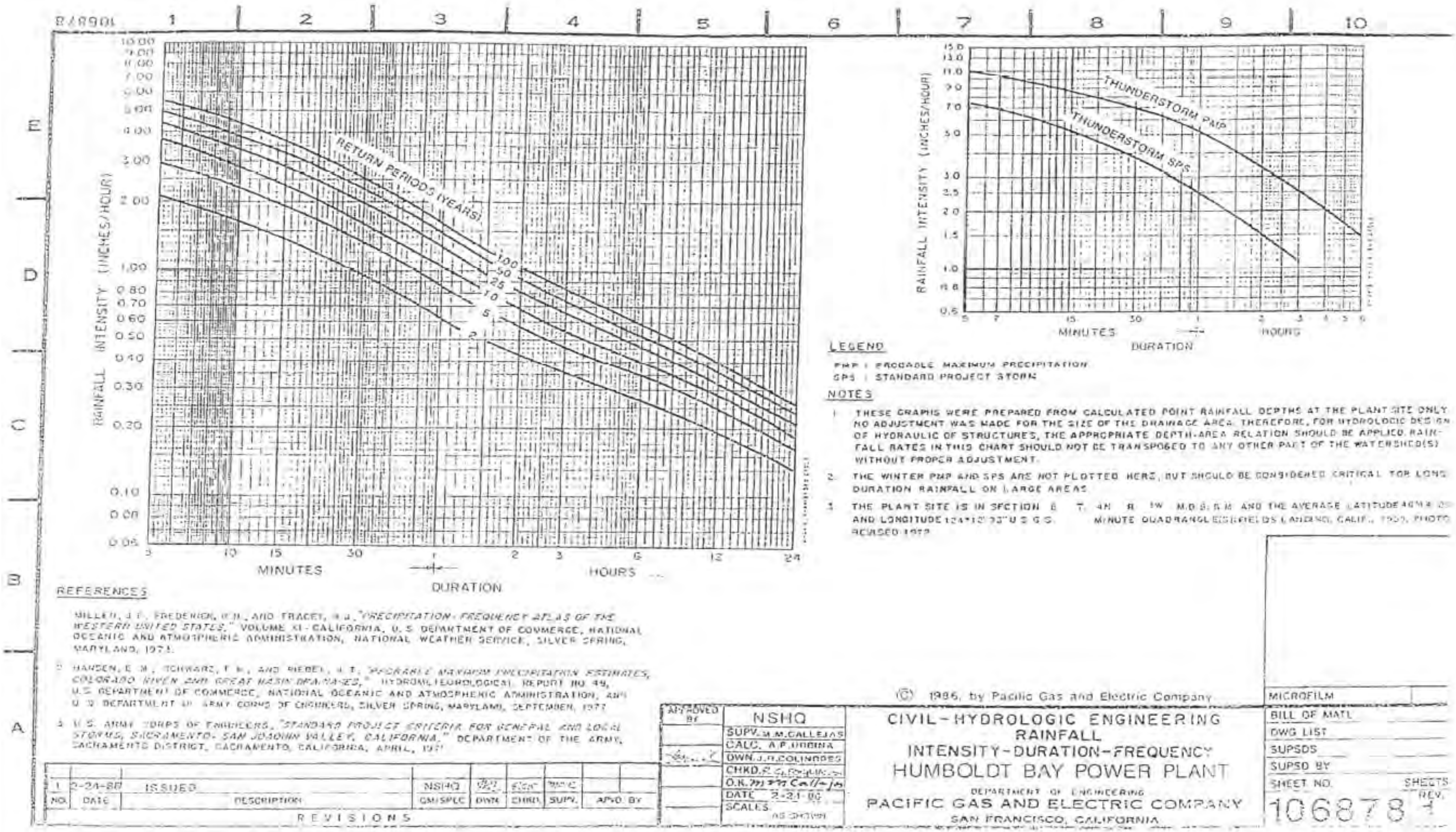

Figure 8: HBPP Specific Intensity-Duration-Frequency (IDF) Curve Set Used to Develop IDF Curves Utilized in Analysis

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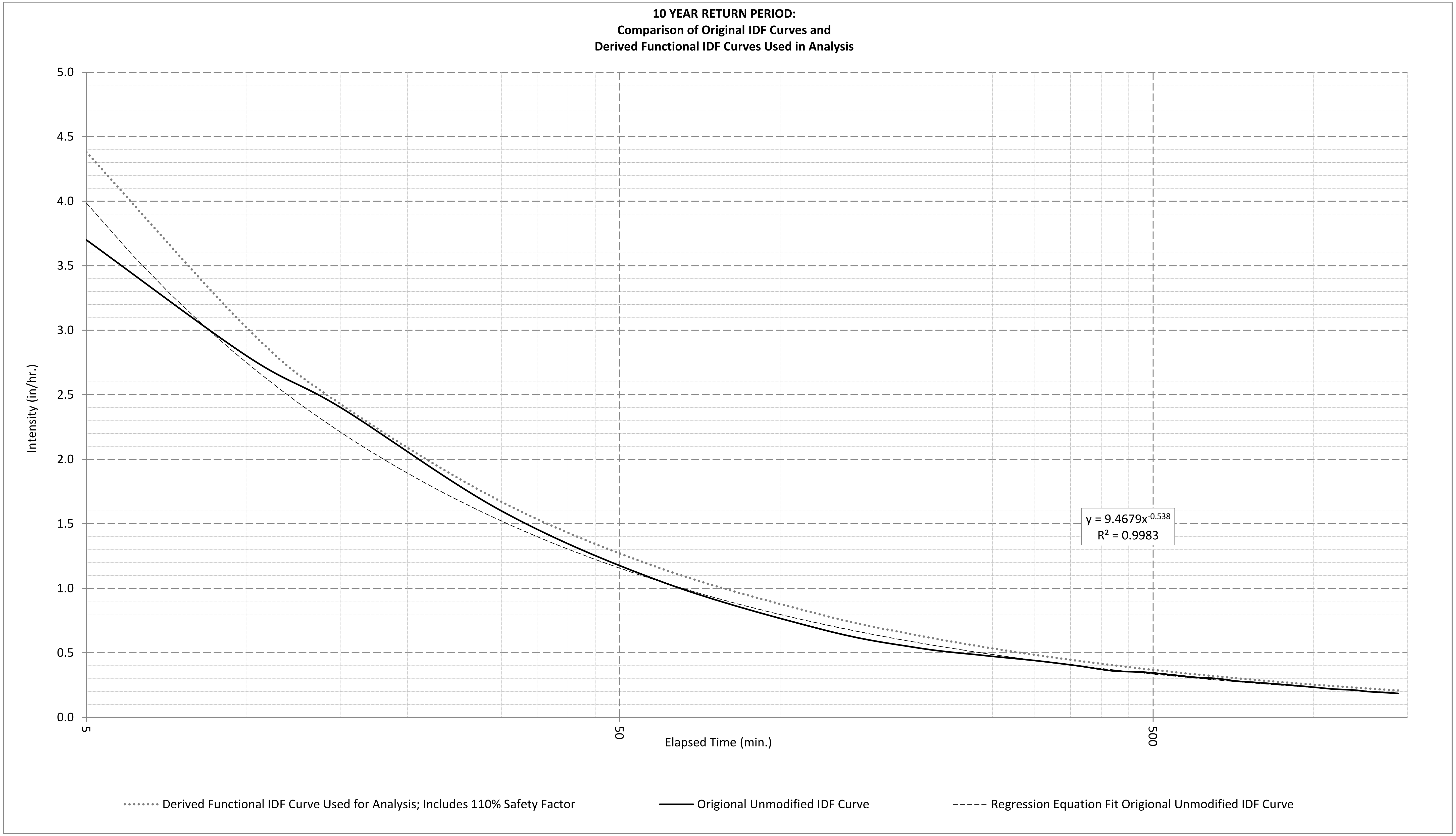

Figure 9: Functionally Derived 10 Year IDF Curve Used in Analysis

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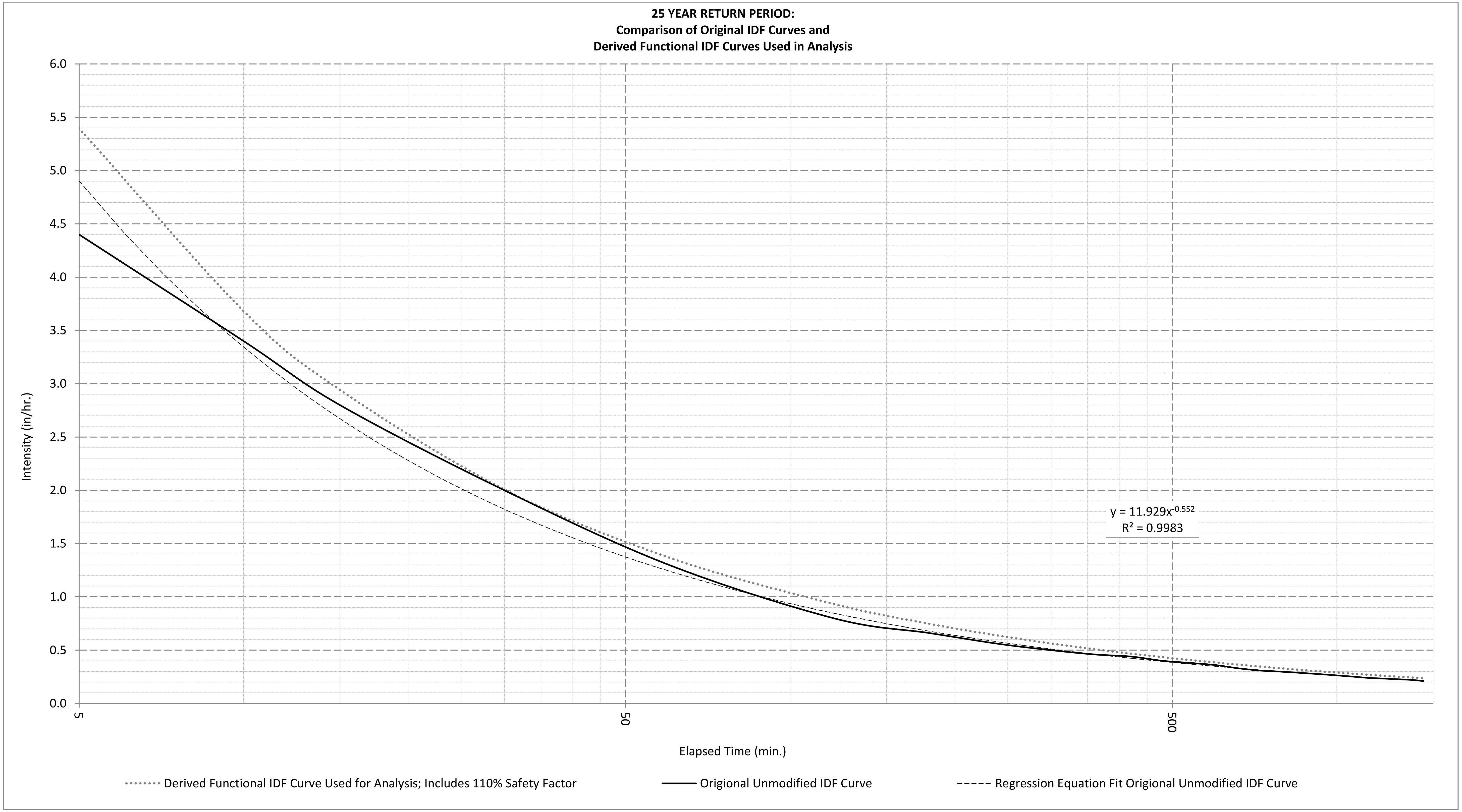

Figure 10: Functionally Derived 25 Year IDF Curve Used in Analysis

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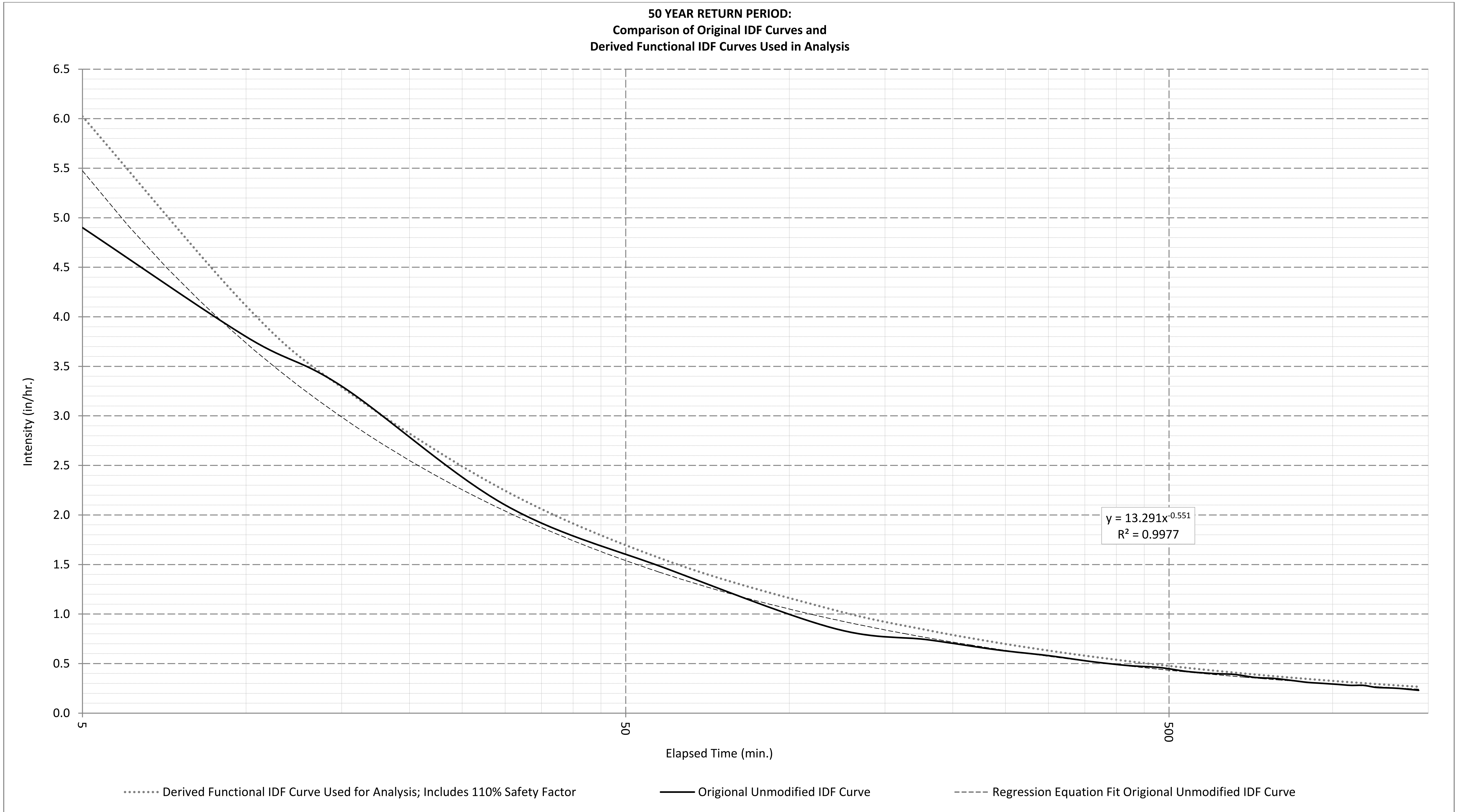

Figure 11: Functionally Derived 50 Year IDF Curve Used in Analysis

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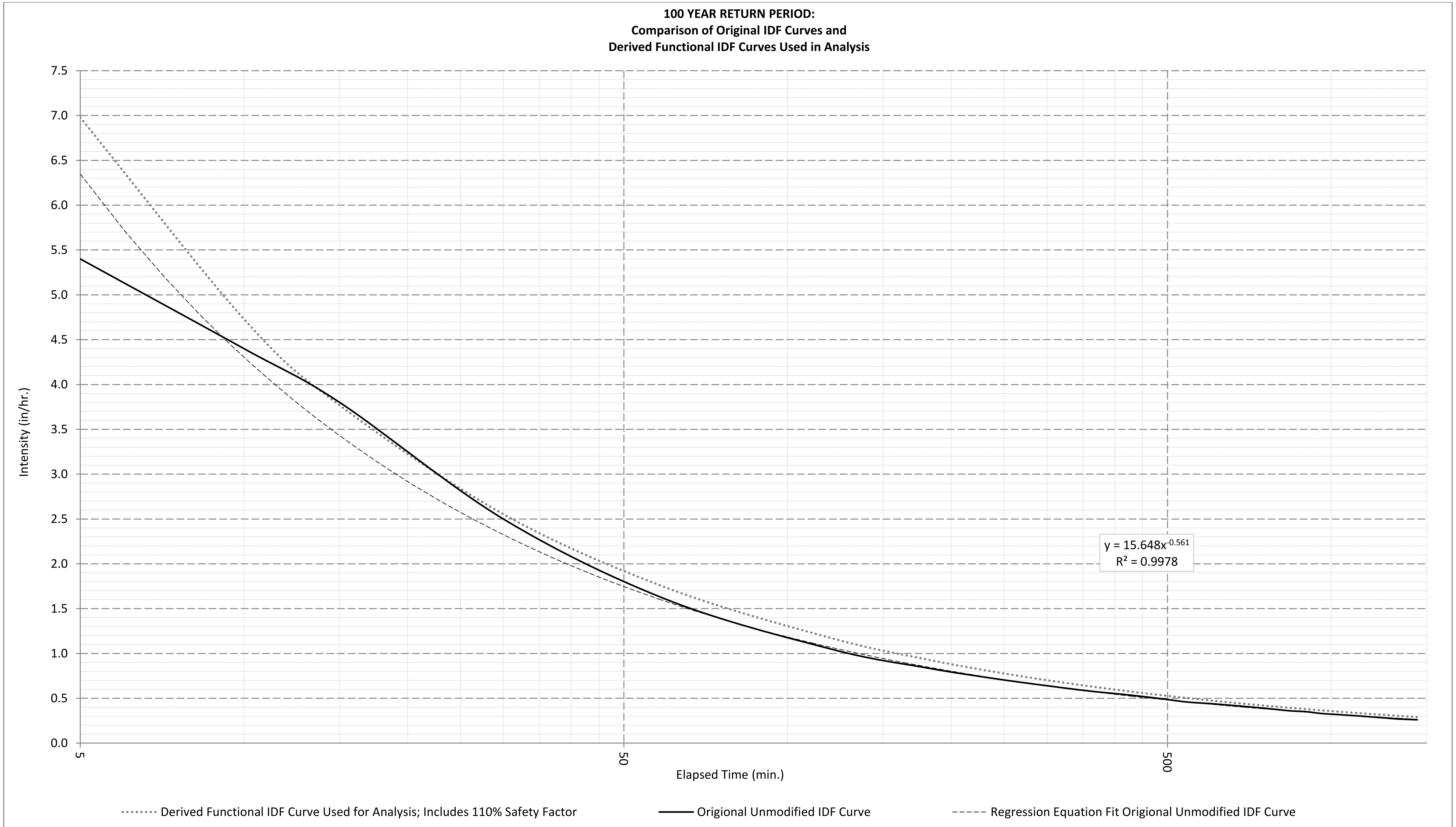

Figure 12: Functionally Derived 100 Year IDF Curve Used in Analysis

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Figure 3-1 Average velocities for estimating travel time for shallow concentrated flow

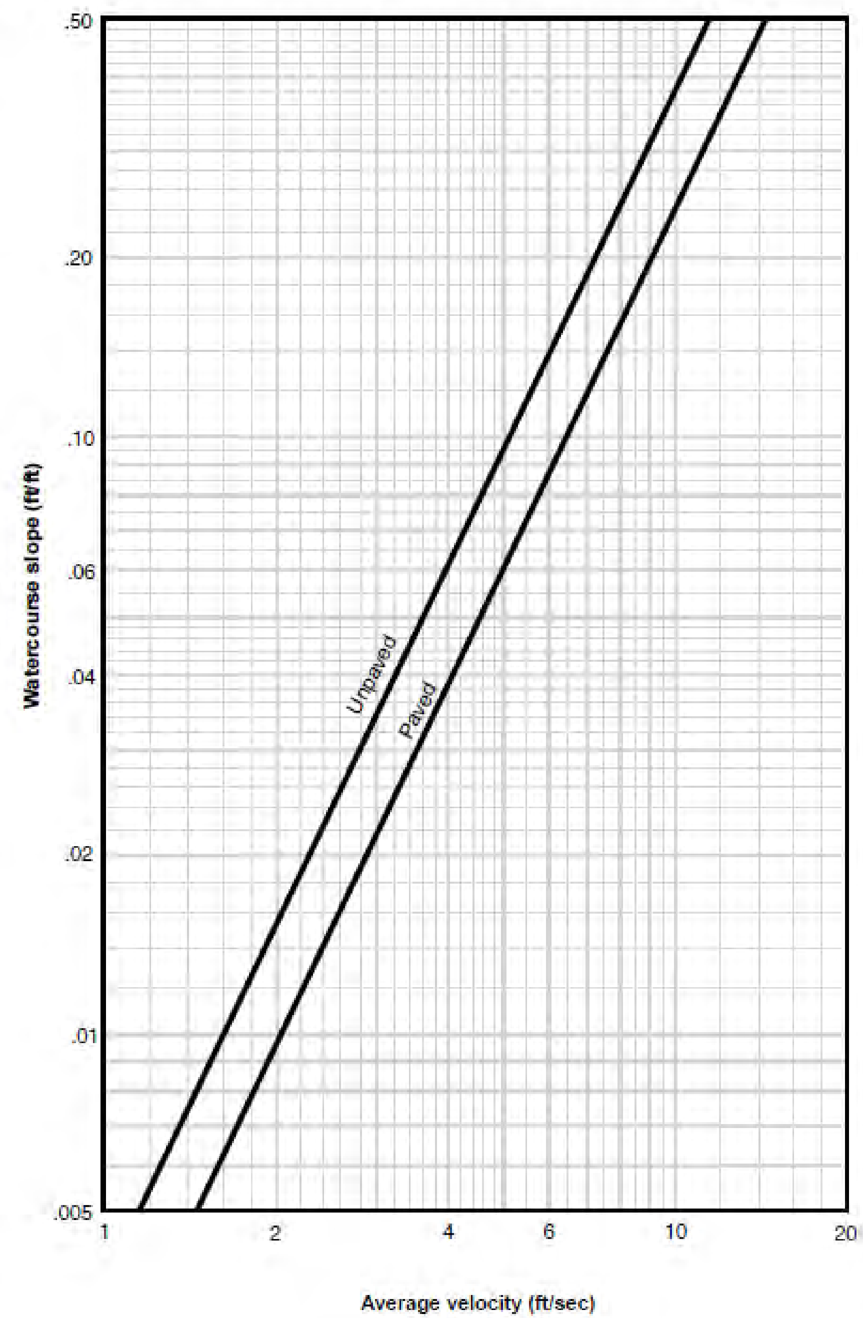

Figure 13: Shallow Concentrated Flow Curves Presented in the USDA TR-55 Manual (USDA 1986)

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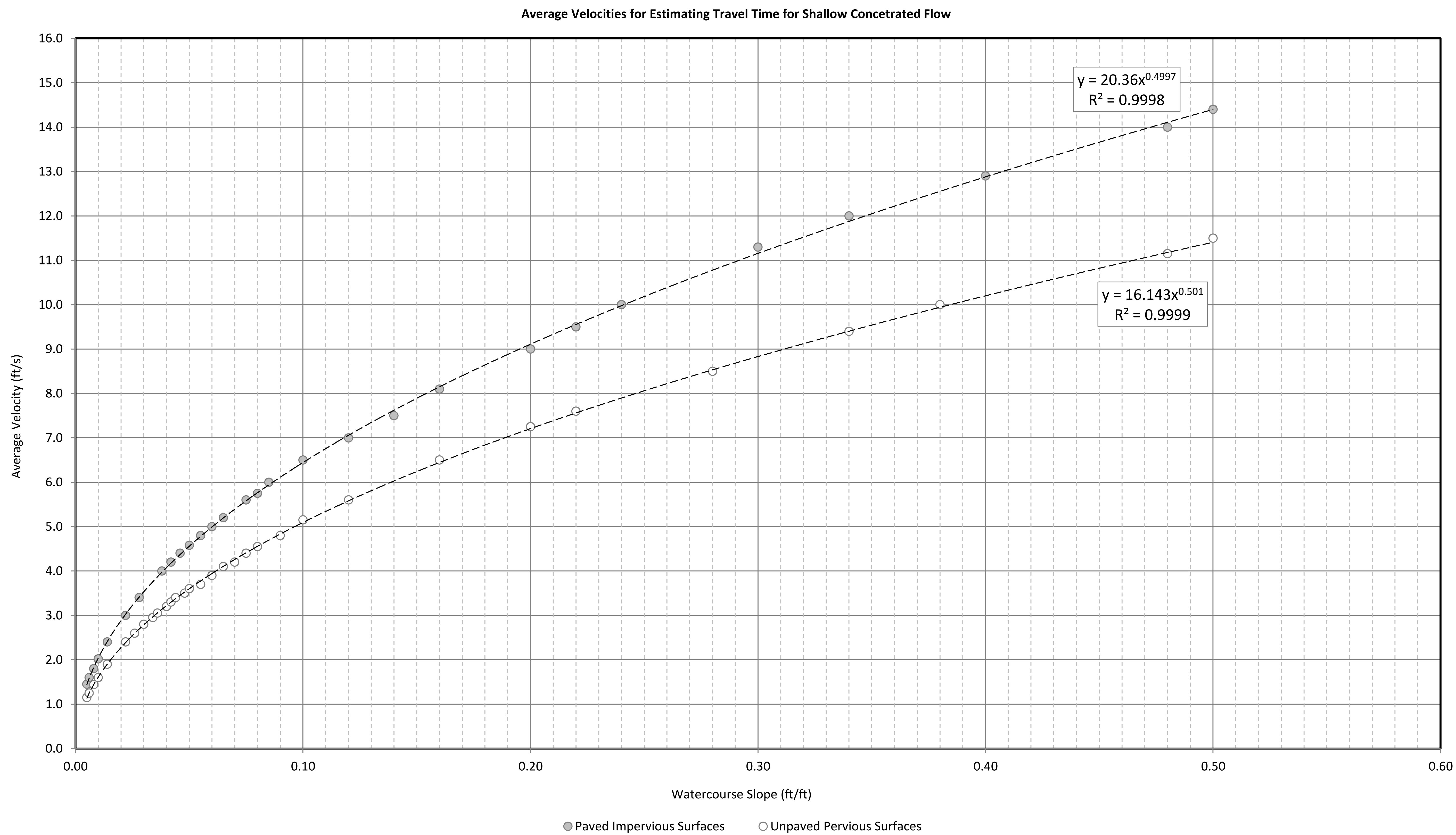

Figure 14: Transformation of Curves from Figure 13 into Functional Relationships Utilized in Analysis

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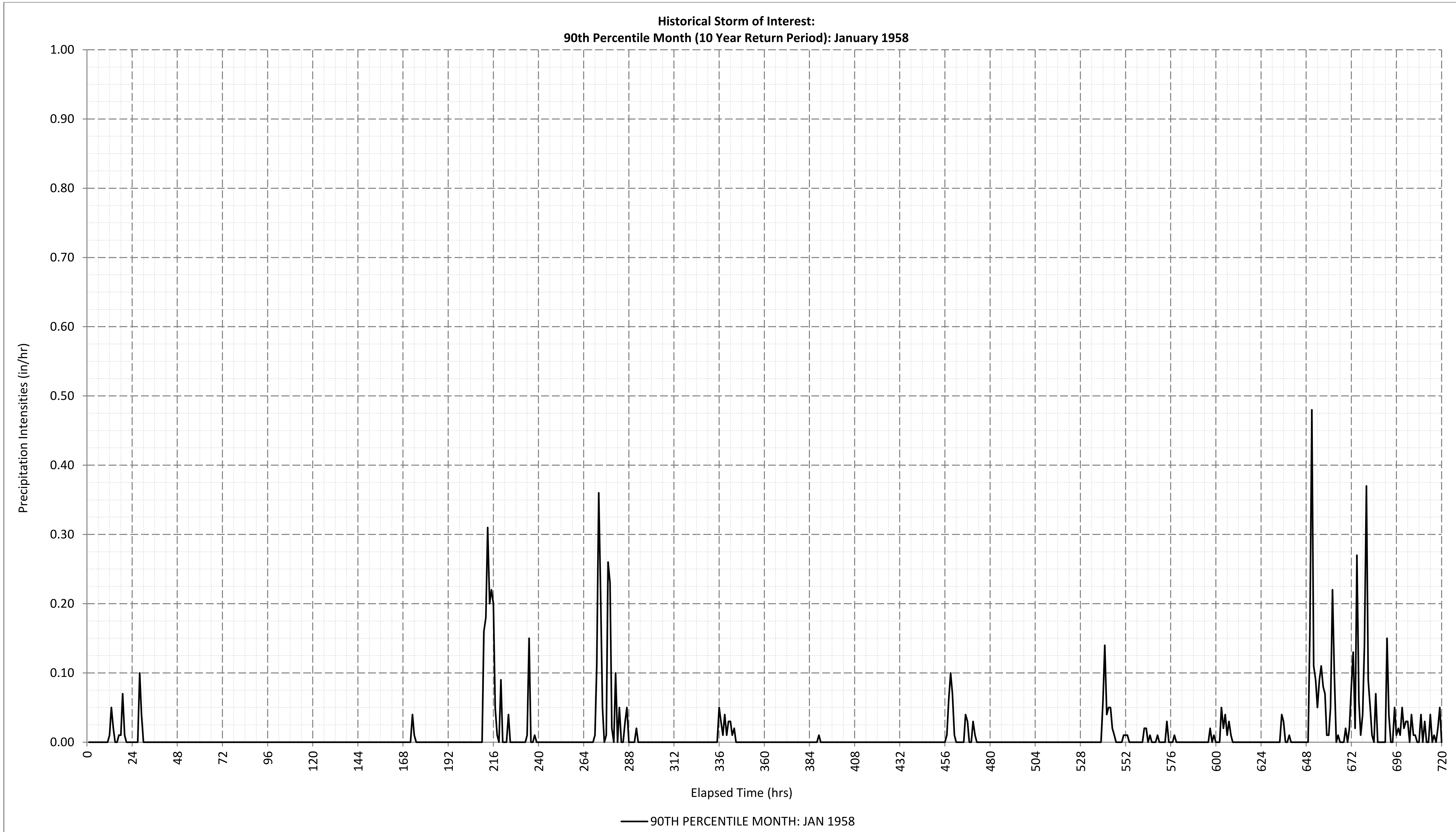

Figure 15: Continuous Observed Historical Precipitation Intensities over the Course of the 90th Percentile Month

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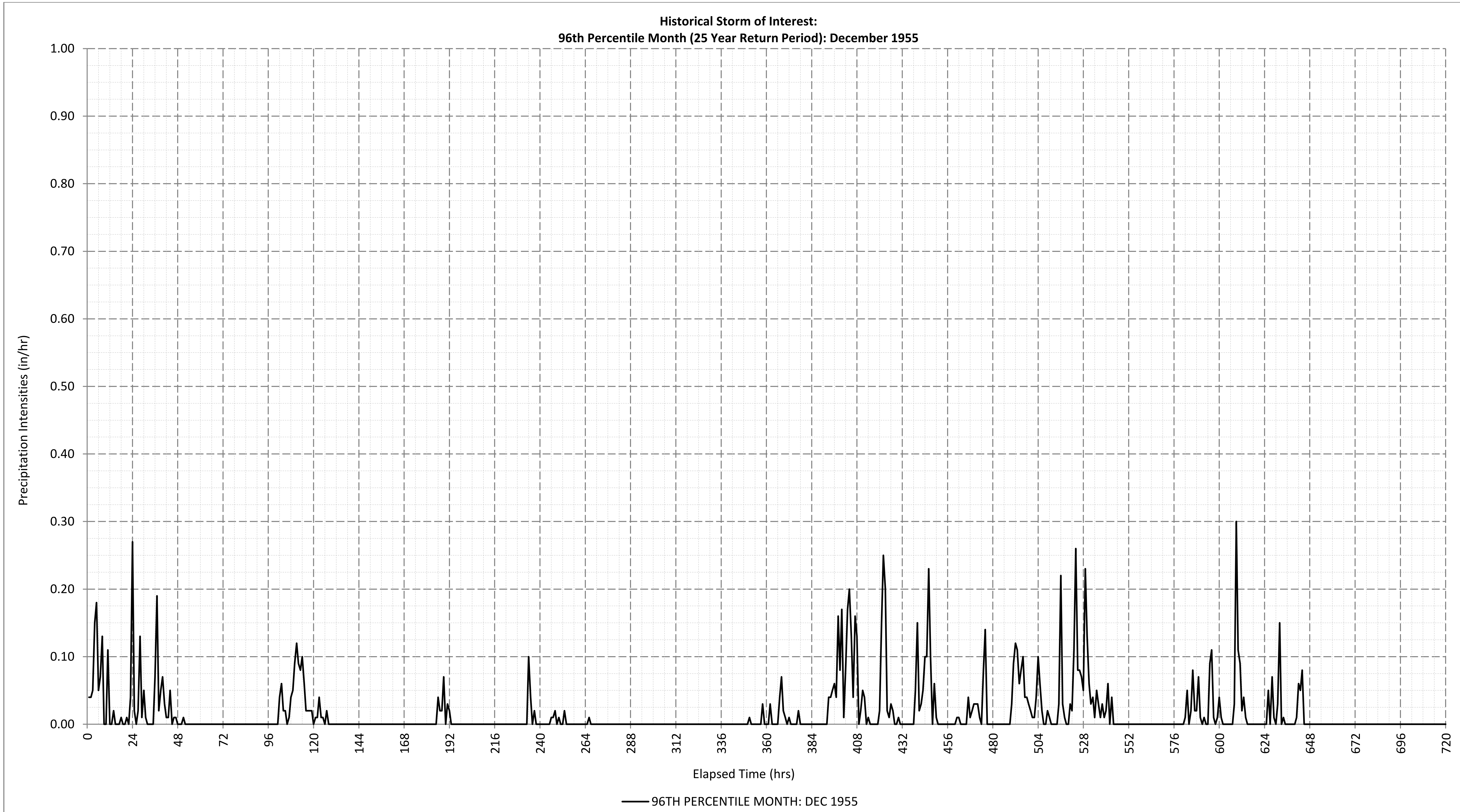

Figure 16: Continuous Observed Historical Precipitation Intensities over the Course of the 96th Percentile Month

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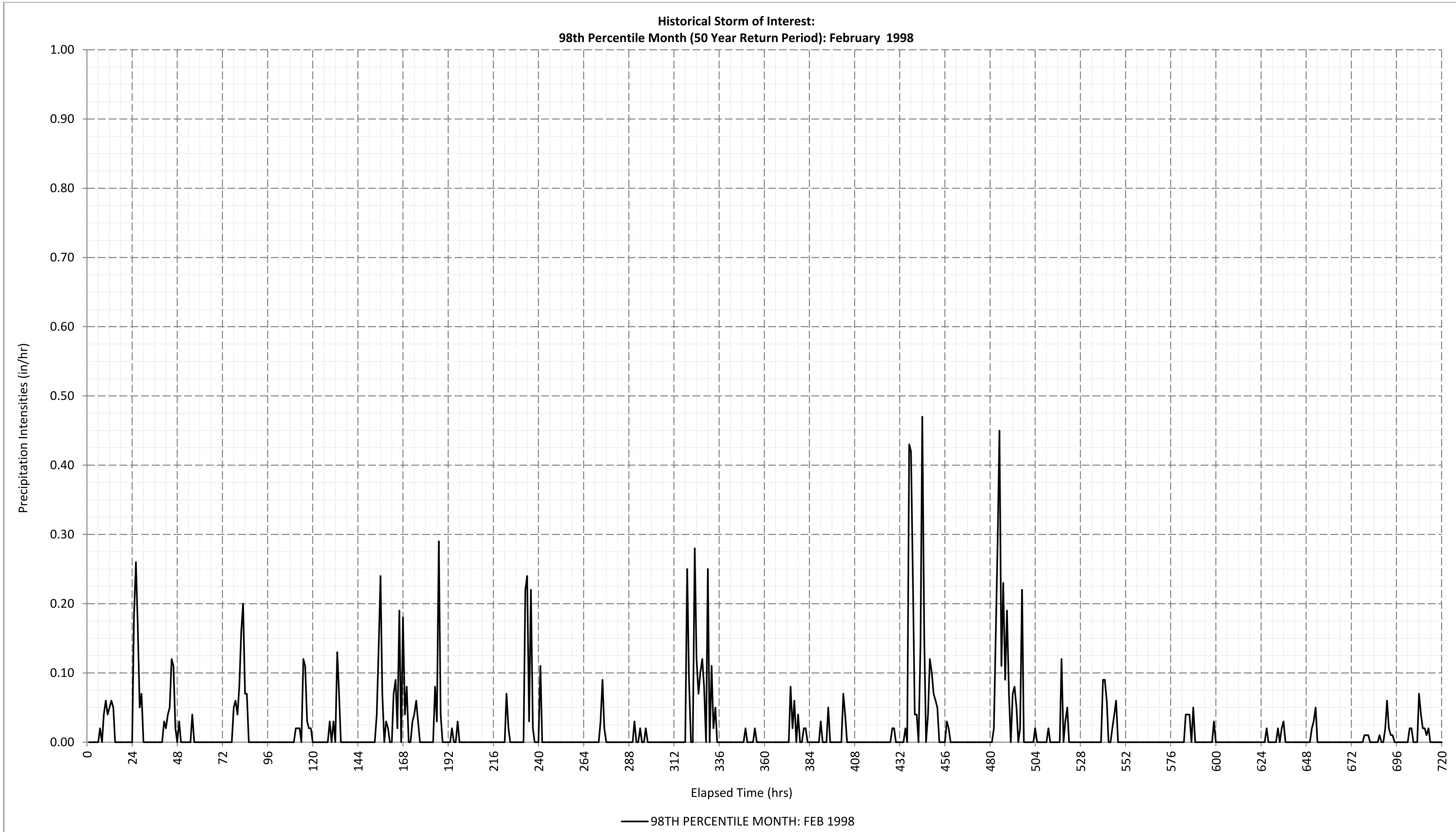

Figure 17: Continuous Observed Historical Precipitation Intensities over the Course of the 98th Percentile Month

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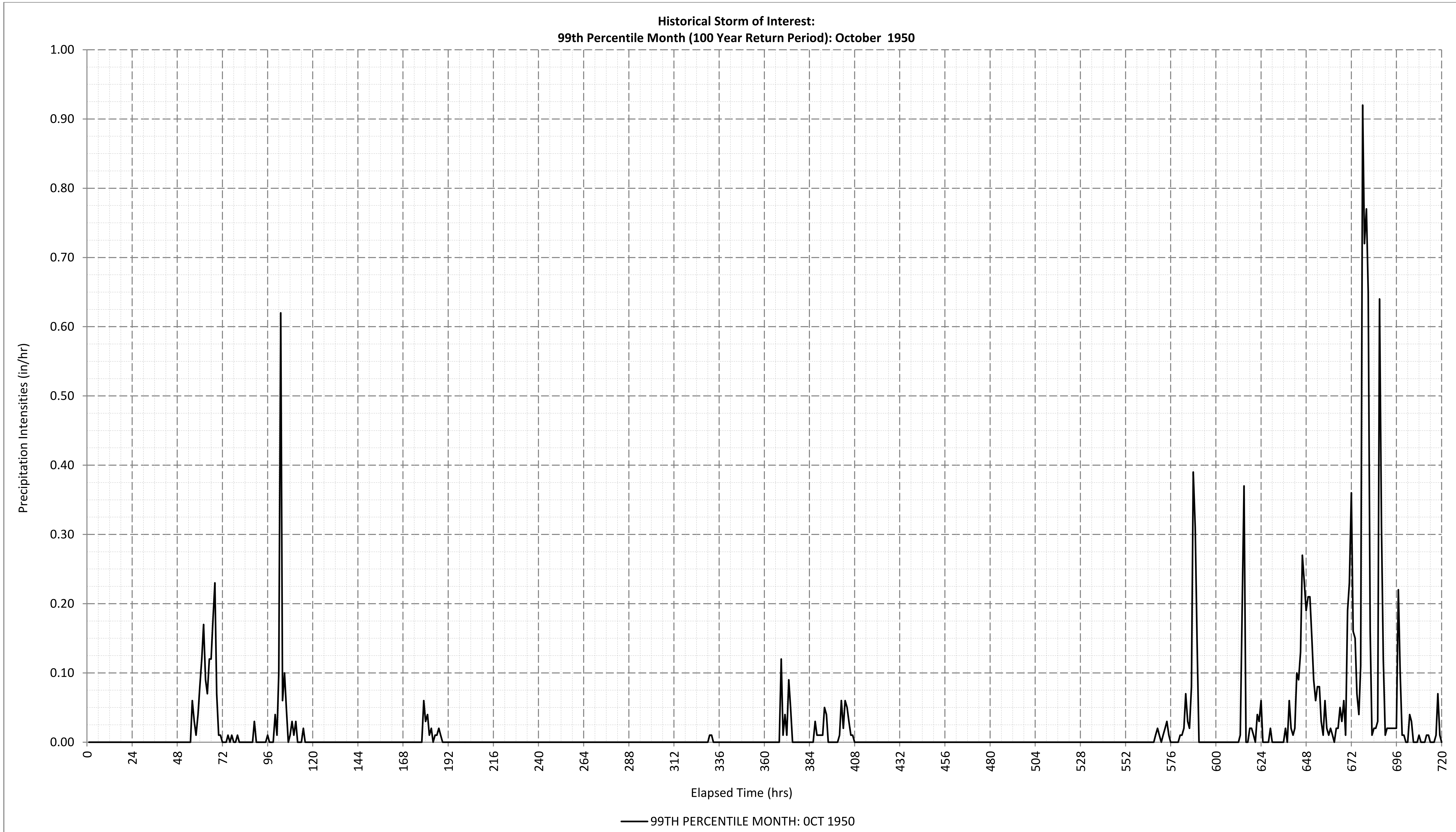

Figure 18: Continuous Observed Historical Precipitation Intensities over the Course of the 99th Percentile Month

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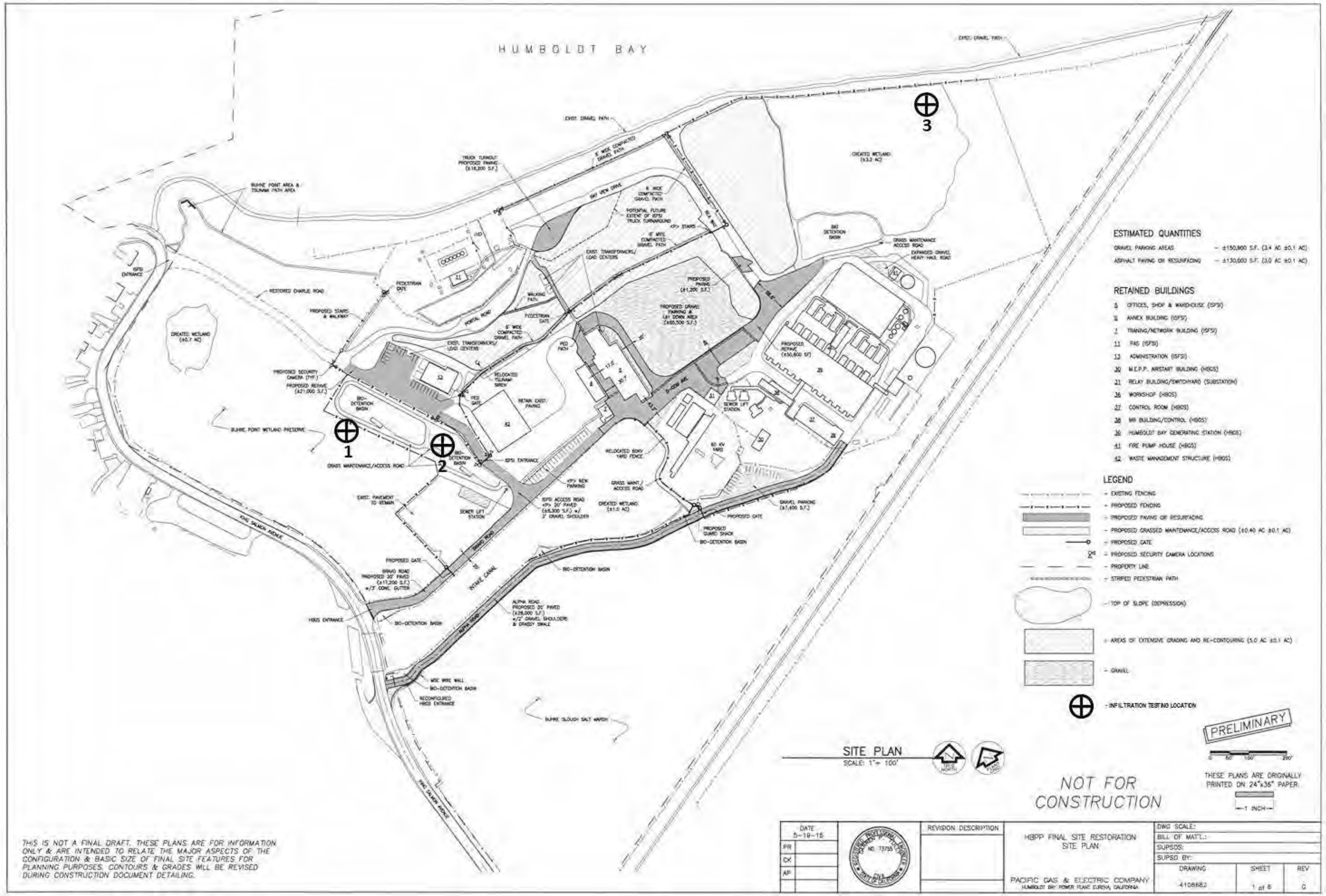

Figure 19: Locations of Infiltrometer Testing Compared to Proposed Bio-detention Basin Locations

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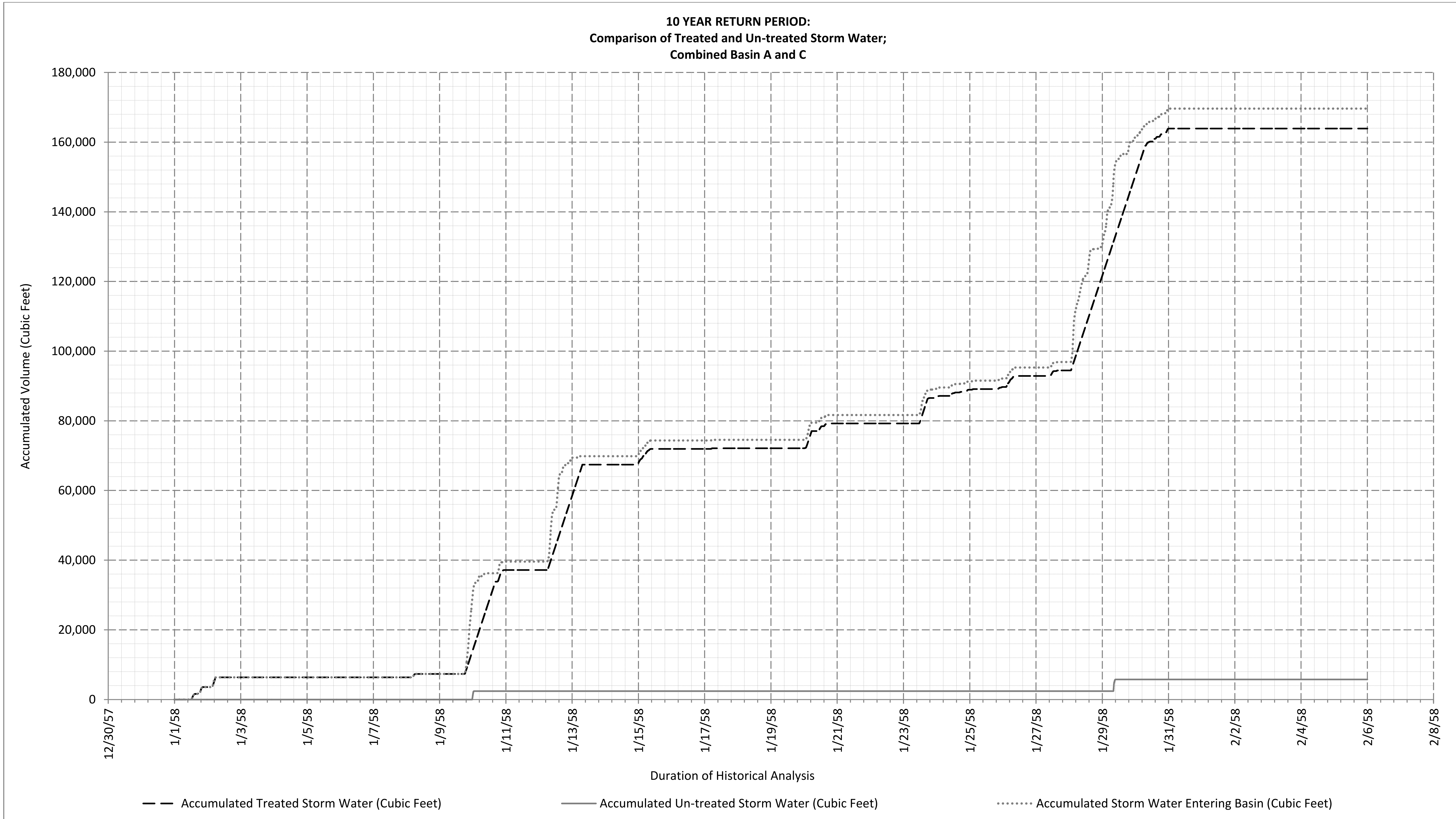

Figure 20: Hydrographs and Performance Curve of Combined Basins A and C, over the Course of the 90th Percentile Month with Saturated Basin Media and Infiltration Rate of 0.75 In/hr

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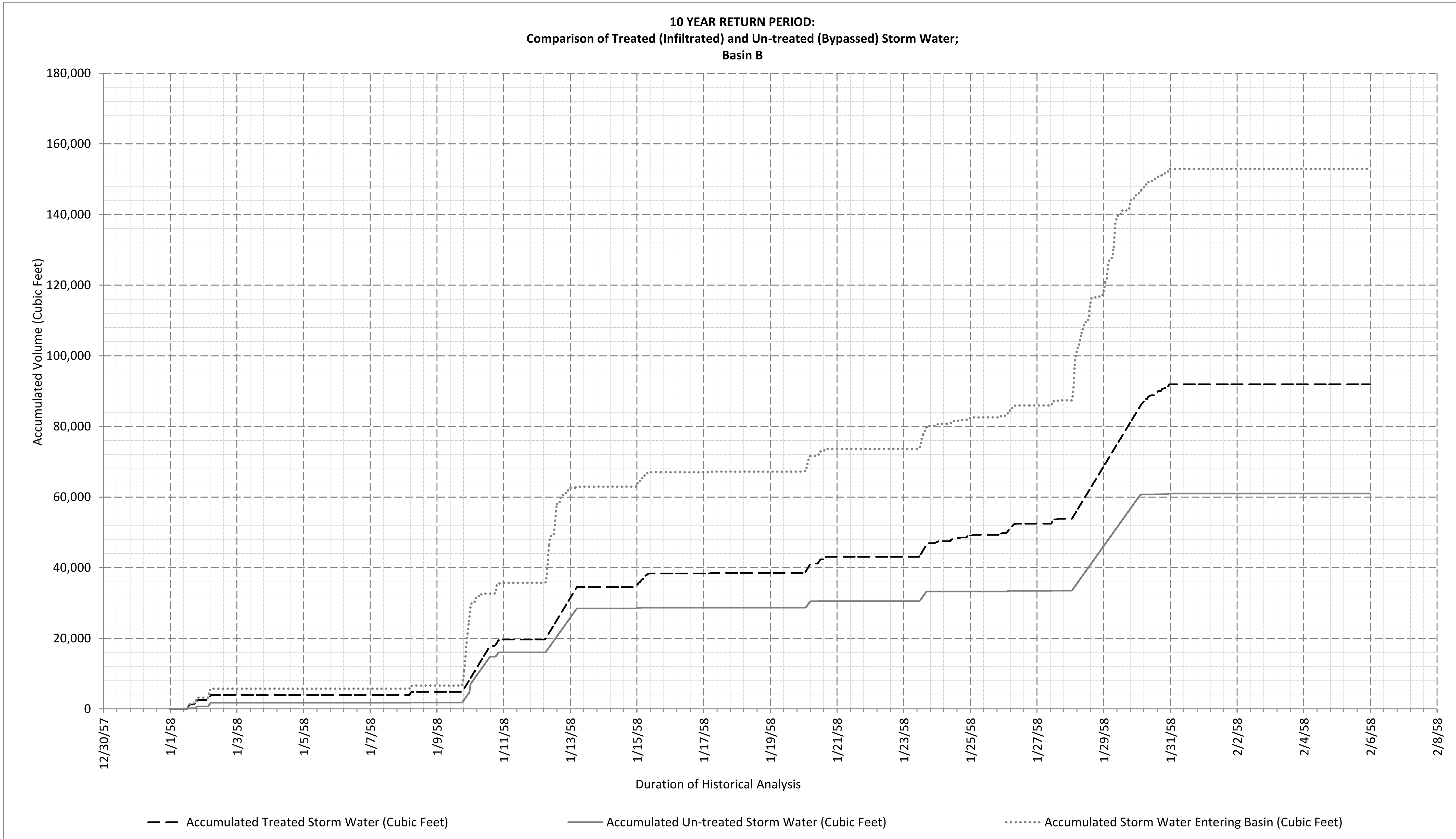

Figure 21: Hydrographs and Performance Curve of Basin B, over the Course of the 90th Percentile Month with Saturated Basin Media and Infiltration Rate of 0.75 In/hr

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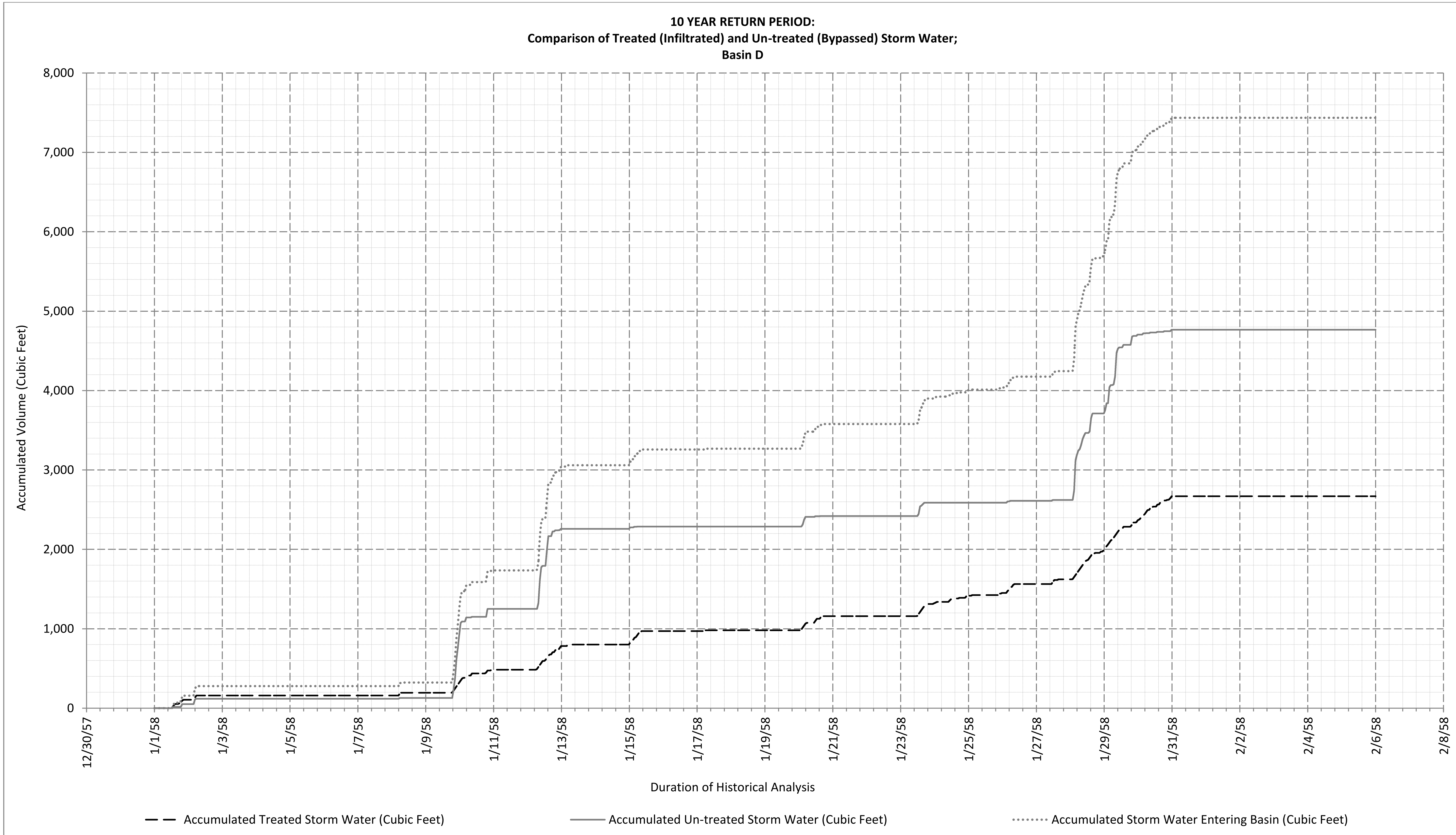

Figure 22: Hydrographs and Performance Curve of Basin D, over the Course of the 90th Percentile Month with Saturated Basin Media and Infiltration Rate of 0.75 In/hr

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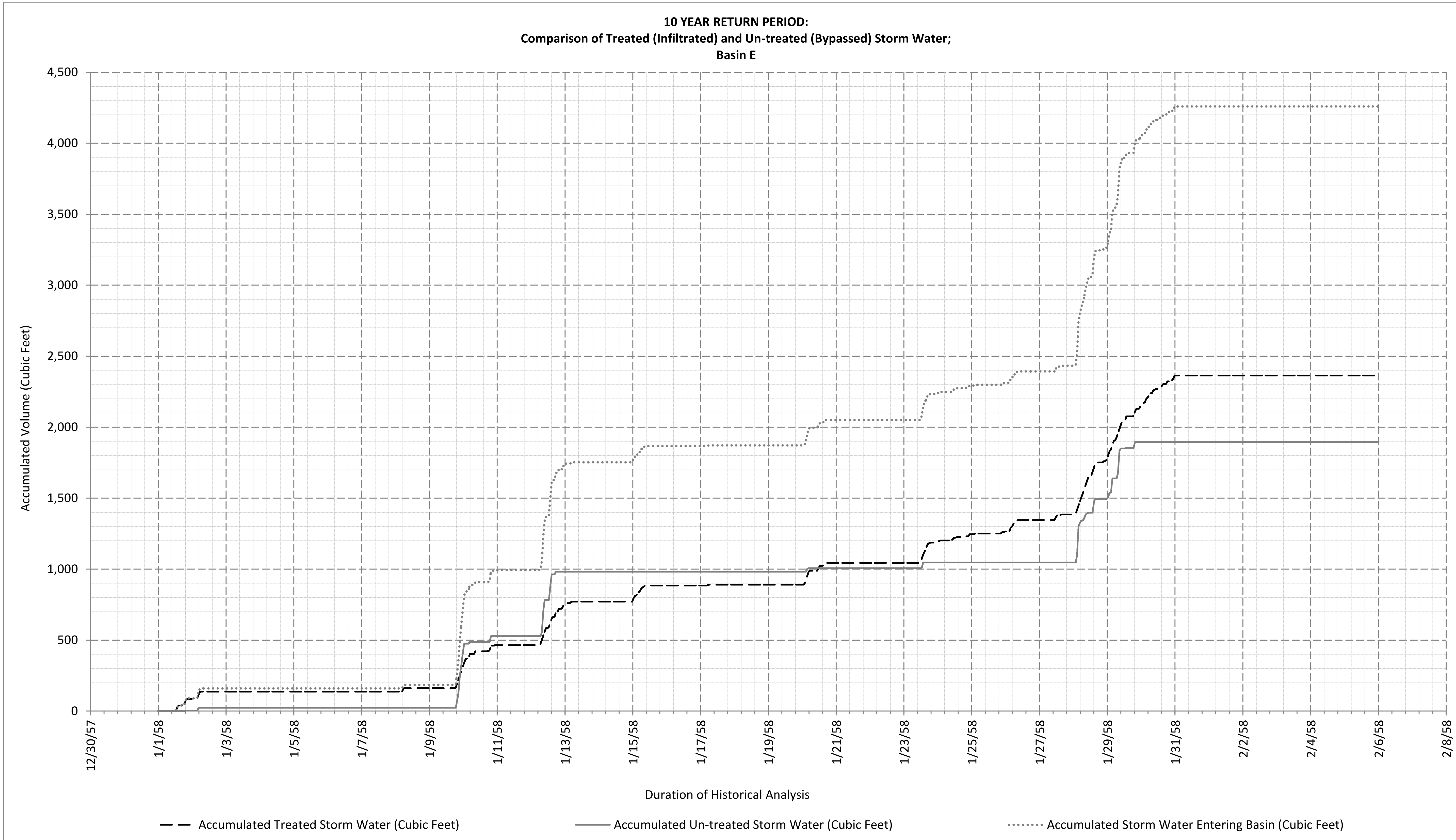

Figure 23: Hydrographs and Performance Curve of Basin E, over the Course of the 90th Percentile Month with Saturated Basin Media and Infiltration Rate of 0.75 In/hr

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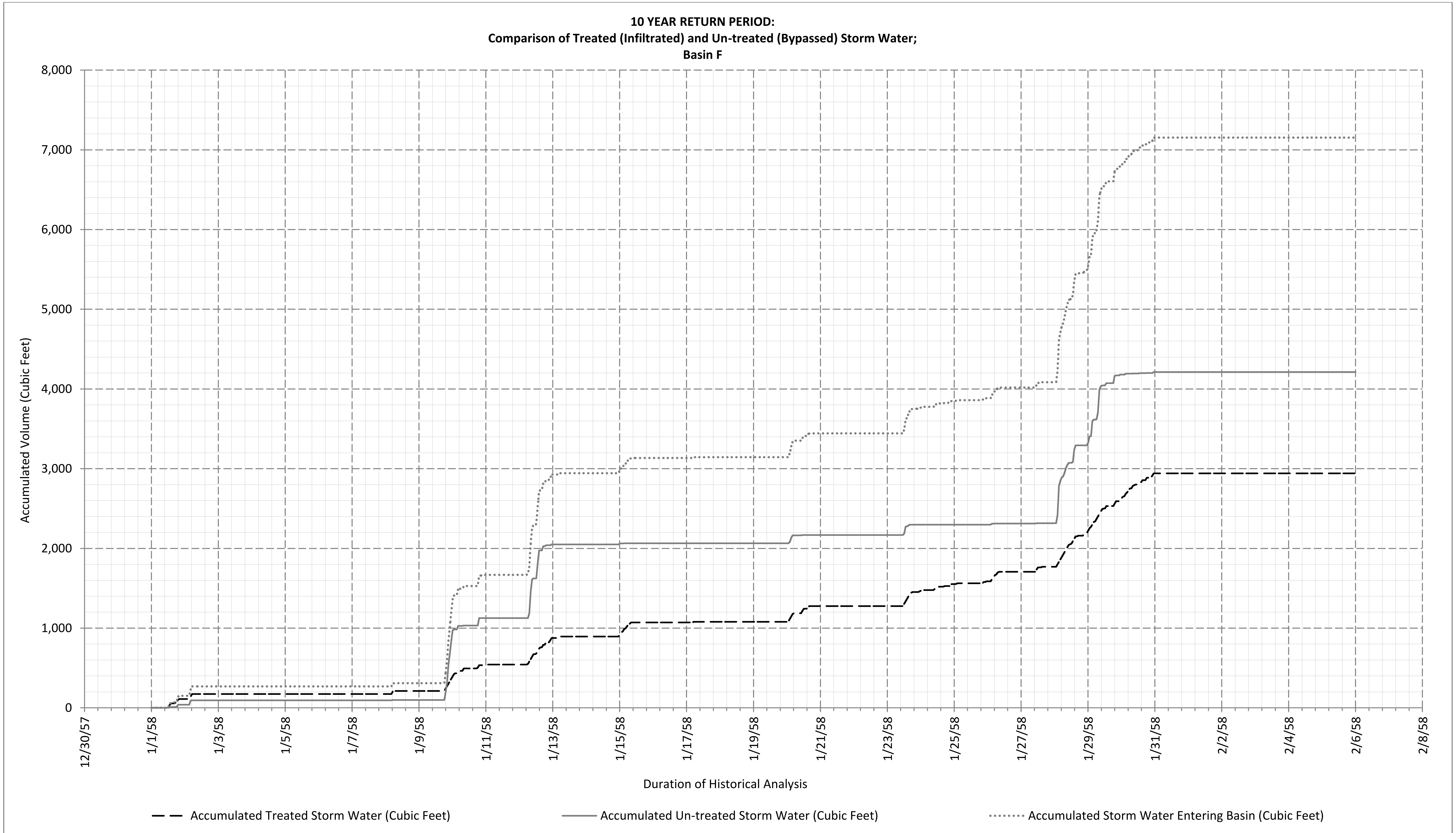

Figure 24: Hydrographs and Performance Curve of Basin F, over the Course of the 90th Percentile Month with Saturated Basin Media and Infiltration Rate of 0.75 In/hr

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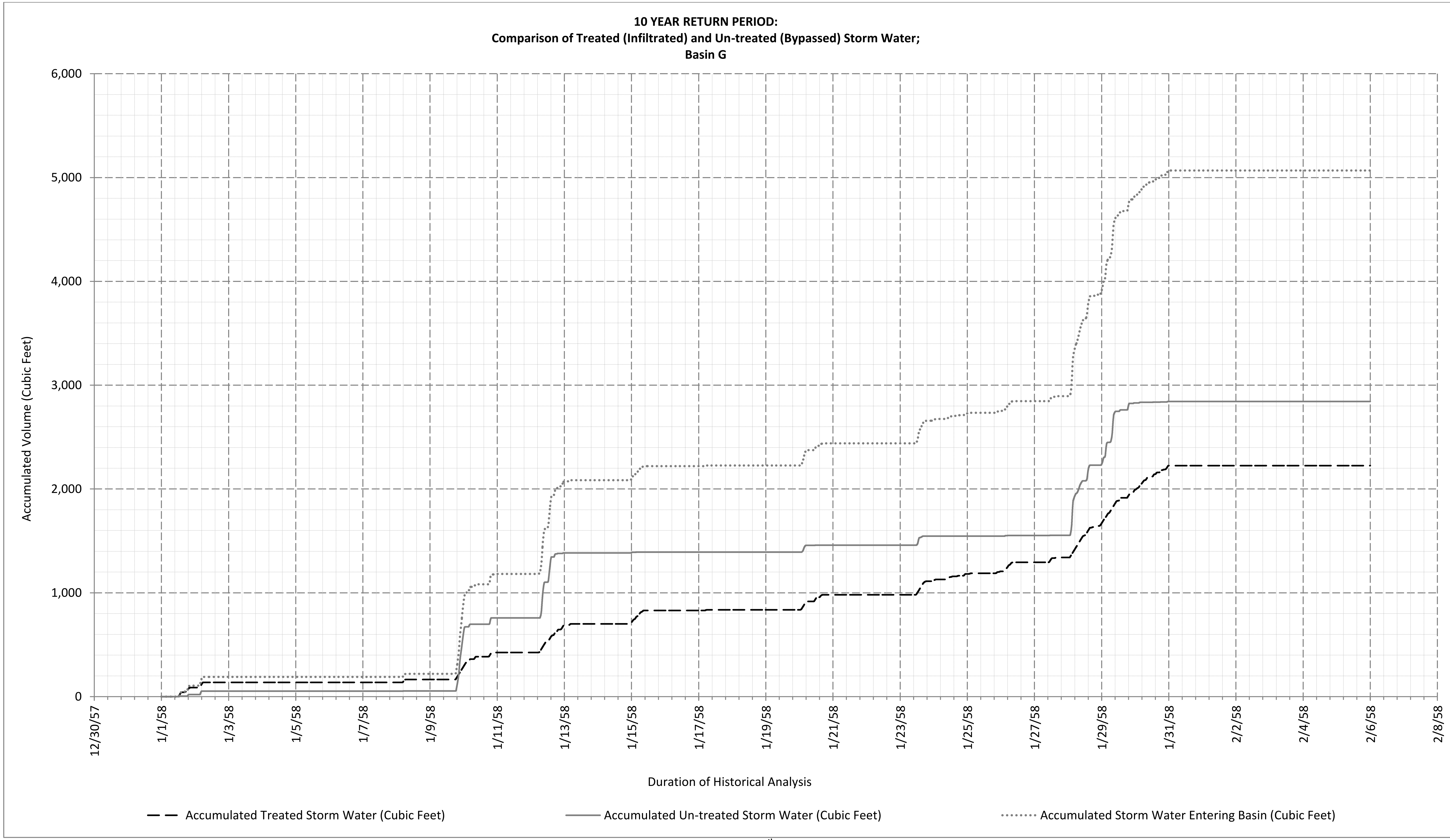

Figure 25: Hydrographs and Performance Curve of Basin G, over the Course of the 90th Percentile Month with Saturated Basin Media and Infiltration Rate of 0.75 In/hr

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8. Appendix of Tables

Table 1: Hourly Design Values for Precipitation Intensities According to Probability Thresholds

DESIGN PERCENTILE	RETURN PERIOD (yrs.)	CORRESPONDING HOURLY PRECIPITATION (in.)	DATE OF OCCURRENCE
90%	10	0.12	11/27/1964
96%	25	0.19	4/3/2003
98%	50	0.24	4/6/1982
99%	100	0.30	3/20/2005

Table 2: Daily Design Values for Precipitation Intensities According to Probability Thresholds

DESIGN PERCENTILE	RETURN PERIOD (yrs.)	CORRESPONDING DAILY PRECIPITATION (in.)	DATE OF OCCURRENCE
85%	6.66	0.68	12/13/1993
90%	10	0.86	9/22/1983
96%	25	1.28	2/11/1951
98%	50	1.62	10/10/1984
99%	100	2.03	1/15/1974

Table 3: Monthly Design Values for Precipitation Intensities According to Probability Thresholds

DESIGN PERCENTILE	RETURN PERIOD (yrs.)	CORRESPONDING MONTHLY PRECIPITATION (in.)	DATE OF OCCURRENCE
90%	10	8.57	1/1/1958
96%	25	11.65	12/1/1955
98%	50	13.36	2/1/1998
99%	100	14.38	10/1/1950

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Table 4: Basins A and C: Critical Design Information for Determining the C Coefficient for all Contributing Watersheds Created by Implementation of the HBPP FSR Plan

RECEIVING BASIN	WATERSHED ID#	WATERSHED & POINT OF CONCENTRATION NAME	IMPERVIOUS COVER (C=0.95) (ACRES)	PERCENT EXPANSION FROM PROPOSED EXTENTS OF IMPERVIOUS COVER	GRASS COVER (C=0.30) (ACRES)	GRAVEL COVER (C=0.80) (ACRES)	TOTAL WATERSHED AREA (ACRES)	RESULTING COMPOSITE C COEFFECIENTS
A	1	A2/A1/A0	0.18	100%	0.88	0.23	1.29	0.48
	2	A2/A1/B0	0.30	110%	0.49	0.02	0.82	0.56
	3	A1/A0	0.25	110%	0.04	-	0.28	0.87
	3C	A1/A0 COMP	0.74	NA	1.41	0.25	2.40	0.55
	4	A0	0.28	110%	0.57	0.01	0.86	0.52
	4C	A0 COMP	1.02	NA	1.98	0.26	3.25	0.54
C	31	C4/B3/B2	0.31	125%	0.79	0.45	1.55	0.57
	32	C3/B2/B1	0.17	110%	0.00	-	0.17	0.95
	32C	C3/B2/B1 COMP	0.48	NA	0.79	0.45	1.72	0.61
	33	C2/A1/A0	0.07	110%	0.00	-	0.07	0.95
	34	C2/A1/B0	0.14	110%	0.00	-	0.14	0.95
	35	C2/B1/A0	0.76	110%	0.10	0.01	0.87	0.87
	36	C2/B1/B0	0.56	100%	-	-	0.56	0.95
	36C	C2/B1/B0 COMP	1.05	NA	0.79	0.45	2.29	0.70
	36.5C	36C+38 COMP	1.87	NA	0.89	0.56	3.32	0.75
	37C	C1/A0 COMP	0.21	NA	0.00	-	0.21	0.95
	38	C1/B0	0.06	110%	0.00	0.10	0.16	0.85
	38C	C1/BO COMP	0.82	NA	0.10	0.11	1.03	0.87
	39	C0	0.70	125%	0.95	0.05	1.71	0.58
	39C	CO COMP	2.78	NA	1.85	0.61	5.24	0.70

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Table 5: Basin B: Critical Design Information for Determining the C Coefficient for all Contributing Watersheds Created by Implementation of the HBPP FSR Plan

RECEIVING BASIN	WATERSHED ID#	WATERSHED & POINT OF CONCENTRATION NAME	IMPERVIOUS COVER (C=0.95) (ACRES)	PERCENT EXPANSION FROM PROPOSED EXTENTS OF IMPERVIOUS COVER	GRASS COVER (C=0.30) (ACRES)	GRAVEL COVER (C=0.80) (ACRES)	TOTAL WATERSHED AREA (ACRES)	RESULTING COMPOSITE C COEFFECIENTS
B	5	B7/D6/A5/A4/C3/B2/B1	0.11	100%	0.00	0.09	0.20	0.88
	6	B6/D5/A4/A3/C2/B1/B0	0.30	125%	0.00	0.08	0.38	0.92
	6C	B6/D5/A4/A3/C2/B1/B0 COMP	0.41	NA	0.00	0.17	0.58	0.91
	7	B6/D5/A4/A3/C2/B1/A0	0.07	100%	-	0.03	0.10	0.91
	8	B5/D4/A3/A2/A1	0.03	110%	0.23	-	0.25	0.37
	9	B5/D4/A3/A2/C1/A0	0.06	100%	-	0.03	0.09	0.90
	10	B5/D4/A3/A2/C1/B0	-	NA	0.08	0.02	0.10	0.39
	10C	B5/D4/A3/A2/C1/B0 COMP	0.49	NA	0.08	0.21	0.77	0.84
	11	B4/C3/B1/A0	0.45	225%	0.04	-	0.49	0.90
	12	B4/C3/B1/B0	0.16	100%	-	-	0.16	0.95
	13	B4/D3/A2/A1/A0	0.08	100%	-	-	0.08	0.95
	13C	B4/D3/A2/A1/A0 COMP	0.11	NA	0.23	-	0.34	0.52
	14	B4/D3/A2/A1/B0	0.02	100%	-	0.01	0.02	0.91
	15	B4/D3/A2/A1/C0	-	NA	0.04	0.02	0.06	0.45
	15C	B4/D3/A2/A1/C0 COMP	0.54	NA	0.12	0.26	0.92	0.82
	16	B3/C2/B0	0.08	100%	-	-	0.08	0.95
	16C	B3/C2/B0 COMP	0.69	NA	0.04	-	0.72	0.92
	17	B3/C2/A0	0.03	100%	-	-	0.03	0.95
	18	B2/A1	0.13	110%	0.14	0.01	0.28	0.62
	19	B3/D2/A1/A0	0.12	125%	0.09	0.08	0.29	0.72
	19C	B3/D2/A1/A0 COMP	0.80	NA	0.43	0.35	1.57	0.74
	20	B1/A0	0.26	100%	-	0.08	0.34	0.91
	20C	B1/A0 COMP	0.38	NA	0.14	0.10	0.62	0.78
	21	B2/B1	-	NA	1.44	0.06	1.50	0.32
	22	B2/C1	0.03	110%	0.14	0.01	0.18	0.44
	22C	B2/C1 COMP	0.75	NA	0.18	0.01	0.94	0.83
	23	B2/D1/B0	0.68	110%	0.12	-	0.79	0.85
	24	B2/D1/A0	0.79	110%	0.11	-	0.90	0.87
	26	B1/B0	0.14	110%	0.24	-	0.37	0.54
	26C	B1/B0 COMP	0.95	NA	1.98	0.08	3.01	0.52
	27	B1/C0	0.07	110%	0.12	0.01	0.20	0.55
	27C	B1/C0 COMP	0.82	NA	0.30	0.02	1.13	0.78
	28C	B1/D0 COMP	2.26	NA	0.65	0.35	3.26	0.80
	30	B0	-	NA	0.83	-	0.83	0.30
	30C	B0 COMP	3.60	NA	3.60	0.52	7.72	0.64

Table 6: Basins D, E, F, and G: Critical Design Information for Determining the C Coefficient for all Contributing Watersheds Created by Implementation of the HBPP FSR Plan

RECEIVING BASIN	WATERSHED ID#	WATERSHED & POINT OF CONCENTRATION NAME	IMPERVIOUS COVER (C=0.95) (ACRES)	PERCENT EXPANSION FROM PROPOSED EXTENTS OF IMPERVIOUS COVER	GRASS COVER (C=0.30) (ACRES)	GRAVEL COVER (C=0.80) (ACRES)	TOTAL WATERSHED AREA (ACRES)	RESULTING COMPOSITE C COEFFECIENTS
D	40	D1/A0	0.12	110%	0.00	-	0.12	0.95
	41	D1/B0	0.13	110%	0.00	-	0.13	0.95
	42	D0	-	0%	0.02	-	0.02	0.30
	42C	DO COMP	0.24	0%	0.02	-	0.27	0.90
E	43	E1/A0	0.08	110%	0.00	-	0.08	0.95
	44	E1/B0	0.05	110%	0.00	-	0.05	0.95
	45	E0	-	0%	0.02	-	0.02	0.30
	45C	EO COMP	0.14	0%	0.02	-	0.16	0.86
F	46	F1/A0	0.13	110%	0.00	-	0.13	0.95
	47	F1/B0	0.10	110%	0.00	-	0.10	0.95
	48	F0	-	0%	0.02	-	0.02	0.30
	48C	F0 COMP	0.23	0%	0.02	-	0.26	0.89
G	49	G1/A0	0.11	110%	0.00	-	0.11	0.95
	50	G1/B0	0.06	110%	0.00	-	0.06	0.95
	51	G0	-	0%	0.01	-	0.01	0.30
	51C	G0 COMP	0.17	0%	0.01	-	0.18	0.91

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Table 7: Basins A and C: Critical Design Information for Determining the Time of Concentration (TOC) for all Contributing Watersheds Created by Implementation of the HBPP FSR Plan

RECEIVING BASIN	WATERSHED ID#	WATERSHED & POINT OF CONCENTRATION NAME	OVERLAND SHEET FLOW				SHALLOW CONCENTRATED FLOW				SWALE FLOW				PIPE FLOW				INTIAL ABSTRACTION TIME (MIN)	TIME OF CONCENTRATION USED IN PEAK FLOW CALCULATION (MIN)	LAG TIME USED IN MODEL TO DETERMINE VARIABLE BASIN VOLUME (MIN)
			ROUTED DISTANCE (FT)	AVERAGE SLOPE	COVER TYPE	TIME OF CONCENTRATION (MIN)	ROUTED DISTANCE (FT)	AVERAGE SLOPE	IS SURFACE PAVED? (Y/N)	TIME OF CONCENTRATION (MIN)	ROUTED DISTANCE (FT)	AVERAGE SLOPE	CHANNEL TYPE	TIME OF CONCENTRATION (MIN)	ROUTED DISTANCE (FT)	AVERAGE SLOPE	PIPE TYPE	TIME OF CONCENTRATION (MIN)			
A	1	A2/A1/A0	295	5.08%	GRSS	23.6	83	21.69%	N	0.2	225	5.33%	SWL_A	1.3	80	2.00%	PIP_D	0.2	5	30.3	30.0
	2	A2/A1/B0	310	9.68%	GRSS	19.0	NA	NA	NA	-	208	1.44%	SWL_A	2.0	92	2.00%	PIP_C	0.3	5	26.3	26.0
	3	A1/A0	254	1.57%	IMPRV	2.8	NA	NA	NA	-	-	0.00%	NA	-	-	0.00%	NA	-	5	7.8	7.0
	3C	A1/A0 COMP	NA	NA	NA	-	NA	NA	NA	-	-	0.00%	NA	-	112	2.00%	PIP_D	0.3	-	30.6	30.0
	4	A0	265	4.91%	GRSS	22.0	NA	NA	NA	-	-	0.00%	NA	-	-	0.00%	NA	-	-	22.0	22.0
	4C	A0 COMP	NA	NA	NA	-	NA	NA	NA	-	-	0.00%	NA	-	-	0.00%	NA	-	-	30.3	30.0
C	31	C4/B3/B2	300	1.77%	GRSS	36.6	72	17.44%	N	0.2	-	0.00%	NA	-	-	0.00%	NA	-	5	41.8	67.0
	32	C3/B2/B1	93	0.54%	IMPRV	2.0	NA	NA	NA	-	88	1.59%	SWL_B	0.5	-	0.00%	NA	-	5	7.4	33.0
	32C	C3/B2/B1 COMP	NA	NA	NA	-	NA	NA	NA	-	-	0.00%	NA	-	155	2.00%	PIP_C	0.4	-	42.2	42.0
	33	C2/A1/A0	22	2.00%	IMPRV	0.4	NA	NA	NA	-	123	0.25%	SWL_K	1.4	-	0.00%	NA	-	5	6.7	7.0
	34	C2/A1/B0	22	2.00%	IMPRV	0.4	NA	NA	NA	-	249	1.50%	SWL_K	1.2	-	0.00%	NA	-	5	6.6	6.0
	35	C2/B1/A0	49	2.04%	IMPRV	0.7	NA	NA	NA	-	229	0.26%	SWL_G	1.3	-	0.00%	NA	-	5	7.0	33.0
	36	C2/B1/B0	165	2.41%	IMPRV	1.7	NA	NA	NA	-	-	0.00%	NA	-	-	0.00%	NA	-	5	6.7	32.0
	36C	C2/B1/B0 COMP	NA	NA	NA	-	NA	NA	NA	-	-	0.00%	NA	-	80	2.00%	PIP_D	0.2	-	42.4	42.0
	36.5C	36C+38 COMP	NA	NA	NA	-	NA	NA	NA	-	186	0.25%	SWL_O	25.1	40	0.27%	PIP_D	0.2	-	67.6	67.0
	37C	C1/A0 COMP	NA	NA	NA	-	NA	NA	NA	-	45	0.27%	SWL_F	0.4	-	0.00%	NA	-	-	7.1	7.0
	38	C1/B0	75	0.27%	GRVL	6.1	NA	NA	NA	-	-	0.00%	NA	-	-	0.00%	NA	-	-	6.1	32.0
	38C	C1/BO COMP	NA	NA	NA	-	NA	NA	NA	-	142	0.28%	SWL_L	0.9	-	0.00%	NA	-	-	7.9	7.0
	39	C0	300	4.57%	GRSS	25.0	116	0.26%	N	2.4	-	0.00%	NA	-	-	0.00%	NA	-	-	27.4	27.0
	39C	CO COMP	NA	NA	NA	-	NA	NA	NA	-	-	0.00%	NA	-	-	0.00%	NA	-	-	67.6	67.0

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Table 8: Basin B: Critical Design Information for Determining the Time of Concentration (TOC) for all Contributing Watersheds Created by Implementation of the HBPP FSR Plan

RECEIVING BASIN	WATERSHED ID#	WATERSHED & POINT OF CONCENTRATION NAME	OVERLAND SHEET FLOW				SHALLOW CONCENTRATED FLOW				SWALE FLOW				PIPE FLOW				INTIAL ABSTRACTION TIME (MIN)	TIME OF CONCENTRATION USED IN PEAK FLOW CALCULATION (MIN)	LAG TIME USED IN MODEL TO DETERMINE VARIABLE BASIN VOLUME (MIN)
			ROUTED DISTANCE (FT)	AVERAGE SLOPE	COVER TYPE	TIME OF CONCENTRATION (MIN)	ROUTED DISTANCE (FT)	AVERAGE SLOPE	IS SURFACE PAVED? (Y/N)	TIME OF CONCENTRATION (MIN)	ROUTED DISTANCE (FT)	AVERAGE SLOPE	CHANNEL TYPE	TIME OF CONCENTRATION (MIN)	ROUTED DISTANCE (FT)	AVERAGE SLOPE	PIPE TYPE	TIME OF CONCENTRATION (MIN)			
B	5	B7/D6/A5/A4/C3/B2/B1	106	2.08%	IMPRV	1.3	NA	NA	NA	-	-	0.00%	NA	-	-	0.00%	NA	-	5	6.3	11.0
	6	B6/D5/A4/A3/C2/B1/B0	60	2.00%	IMPRV	0.8	NA	NA	NA	-	-	0.00%	NA	-	-	0.00%	NA	-	5	5.8	10.0
	6C	B6/D5/A4/A3/C2/B1/B0 COMP	NA	NA	NA	-	NA	NA	NA	-	212	0.25%	SWL_G	1.4	-	0.00%	NA	-	-	8.6	8.0
	7	B6/D5/A4/A3/C2/B1/A0	NA	NA	NA	-	NA	NA	NA	-	139	0.25%	SWL_N	1.1	-	0.00%	NA	-	5	6.1	10.0
	8	B5/D4/A3/A2/A1	207	7.20%	GRSS	15.5	NA	NA	NA	-	-	0.00%	NA	-	-	0.00%	NA	-	5	20.5	23.0
	9	B5/D4/A3/A2/C1/A0	NA	NA	NA	-	NA	NA	NA	-	121	0.26%	SWL_N	1.0	-	0.00%	NA	-	5	6.0	8.0
	10	B5/D4/A3/A2/C1/B0	50	0.26%	IMPRV	1.6	NA	NA	NA	-	-	0.00%	NA	-	-	0.00%	NA	-	5	6.6	9.0
	10C	B5/D4/A3/A2/C1/B0 COMP	NA	NA	NA	-	NA	NA	NA	-	97	0.26%	SWL_G	0.6	-	0.00%	NA	-	-	8.3	8.0
	11	B4/C3/B1/A0	75	2.00%	IMPRV	1.0	NA	NA	NA	-	400	2.01%	SWL_F	1.3	42	2.00%	PIP_C	0.1	5	7.4	8.0
	12	B4/C3/B1/B0	265	2.69%	IMPRV	2.4	NA	NA	NA	-	-	0.00%	NA	-	130	2.00%	PIP_C	0.4	5	7.8	9.0
	13	B4/D3/A2/A1/A0	27	1.70%	IMPRV	0.5	NA	NA	NA	-	-	0.00%	NA	-	-	0.00%	NA	-	5	5.5	7.0
	13C	B4/D3/A2/A1/A0 COMP	NA	NA	NA	-	NA	NA	NA	-	87	10.68%	SWL_B	0.3	-	0.00%	NA	-	-	20.8	20.0
	14	B4/D3/A2/A1/B0	NA	NA	NA	-	NA	NA	NA	-	70	0.26%	SWL_A	2.2	-	0.00%	NA	-	5	7.2	9.0
	15	B4/D3/A2/A1/C0	23	0.26%	IMPRV	0.9	NA	NA	NA	-	-	0.00%	NA	-	-	0.00%	NA	-	5	5.9	8.0
	15C	B4/D3/A2/A1/C0 COMP	NA	NA	NA	-	NA	NA	NA	-	82	0.26%	SWL_G	0.5	-	0.00%	NA	-	-	8.3	8.0
	16	B3/C2/B0	34	1.18%	IMPRV	0.6	NA	NA	NA	-	86	2.50%	SWL_B	0.5	-	0.00%	NA	-	5	6.1	7.0
	16C	B3/C2/B0 COMP	NA	NA	NA	-	NA	NA	NA	-	-	0.00%	NA	-	127	2.00%	PIP_C	0.3	-	8.1	8.0
	17	B3/C2/A0	16	1.81%	IMPRV	0.3	NA	NA	NA	-	91	2.23%	SWL_B	0.6	43	2.00%	PIP_C	0.2	5	6.0	7.0
	18	B2/A1	260	3.89%	IMPRV	2.0	NA	NA	NA	-	101	0.30%	SWL_F	0.8	-	0.00%	NA	-	5	7.9	9.0
	19	B3/D2/A1/A0	75	1.33%	IMPRV	1.1	NA	NA	NA	-	-	0.00%	NA	-	-	0.00%	NA	-	5	6.1	8.0
	19C	B3/D2/A1/A0 COMP	NA	NA	NA	-	NA	NA	NA	-	156	0.26%	SWL_G	0.9	-	0.00%	NA	-	-	21.7	21.0
	20	B1/A0	135	4.36%	IMPRV	1.1	NA	NA	NA	-	-	0.00%	NA	-	-	0.00%	NA	-	5	6.1	7.0
	20C	B1/A0 COMP	NA	NA	NA	-	NA	NA	NA	-	168	0.30%	SWL_L	1.2	82	0.61%	PIP_C	0.3	-	9.4	9.0
	21	B2/B1	133	9.92%	GRSS	9.6	NA	NA	NA	-	426	3.29%	SWL_A	2.9	-	0.00%	NA	-	5	17.5	18.0
	22	B2/C1	9	1.67%	IMPRV	0.2	NA	NA	NA	-	124	2.15%	SWL_B	0.7	-	0.00%	NA	-	5	5.8	7.0
	22C	B2/C1 COMP	NA	NA	NA	-	NA	NA	NA	-	-	0.00%	NA	-	95	2.00%	PIP_D	0.2	-	8.3	8.0
	23	B2/D1/B0	300	2.00%	GRVL	8.3	65	2.00%	Y	0.4	265	2.00%	SWL_C	3.8	153	0.25%	PIP_A	0.9	5	18.3	19.0
	0	BLANK	NA	NA	NA	-	NA	NA	NA	-	-	0.00%	NA	-	-	0.00%	NA	-	-	-	-
	24	B2/D1/A0	300	2.00%	GRVL	8.3	53	1.98%	Y	0.3	265	2.00%	SWL_C	3.6	-	0.00%	NA	-	5	17.2	18.0
	0	BLANK	NA	NA	NA	-	NA	NA	NA	-	-	0.00%	NA	-	200	0.25%	PIP_A	5.0	-	5.0	5.0
	0	BLANK	116	3.97%	IMPRV	1.1	NA	NA	NA	-	-	0.00%	NA	-	-	0.00%	NA	-	5	6.1	6.0
	0	BLANK	NA	NA	NA	-	NA	NA	NA	-	-	0.00%	NA	-	35	2.00%	PIP_A	0.4	-	0.4	-
	26	B1/B0	53	2.51%	IMPRV	0.7	NA	NA	NA	-	146	5.08%	SWL_B	0.5	-	0.00%	NA	-	5	6.1	6.0
	26C	B1/B0 COMP	NA	NA	NA	-	NA	NA	NA	-	-	0.00%	NA	-	206	2.00%	PIP_A	0.4	-	18.6	18.0
	27	B1/C0	31	1.29%	IMPRV	0.6	NA	NA	NA	-	90	3.67%	SWL_B	0.4	-	0.00%	NA	-	5	5.9	6.0
	27C	B1/C0 COMP	NA	NA	NA	-	NA	NA	NA	-	-	0.00%	NA	-	287	2.00%	PIP_E	0.6	-	8.9	8.0
	0	BLANK	300	2.00%	GRVL	8.3	65	2.00%	Y	0.4	265	2.00%	SWL_C	13.2	-	0.00%	NA	-	-	21.8	21.0
	28C	B1/D0 COMP	NA	NA	NA	-	NA	NA	NA	-	-	0.00%	NA	-	350	0.34%	PIP_A	1.2	-	22.9	22.0
	0	BLANK	300	2.00%	GRVL	8.3	150	2.00%	Y	0.9	-	0.00%	NA	-	-	0.00%	NA	-	5	14.2	14.0
	30	B0	300	1.94%	GRSS	35.2	608	4.14%	N	3.1	-	0.00%	NA	-	-	0.00%	NA	-	-	38.3	38.0
	30C	B0 COMP	NA	NA	NA	-	NA	NA	NA	-	-	0.00%	NA	-	-	0.00%	NA	-	-	43.4	43.0

Table 9: Basins D, E, F, and G: Critical Design Information for Determining the Time of Concentration (TOC) for all Contributing Watersheds Created by Implementation of the HBPP FSR Plan

RECEIVING BASIN	WATERSHED ID#	WATERSHED & POINT OF CONCENTRATION NAME	OVERLAND SHEET FLOW				SHALLOW CONCENTRATED FLOW				SWALE FLOW				PIPE FLOW				INTIAL ABSTRACTION TIME (MIN)	TIME OF CONCENTRATION USED IN PEAK FLOW CALCULATION (MIN)	LAG TIME USED IN MODEL TO DETERMINE VARIABLE BASIN VOLUME (MIN)
			ROUTED DISTANCE (FT)	AVERAGE SLOPE	COVER TYPE	TIME OF CONCENTRATION (MIN)	ROUTED DISTANCE (FT)	AVERAGE SLOPE	IS SURFACE PAVED? (Y/N)	TIME OF CONCENTRATION (MIN)	ROUTED DISTANCE (FT)	AVERAGE SLOPE	CHANNEL TYPE	TIME OF CONCENTRATION (MIN)	ROUTED DISTANCE (FT)	AVERAGE SLOPE	PIPE TYPE	TIME OF CONCENTRATION (MIN)			
D	40	D1/A0	20	2.00%	IMPRV	0.3	NA	NA	NA	-	227	0.51%	SWL_I	29.9	-	0.00%	NA	-	5	35.2	41.0
	41	D1/B0	20	2.00%	IMPRV	0.3	NA	NA	NA	-	260	0.50%	SWL_I	34.3	-	0.00%	NA	-	5	39.6	46.0
	42	D0	18	22.22%	GRSS	1.4	NA	NA	NA	-	-	0.00%	NA	-	-	0.00%	NA	-	5	6.4	6.0
	42C	DO COMP	NA	NA	NA	-	NA	NA	NA	-	-	0.00%	NA	-	-	0.00%	NA	-	-	46.0	46.0
E	43	E1/A0	20	2.00%	IMPRV	0.3	NA	NA	NA	-	161	0.25%	SWL_M	25.5	-	0.00%	NA	-	5	30.8	40.0
	44	E1/B0	20	2.00%	IMPRV	0.3	NA	NA	NA	-	100	0.26%	SWL_I	19.8	-	0.00%	NA	-	5	25.1	34.0
	45	E0	46	8.70%	GRSS	4.3	NA	NA	NA	-	-	0.00%	NA	-	-	0.00%	NA	-	5	9.3	9.0
	45C	EO COMP	NA	NA	NA	-	NA	NA	NA	-	-	0.00%	NA	-	-	0.00%	NA	-	-	34.8	34.0
F	46	F1/A0	20	2.00%	IMPRV	0.3	NA	NA	NA	-	242	0.76%	SWL_K	1.6	-	0.00%	NA	-	5	6.9	11.0
	47	F1/B0	42	2.14%	IMPRV	0.6	NA	NA	NA	-	139	0.76%	SWL_K	1.0	-	0.00%	NA	-	5	6.6	11.0
	48	F0	50	8.00%	GRSS	4.8	NA	NA	NA	-	NA	0.00%	NA	-	-	0.00%	NA	-	-	4.8	4.0
	48C	F0 COMP	NA	NA	NA	-	NA	NA	NA	-	NA	0.00%	NA	-	-	0.00%	NA	-	-	11.7	11.0
G	49	G1/A0	20	2.00%	IMPRV	0.3	NA	NA	NA	-	189	0.51%	SWL_I	24.7	-	0.00%	NA	-	5	30.1	37.0
	50	G1/B0	75	0.21%	IMPRV	2.4	NA	NA	NA	-	-	0.00%	NA	-	-	0.00%	NA	-	5	7.4	14.0
	51	G0	24	16.67%	GRSS	2.0	NA	NA	NA	-	-	0.00%	NA	-	-	0.00%	NA	-	5	7.0	7.0
	51C	G0 COMP	NA	NA	NA	-	NA	NA	NA	-	-	0.00%	NA	-	-	0.00%	NA	-	-	37.1	37.0

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Table 10: Basins and C: Predicted Peak Flows (Qp) for all Contributing Watersheds and Design Storms

				90th PERCENTILE DESIGN STORM (10 YEAR RETURN PERIOD)				96th PERCENTILE DESIGN STORM (25 YEAR RETURN PERIOD)				98th PERCENTILE DESIGN STORM (50 YEAR RETURN PERIOD)				99th PERCENTILE DESIGN STORM (100 YEAR RETURN PERIOD)			
RECEIVING BASIN	WATERSHED ID#	WATERSHED & POINT OF CONCENTRATION NAME	ARE FEATURES EXISTING <E> OR PROPOSED <P> ?	PERCENT OF AVAILABLE SWALE DEPTH OCCUPIED BY PEAK FLOW	PERCENT OF AVAILABLE PIPE DEPTH OCCUPIED BY PEAK FLOW	INTENSITY OF PRECIPITATION USED FOR CALCULATION (IN/HR)	ANTICIPATED PEAK FLOW (CFS)	PERCENT OF AVAILABLE SWALE DEPTH OCCUPIED BY PEAK FLOW	PERCENT OF AVAILABLE PIPE DEPTH OCCUPIED BY PEAK FLOW	INTENSITY OF PRECIPITATION USED FOR CALCULATION (IN/HR)	ANTICIPATED PEAK FLOW (CFS)	PERCENT OF AVAILABLE SWALE DEPTH OCCUPIED BY PEAK FLOW	PERCENT OF AVAILABLE PIPE DEPTH OCCUPIED BY PEAK FLOW	INTENSITY OF PRECIPITATION USED FOR CALCULATION (IN/HR)	ANTICIPATED PEAK FLOW (CFS)	PERCENT OF AVAILABLE SWALE DEPTH OCCUPIED BY PEAK FLOW	PERCENT OF AVAILABLE PIPE DEPTH OCCUPIED BY PEAK FLOW	INTENSITY OF PRECIPITATION USED FOR CALCULATION (IN/HR)	ANTICIPATED PEAK FLOW (CFS)
A	1	A2/A1/A0	<E>	88%	21%	1.66	1.03	91%	22%	2.00	1.24	93%	22%	2.23	1.38	99%	24%	2.54	1.58
	2	A2/A1/B0	<E>	107%	25%	1.79	0.82	111%	27%	2.16	0.99	119%	29%	2.42	1.11	119%	29%	2.76	1.26
	3	A1/A0	<E>	0%	0%	3.45	0.85	0%	0%	4.22	1.04	0%	0%	4.71	1.17	0%	0%	5.44	1.35
	3C	A1/A0 COMP	<E>	0%	29%	1.65	2.19	0%	32%	1.99	2.63	0%	34%	2.22	2.95	0%	36%	2.53	3.36
	4	A0	<E>	0%	0%	1.97	0.87	0%	0%	2.38	1.06	0%	0%	2.66	1.18	0%	0%	3.04	1.35
	4C	A0 COMP	<E>	0%	0%	1.66	2.94	0%	0%	2.00	3.53	0%	0%	2.23	3.95	0%	0%	2.54	4.50
C	31	C4/B3/B2	<E>	0%	0%	1.40	1.24	0%	0%	1.67	1.49	0%	0%	1.87	1.66	0%	0%	2.12	1.89
	32	C3/B2/B1	<E>	125%	0%	3.55	0.59	139%	0%	4.35	0.72	145%	0%	4.85	0.80	151%	0%	5.60	0.93
	32C	C3/B2/B1 COMP	<E>	0%	32%	1.39	1.47	0%	35%	1.66	1.76	0%	37%	1.86	1.96	0%	40%	2.11	2.23
	33	C2/A1/A0	<P>	99%	0%	3.74	0.25	110%	0%	4.59	0.30	110%	0%	5.13	0.34	110%	0%	5.92	0.39
	34	C2/A1/B0	<P>	91%	0%	3.77	0.51	97%	0%	4.67	0.63	102%	0%	5.21	0.71	106%	0%	6.02	0.81
	35	C2/B1/A0	<P>	84%	0%	3.66	2.78	92%	0%	4.52	3.43	96%	0%	5.04	3.83	101%	0%	5.82	4.43
	36	C2/B1/B0	<E>	0%	0%	3.74	2.00	0%	0%	4.59	2.46	0%	0%	5.13	2.74	0%	0%	5.92	3.17
	36C	C2/B1/B0 COMP	<E>	0%	29%	1.39	2.21	0%	32%	1.66	2.64	0%	34%	1.86	2.96	0%	36%	2.11	3.35
	36.5C	36C+38 COMP	<P>	67%	57%	1.08	2.68	72%	65%	1.30	3.22	75%	71%	1.46	3.63	79%	80%	1.66	4.12
	37C	C1/A0 COMP	<P>	96%	0%	3.63	0.73	100%	0%	4.45	0.90	108%	0%	5.00	1.01	112%	0%	5.78	1.16
	38	C1/B0	<P>	0%	0%	3.94	0.53	0%	0%	4.84	0.65	0%	0%	5.40	0.73	0%	0%	6.24	0.84
	38C	C1/BO COMP	<P>	93%	0%	3.43	3.07	101%	0%	4.25	3.81	105%	0%	4.75	4.25	112%	0%	5.48	4.90
	39	C0	<P>	0%	0%	1.75	1.75	0%	0%	2.11	2.10	0%	0%	2.36	2.35	0%	0%	2.69	2.68
	39C	CO COMP	<P>	0%	0%	1.08	3.98	0%	0%	1.30	4.77	0%	0%	1.46	5.37	0%	0%	1.66	6.10

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Table 11: Basin B: Predicted Peak Flows (Qp) for all Contributing Watersheds and Design Storms

				90th PERCENTILE DESIGN STORM (10 YEAR RETURN PERIOD)				96th PERCENTILE DESIGN STORM (25 YEAR RETURN PERIOD)				98th PERCENTILE DESIGN STORM (50 YEAR RETURN PERIOD)				99th PERCENTILE DESIGN STORM (100 YEAR RETURN PERIOD)			
RECEIVING BASIN	WATERSHED ID#	WATERSHED & POINT OF CONCENTRATION NAME	ARE FEATURES EXISTING <E> OR PROPOSED <P> ?	PERCENT OF AVAILABLE SWALE DEPTH OCCUPIED BY PEAK FLOW	PERCENT OF AVAILABLE PIPE DEPTH OCCUPIED BY PEAK FLOW	INTENSITY OF PRECIPITATION USED FOR CALCULATION (IN/HR)	ANTICIPATED PEAK FLOW (CFS)	PERCENT OF AVAILABLE SWALE DEPTH OCCUPIED BY PEAK FLOW	PERCENT OF AVAILABLE PIPE DEPTH OCCUPIED BY PEAK FLOW	INTENSITY OF PRECIPITATION USED FOR CALCULATION (IN/HR)	ANTICIPATED PEAK FLOW (CFS)	PERCENT OF AVAILABLE SWALE DEPTH OCCUPIED BY PEAK FLOW	PERCENT OF AVAILABLE PIPE DEPTH OCCUPIED BY PEAK FLOW	INTENSITY OF PRECIPITATION USED FOR CALCULATION (IN/HR)	ANTICIPATED PEAK FLOW (CFS)	PERCENT OF AVAILABLE SWALE DEPTH OCCUPIED BY PEAK FLOW	PERCENT OF AVAILABLE PIPE DEPTH OCCUPIED BY PEAK FLOW	INTENSITY OF PRECIPITATION USED FOR CALCULATION (IN/HR)	ANTICIPATED PEAK FLOW (CFS)
	5	B7/D6/A5/A4/C3/B2/B1	<P>	0%	0%	3.87	0.69	0%	0%	4.75	0.85	0%	0%	5.30	0.94	0%	0%	6.13	1.09
	6	B6/D5/A4/A3/C2/B1/B0	<P>	0%	0%	4.05	1.41	0%	0%	4.97	1.73	0%	0%	5.55	1.93	0%	0%	6.42	2.23
	6C	B6/D5/A4/A3/C2/B1/B0 COMP	<P>	71%	0%	3.27	1.72	77%	0%	4.05	2.13	79%	0%	4.53	2.38	84%	0%	5.25	2.76
	7	B6/D5/A4/A3/C2/B1/A0	<P>	76%	0%	3.94	0.35	82%	0%	4.84	0.44	82%	0%	5.40	0.49	88%	0%	6.30	0.57
	8	B5/D4/A3/A2/A1	<E>	0%	0%	2.05	0.19	0%	0%	2.48	0.24	0%	0%	2.77	0.26	0%	0%	3.16	0.30
	9	B5/D4/A3/A2/C1/A0	<P>	76%	0%	3.97	0.31	76%	0%	4.88	0.38	82%	0%	5.50	0.43	82%	0%	6.36	0.50
	10	B5/D4/A3/A2/C1/B0	<P>	0%	0%	3.77	0.14	0%	0%	4.63	0.17	0%	0%	5.17	0.19	0%	0%	5.97	0.23
	10C	B5/D4/A3/A2/C1/B0 COMP	<P>	77%	0%	3.34	2.18	83%	0%	4.08	2.67	87%	0%	4.59	3.00	92%	0%	5.32	3.48
	11	B4/C3/B1/A0	<P>	82%	33%	3.55	1.56	90%	37%	4.38	1.93	93%	39%	4.89	2.15	99%	42%	5.69	2.51
	12	B4/C3/B1/B0	<E>	0%	21%	3.45	0.52	0%	22%	4.22	0.63	0%	24%	4.71	0.70	0%	25%	5.48	0.82
	13	B4/D3/A2/A1/A0	<E>	0%	0%	4.16	0.33	0%	0%	5.12	0.41	0%	0%	5.71	0.46	0%	0%	6.61	0.53
	13C	B4/D3/A2/A1/A0 COMP	<E>	75%	0%	2.03	0.36	81%	0%	2.46	0.43	81%	0%	2.75	0.48	87%	0%	3.14	0.55
	14	B4/D3/A2/A1/B0	<P>	65%	0%	3.60	0.07	65%	0%	4.41	0.09	83%	0%	5.04	0.10	83%	0%	5.82	0.12
	15	B4/D3/A2/A1/C0	<P>	0%	0%	4.01	0.11	0%	0%	4.93	0.13	0%	0%	5.50	0.15	0%	0%	6.36	0.17
	15C	B4/D3/A2/A1/C0 COMP	<P>	82%	0%	3.34	2.53	89%	0%	4.14	3.13	92%	0%	4.62	3.50	98%	0%	5.32	4.03
	16	B3/C2/B0	<E>	88%	0%	3.94	0.29	98%	0%	4.84	0.36	98%	0%	5.40	0.40	106%	0%	6.30	0.46
	16C	B3/C2/B0 COMP	<E>	0%	40%	3.38	2.24	0%	43%	4.14	2.75	0%	46%	4.62	3.07	0%	50%	5.36	3.56
	17	B3/C2/A0	<E>	79%	12%	3.97	0.12	79%	12%	4.88	0.14	79%	12%	5.45	0.16	79%	12%	6.30	0.19
	18	B2/A1	<P>	86%	0%	3.43	0.58	96%	0%	4.22	0.72	100%	0%	4.71	0.80	104%	0%	5.44	0.93
	19	B3/D2/A1/A0	<P>	0%	0%	3.94	0.82	0%	0%	4.84	1.00	0%	0%	5.40	1.12	0%	0%	6.24	1.30
B	19C	B3/D2/A1/A0 COMP	<P>	79%	0%	1.99	2.31	84%	0%	2.41	2.80	89%	0%	2.69	3.12	93%	0%	3.07	3.57
	20	B1/A0	<P>	0%	0%	3.94	1.22	0%	0%	4.84	1.50	0%	0%	5.40	1.68	0%	0%	6.24	1.94
	20C	B1/A0 COMP	<P>	72%	48%	3.12	1.50	77%	53%	3.85	1.85	80%	57%	4.30	2.07	84%	62%	4.96	2.38
	21	B2/B1	<P>	96%	0%	2.23	1.07	99%	0%	2.71	1.30	105%	0%	3.04	1.46	110%	0%	3.49	1.67
	22	B2/C1	<E>	102%	0%	4.05	0.32	102%	0%	4.97	0.40	111%	0%	5.55	0.45	119%	0%	6.42	0.52
	22C	B2/C1 COMP	<E>	0%	31%	3.34	2.58	0%	35%	4.08	3.16	0%	37%	4.56	3.52	0%	41%	5.32	4.12
	23	B2/D1/B0	<P>	63%	21%	2.18	1.48	68%	23%	2.65	1.80	72%	25%	2.98	2.02	76%	27%	3.42	2.32
	0	BLANK	NA	0%	0%	0.00	0.00	0%	0%	0.00	0.00	0%	0%	0.00	0.00	0%	0%	0.00	0.00
	24	B2/D1/A0	<P>	68%	0%	2.25	1.76	73%	0%	2.75	2.15	77%	0%	3.08	2.41	81%	0%	3.54	2.77
	0	BLANK	<P>	0%	2%	4.38	0.00	0%	2%	5.40	0.00	0%	2%	6.02	0.00	0%	2%	6.98	0.00
	0	BLANK	<P>	0%	0%	3.94	0.00	0%	0%	4.84	0.00	0%	0%	5.40	0.00	0%	0%	6.24	0.00
	0	BLANK	<P>	0%	1%	17.05	0.00	0%	1%	21.76	0.00	0%	1%	24.22	0.00	0%	1%	28.78	0.00
	26	B1/B0	<E>	111%	0%	3.94	0.79	121%	0%	4.84	0.97	125%	0%	5.40	1.08	129%	0%	6.24	1.25
	26C	B1/B0 COMP	<E>	0%	19%	2.16	3.37	0%	21%	2.62	4.09	0%	22%	2.94	4.59	0%	24%	3.37	5.26
	27	B1/C0	<E>	100%	0%	4.01	0.43	107%	0%	4.93	0.53	107%	0%	5.50	0.59	113%	0%	6.36	0.69
	27C	B1/C0 COMP	<E>	0%	26%	3.21	2.83	0%	28%	3.95	3.48	0%	30%	4.41	3.89	0%	33%	5.11	4.51
	0	BLANK	NA	10%	0%	1.98	0.00	10%	0%	2.39	0.00	10%	0%	2.68	0.00	10%	0%	3.05	0.00
	28C	B1/D0 COMP	<P>	0%	41%	1.93	5.07	0%	45%	2.34	6.12	0%	48%	2.62	6.86	0%	52%	2.99	7.85
	0	BLANK	<P>	0%	0%	2.50	0.00	0%	0%	3.03	0.00	0%	0%	3.39	0.00	0%	0%	3.89	0.00
	30	B0	<P>	0%	0%	1.47	0.37	0%	0%	1.75	0.44	0%	0%	1.96	0.49	0%	0%	2.23	0.56
	30C	B0 COMP	<P>	0%	0%	1.37	6.73	0%	0%	1.64	8.07	0%	0%	1.84	9.02	0%	0%	2.08	10.24

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Table 12: Basins D, E, F, and G: Predicted Peak Flows (Qp) for all Contributing Watersheds and Design Storms

				90th PERCENTILE DESIGN STORM (10 YEAR RETURN PERIOD)				96th PERCENTILE DESIGN STORM (25 YEAR RETURN PERIOD)				98th PERCENTILE DESIGN STORM (50 YEAR RETURN PERIOD)				99th PERCENTILE DESIGN STORM (100 YEAR RETURN PERIOD)			
RECEIVING BASIN	WATERSHED ID#	WATERSHED & POINT OF CONCENTRATION NAME	ARE FEATURES EXISTING <E> OR PROPOSED <P> ?	PERCENT OF AVAILABLE SWALE DEPTH OCCUPIED BY PEAK FLOW	PERCENT OF AVAILABLE PIPE DEPTH OCCUPIED BY PEAK FLOW	INTENSITY OF PRECIPITATION USED FOR CALCULATION (IN/HR)	ANTICIPATED PEAK FLOW (CFS)	PERCENT OF AVAILABLE SWALE DEPTH OCCUPIED BY PEAK FLOW	PERCENT OF AVAILABLE PIPE DEPTH OCCUPIED BY PEAK FLOW	INTENSITY OF PRECIPITATION USED FOR CALCULATION (IN/HR)	ANTICIPATED PEAK FLOW (CFS)	PERCENT OF AVAILABLE SWALE DEPTH OCCUPIED BY PEAK FLOW	PERCENT OF AVAILABLE PIPE DEPTH OCCUPIED BY PEAK FLOW	INTENSITY OF PRECIPITATION USED FOR CALCULATION (IN/HR)	ANTICIPATED PEAK FLOW (CFS)	PERCENT OF AVAILABLE SWALE DEPTH OCCUPIED BY PEAK FLOW	PERCENT OF AVAILABLE PIPE DEPTH OCCUPIED BY PEAK FLOW	INTENSITY OF PRECIPITATION USED FOR CALCULATION (IN/HR)	ANTICIPATED PEAK FLOW (CFS)
D	40	D1/A0	<P>	98%	0%	1.53	0.17	113%	0%	1.92	0.21	113%	0%	2.15	0.24	113%	0%	2.45	0.27
	41	D1/B0	<P>	98%	0%	1.44	0.18	113%	0%	1.80	0.22	113%	0%	2.01	0.25	113%	0%	2.29	0.28
	42	D0	<P>	0%	0%	3.84	0.03	0%	0%	4.71	0.03	0%	0%	5.26	0.04	0%	0%	6.08	0.04
	42C	DO COMP	<P>	0%	0%	1.33	0.32	0%	0%	1.65	0.39	0%	0%	1.84	0.44	0%	0%	2.09	0.50
E	43	E1/A0	<P>	87%	0%	1.65	0.13	87%	0%	1.98	0.16	87%	0%	2.21	0.18	101%	0%	2.63	0.21
	44	E1/B0	<P>	88%	0%	1.84	0.09	112%	0%	2.37	0.12	112%	0%	2.65	0.13	112%	0%	3.02	0.15
	45	E0	<P>	0%	0%	3.14	0.02	0%	0%	3.83	0.02	0%	0%	4.28	0.03	0%	0%	4.93	0.03
	45C	EO COMP	<P>	0%	0%	1.54	0.21	0%	0%	1.85	0.25	0%	0%	2.07	0.28	0%	0%	2.45	0.33
F	46	F1/A0	<P>	97%	0%	3.68	0.47	104%	0%	4.52	0.58	110%	0%	5.08	0.65	116%	0%	5.92	0.76
	47	F1/B0	<P>	89%	0%	3.77	0.36	97%	0%	4.67	0.44	97%	0%	5.21	0.50	104%	0%	6.02	0.57
	48	F0	<P>	0%	0%	4.48	0.03	0%	0%	5.52	0.04	0%	0%	6.16	0.04	0%	0%	7.14	0.05
	48C	F0 COMP	<P>	0%	0%	2.77	0.64	0%	0%	3.38	0.78	0%	0%	3.79	0.87	0%	0%	4.37	1.01
G	49	G1/A0	<P>	98%	0%	1.67	0.18	113%	0%	2.09	0.22	113%	0%	2.34	0.25	113%	0%	2.67	0.28
	50	G1/B0	<P>	0%	0%	3.55	0.19	0%	0%	4.35	0.24	0%	0%	4.85	0.26	0%	0%	5.60	0.30
	51	G0	<P>	0%	0%	3.66	0.01	0%	0%	4.48	0.01	0%	0%	5.00	0.02	0%	0%	5.78	0.02
	51C	G0 COMP	<P>	0%	0%	1.49	0.24	0%	0%	1.85	0.30	0%	0%	2.07	0.34	0%	0%	2.35	0.38

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Table 13: Comparison of Cover Types Present at HBPP Before and After Execution of Final Site Restoration – 2003 vs. 2018

SITE SURFACE COVERAGE TYPES PRESENT IN 2003 AREIAL PHOTO		
AREAS	ACRES	CURVE NUMBER
TOTAL PARCEL	95.33	NA
TOTAL PAVED	9.65	98
TOTAL GRAVEL	0.71	91
TOTAL GRASS	84.97	78
COMPOSITE CURVE NUMBER		80.1

PROPOSED SITE SURFACE COVERAGE TYPES AFTER 2018 FSR		
AREAS	ACRES	CURVE NUMBER
TOTAL PARCEL	95.33	NA
TOTAL PAVED	8.85	98
TOTAL GRAVEL	1.39	91
TOTAL GRASS	85.09	78
COMPOSITE CURVE NUMBER		80.0

COMPARISON OF 2003 AND 2018 SITE SURFACE COVERAGE	
AREAS	ACRES
TOTAL PARCEL	0.00
TOTAL PAVED	-0.80
TOTAL GRAVEL	0.68
TOTAL GRASS	0.12
COMPOSITE CURVE NUMBER	-0.1

Table 14: Critical Design Values for, and Resulting Sizes of, Proposed Bio-detention Basins

BASIN NAME	TOTAL CONTRIBUTING AREA (SQFT)	IMPERVIOUS CONTRIBUTING AREA (SQFT)	GRAVEL CONTRIBUTING AREA (SQFT)	GRASS CONTRIBUTING AREA (SQFT)	FOOTPRINT AVAILABLE FOR INFILTRATION (SQFT)	MINIMUM REQUIRED CAPTURE VOLUME (CUFT)	APPLIED FACTOR OF SAFETY	FINAL DESIGNED CAPTURE VOLUME (CUFT)
BASIN A	141,769	44,305	11,338	86,126	11,928	2,958	1.5	4,438
BASIN B	336,374	156,622	22,742	157,010	10,400	8,917	1.5	13,376
BASIN C	228082	121300	26406	80376	7,310	7135	1.5	10,703
BASIN D	11621	10646	0	975	400	639	1.5	959
BASIN E	6907	5982	0	925	469	342	1.5	513
BASIN F	11225	10223	0	1002	486	611	1.5	916
BASIN G	7779	7322	0	457	350	453	1.5	679
BASIN A + C	369,851	165,605	37,744	166,502	19,238	10,094	1.5	15,140

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Table 15: Recommended Bypass Design Flows

BASINS	RECOMMENDED BYPASS FLOWS FOR WORST CASE (100-YEAR) EVENT (CFS)
A+C	10.60
B	10.24
D	0.50
E	0.33
F	1.01
G	0.38

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Table 16: Comparison of Predicted Peak Flows with the Conventional CIA Method vs. the Lagging Method Employed by the Hydrologic Model

10 YEAR RETURN PERIOD (90TH PERCENTILE)				
BASIN NAME	INTENSITY (IN/HR)	Qp WITH CIA METHOD	Qp WITH LAGGING METHOD	PERCENT ERROR
A+C	1.33	6.92	6.69	-3.3%
B	1.37	6.73	6.79	0.9%
D	1.33	0.32	0.32	0.0%
E	1.54	0.21	0.21	0.0%
F	2.77	0.64	0.64	0.0%
G	1.49	0.24	0.24	0.0%
25 YEAR RETURN PERIOD (96TH PERCENTILE)				
BASIN NAME	INTENSITY (IN/HR)	Qp WITH CIA METHOD	Qp WITH LAGGING METHOD	PERCENT ERROR
A+C	1.60	8.30	8.08	-2.7%
B	1.64	8.07	8.13	0.7%
D	1.65	0.39	0.40	2.6%
E	1.85	0.25	0.26	4.0%
F	3.38	0.78	0.78	0.0%
G	1.85	0.30	0.30	0.0%
50 YEAR RETURN PERIOD (98TH PERCENTILE)				
BASIN NAME	INTENSITY (IN/HR)	Qp WITH CIA METHOD	Qp WITH LAGGING METHOD	PERCENT ERROR
A+C	1.79	9.32	9.82	5.4%
B	1.84	9.02	9.12	1.1%
D	1.84	0.44	0.44	0.0%
E	2.07	0.28	0.29	3.6%
F	3.79	0.87	0.88	1.1%
G	2.07	0.34	0.34	0.0%
100 YEAR RETURN PERIOD (99TH PERCENTILE)				
BASIN NAME	INTENSITY (IN/HR)	Qp WITH CIA METHOD	Qp WITH LAGGING METHOD	PERCENT ERROR
A+C	2.03	10.60	11.17	5.4%
B	2.08	10.24	10.31	0.7%
D	2.09	0.50	0.50	0.0%
E	2.45	0.33	0.34	3.0%
F	4.37	1.01	1.01	0.0%
G	2.35	0.38	0.39	2.6%

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Table 17: Predicted Percent Effectiveness of Each Bio-Detention Basin with Varying Design Storms and Infiltration Rates

PERCENT OF TOTAL WATER TREATED: 100 YR RETURN PERIOD						
INFILTRATION RATE (in/hr)	A + C	B	D	E	F	G
4"	94%	75%	65%	82%	71%	73%
3"	91%	68%	57%	76%	63%	64%
2"	88%	60%	47%	66%	52%	53%
1"	76%	48%	31%	47%	35%	37%
0.75"	70%	42%	25%	40%	29%	30%
0.5"	62%	36%	19%	31%	22%	23%
PERCENT OF TOTAL WATER TREATED: 50 YR RETURN PERIOD						
INFILTRATION RATE (in/hr)	A + C	B	D	E	F	G
4"	100%	89%	82%	97%	88%	90%
3"	100%	83%	74%	93%	80%	83%
2"	100%	75%	62%	83%	68%	71%
1"	96%	61%	42%	64%	48%	50%
0.75"	93%	57%	34%	55%	39%	42%
0.5"	88%	51%	25%	43%	29%	31%
PERCENT OF TOTAL WATER TREATED: 25 YR RETURN PERIOD						
INFILTRATION RATE (in/hr)	A + C	B	D	E	F	G
4"	100%	97%	93%	100%	97%	98%
3"	100%	92%	86%	99%	91%	93%
2"	100%	83%	74%	94%	81%	83%
1"	100%	66%	51%	75%	58%	60%
0.75"	100%	61%	42%	65%	48%	50%
0.5"	98%	56%	31%	52%	36%	38%
PERCENT OF TOTAL WATER TREATED: 10 YR RETURN PERIOD						
INFILTRATION RATE (in/hr)	A + C	B	D	E	F	G
4"	100%	90%	83%	97%	88%	90%
3"	100%	83%	75%	93%	81%	83%
2"	100%	74%	63%	83%	69%	72%
1"	100%	64%	44%	64%	49%	52%
0.75"	97%	60%	36%	55%	41%	44%
0.5"	86%	50%	27%	44%	31%	33%
PERCENT OF TOTAL WATER TREATED: AVERAGE OF ALL RETURN PERIODS						
INFILTRATION RATE (in/hr)	A + C	B	D	E	F	G
4"	98%	88%	81%	94%	88%	88%
3"	98%	81%	73%	90%	81%	81%
2"	97%	73%	61%	82%	70%	70%
1"	93%	60%	42%	62%	50%	50%
0.75"	90%	55%	34%	54%	42%	42%
0.5"	83%	48%	25%	43%	31%	31%

Table 18: Results of On Site Infiltrometer Testing

Location I.D. Number	Location Description	Infiltration Rate (in/hr)
1	ISFSI Basin	1.0
2	Frog Pond Basin	0.0
3	Trailer City Basin	0.5

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Table 19: Description of Analytical Requirements: PG&E Humboldt Bay Generating Station

Analysis Parameter	Analysis Method	Preservation Method	Reporting Limit	Holding time
Total Suspended Solids	SM 2540C	Cool <= 4°C	5 mg/L	7 days
pH	Portable Field Instrument	Cool <= 4°C	0.1 SU	15 minutes
Oil and grease	USEPA 1664A	Cool <= 4°C and acidify with HCl to pH < 2	5 mg/L	28 days
Total Iron	USEPA 200.7	Acidify with HNO_3 to pH<2	0.1 mg/L	6 months
mg/L = milligrams per liter. SU = standard unit. USEPA = United States Environmental Protection Agency				

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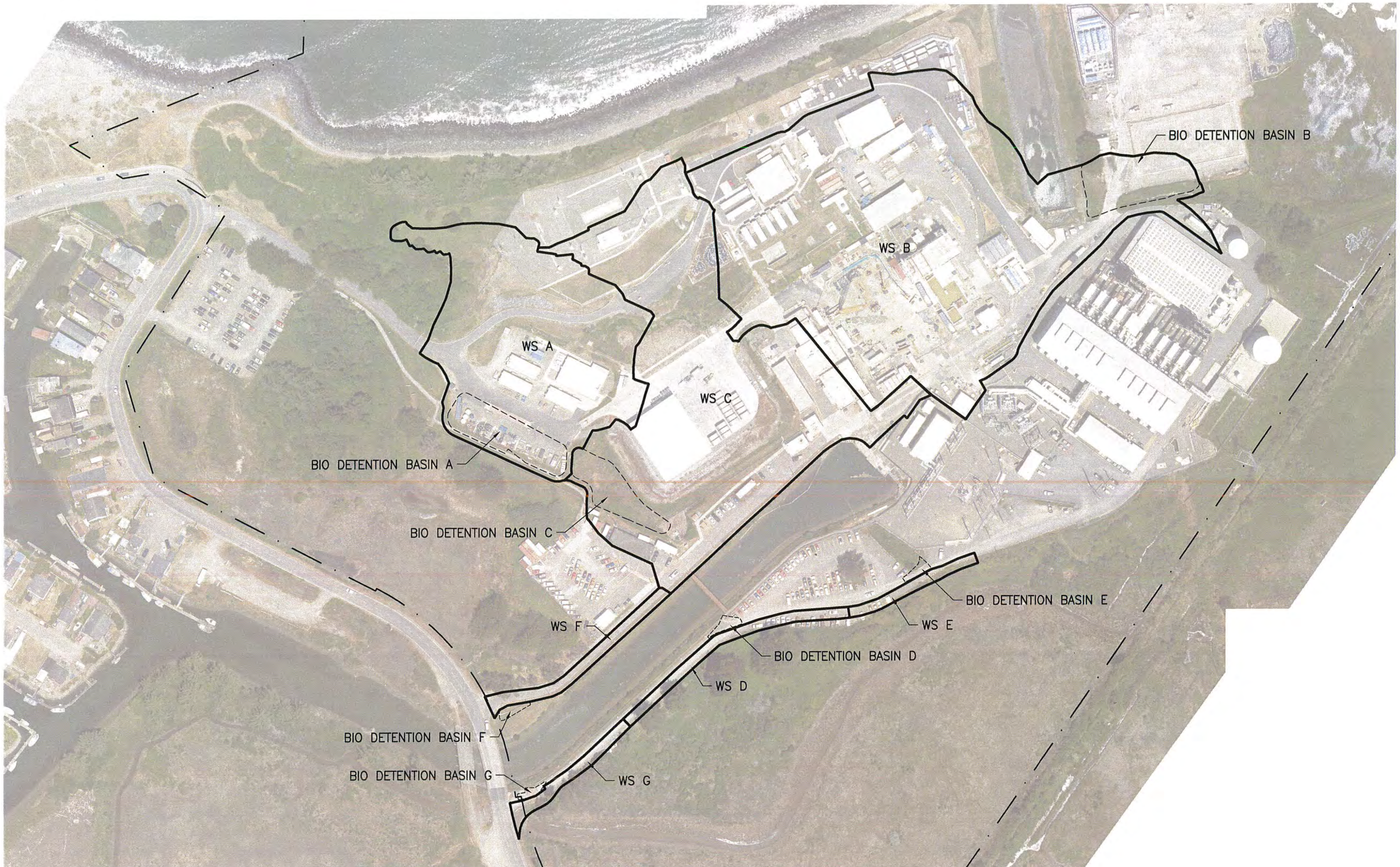
Attachments



PRELIMINARY

WATERSHEDS (WS) WITHIN REGION OF ANALYSIS

SCALE: 1"=200'

0 100' 200' 400'

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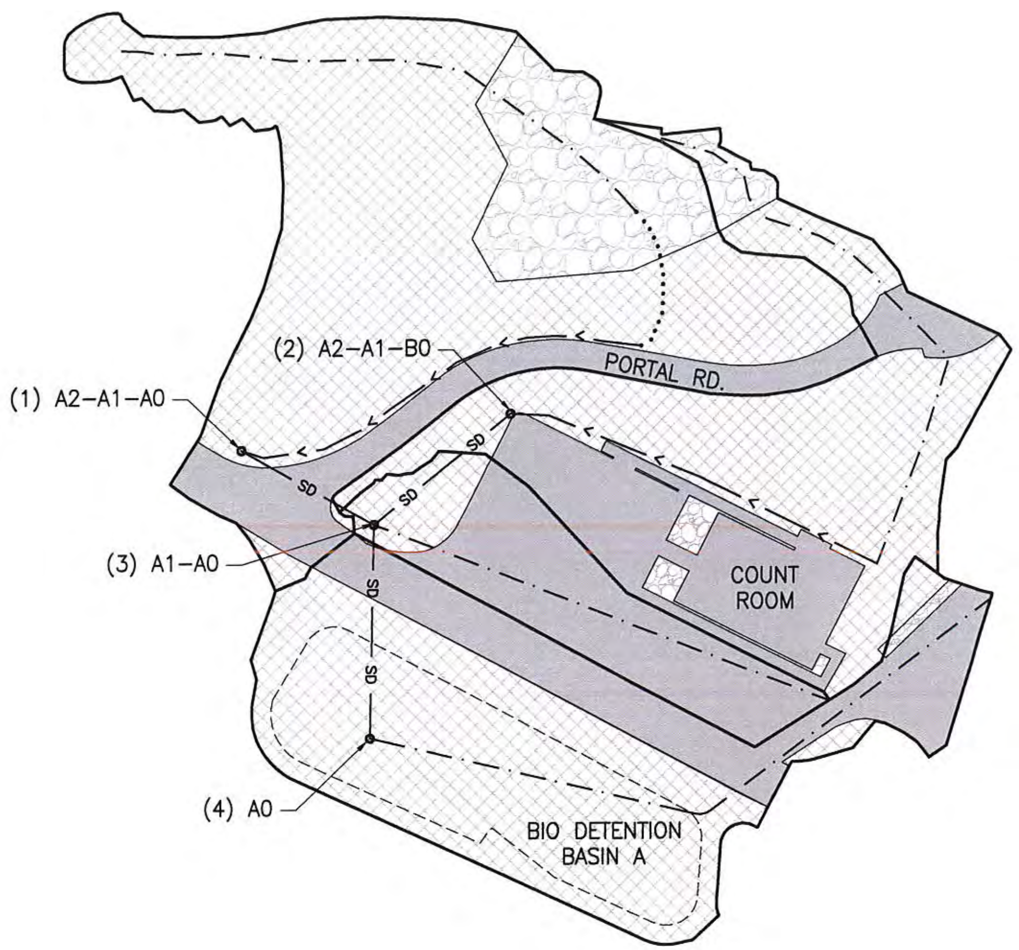
|— 1 INCH —|

DATE	REVISION DESCRIPTION	HUMBOLDT BAY POWER PLANT – PG&E CO. UNIT 3 DECOMMISSIONING		
5-28-15		WATERSHEDS		
PR BWW		DRAWING	SHEET	REV
CK DPT		4108684	1 of 10	A
AP JTL				



WATERCOURSE ROUTING & POINTS OF CONCENTRATION WITHIN WATERSHED A

SCALE: 1"=100'

FLOW TYPE LEGEND

- SHEET FLOW
- SHALLOW CONCENTRATED FLOW
- SWALE
- STORM DRAIN PIPE

COVER TYPES LEGEND

- IMPERVIOUS
- GRAVEL
- GRASS
- POINT OF CONCENTRATION

(##) X#–X#

WATERSHED ID#

WATERSHED NAME

0 50' 100' 200'

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DATE		REVISION DESCRIPTION	HUMBOLDT BAY POWER PLANT – PG&E CO. UNIT 3 DECOMMISSIONING WATERSHED A		
5–28–15			DRAWING	SHEET	REV
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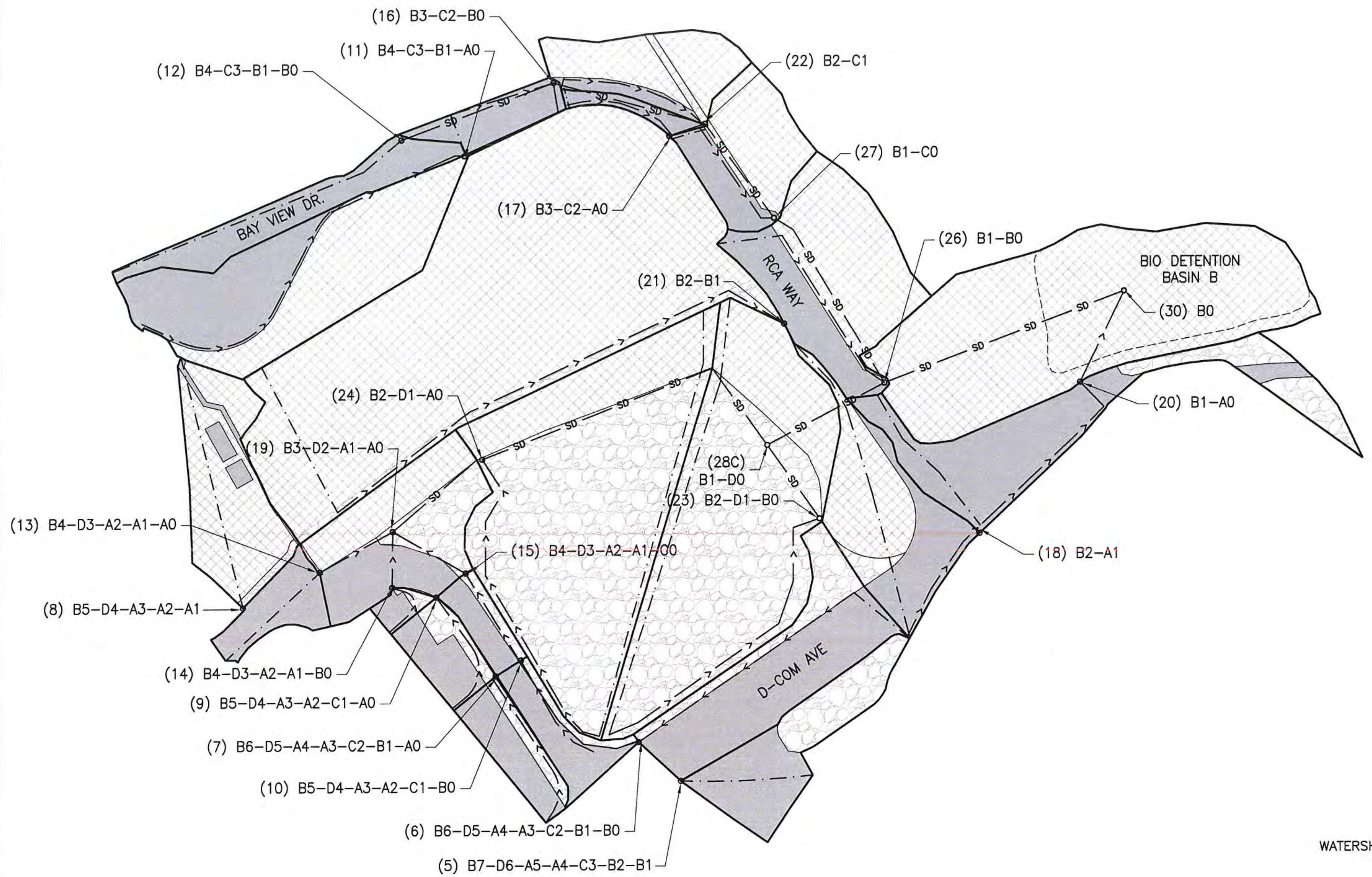
CK	DPT				
AP	JTL				



FLOW TYPE LEGEND

- SHEET FLOW
- SHALLOW CONCENTRATED FLOW
- SWALE
- STORM DRAIN PIPE

COVER TYPES LEGEND

- IMPERVIOUS
- GRAVEL
- GRASS
- POINT OF CONCENTRATION

(##) X#-X#

WATERSHED ID# — WATERSHED NAME

WATERCOURSE ROUTING & POINTS OF CONCENTRATION WITHIN WATERSHED B

SCALE: 1"=100' (2013 AERIAL SHOWN FOR CONTEXT)

0 50' 100' 200'

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1 INCH

PRELIMINARY

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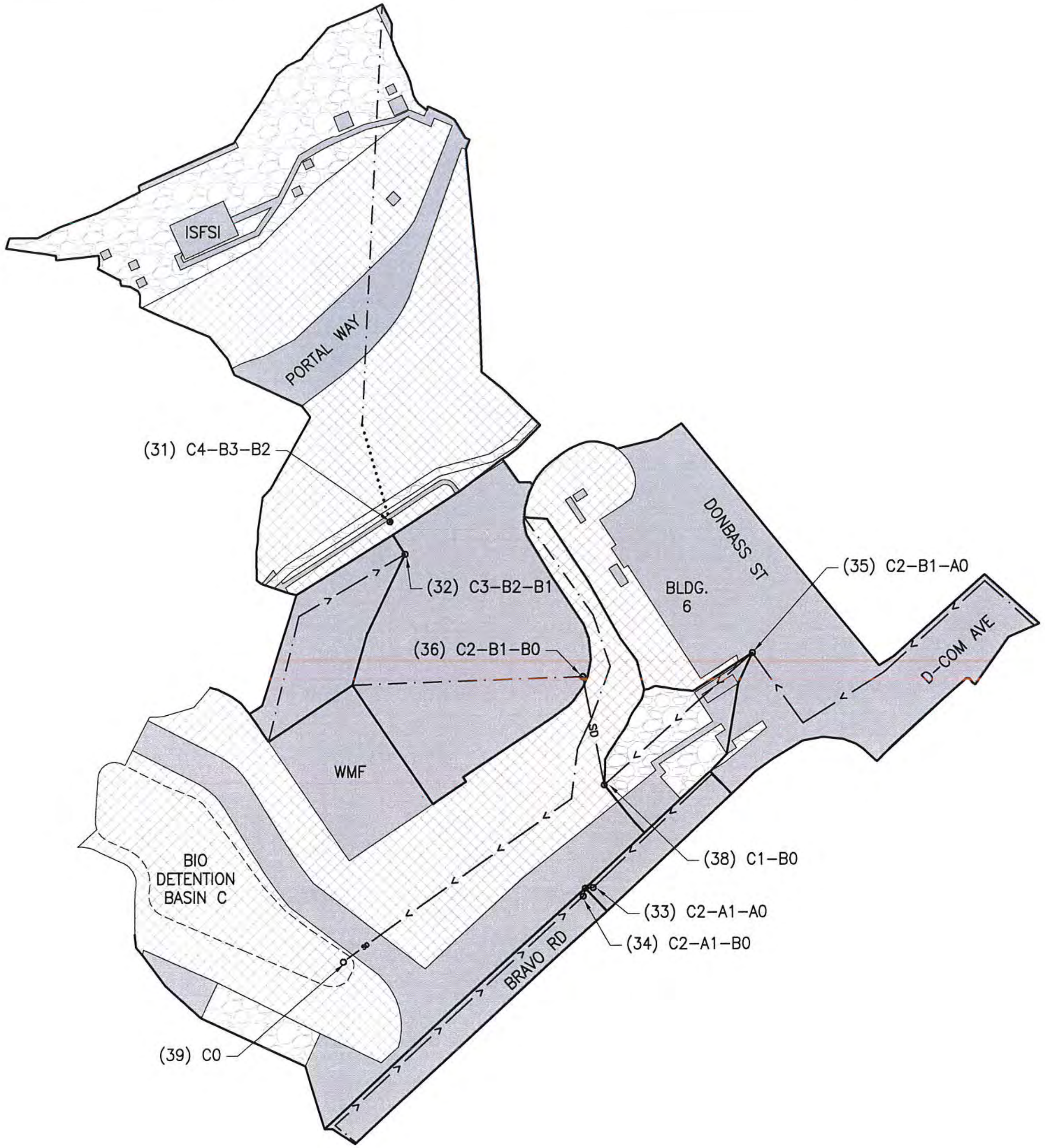
5-28-15					
PR	BWW		DRAWING	SHEET	REV
CK	DPT				
AP	JTL		4108684	3 of 10	A



FLOW TYPE LEGEND

- SHEET FLOW
- SHALLOW CONCENTRATED FLOW
- SWALE
- SD — SD — SD — STORM DRAIN PIPE

COVER TYPES LEGEND

- IMPERVIOUS
- GRAVEL
- GRASS
- POINT OF CONCENTRATION

(##) X#–X#

WATERSHED ID#

WATERSHED NAME

PRELIMINARY

WATERCOURSE ROUTING & POINTS OF CONCENTRATION WITHIN WATERSHED C

SCALE: 1"=100'

0 50' 100' 200'

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DATE		REVISION DESCRIPTION	HUMBOLDT BAY POWER PLANT – PG&E CO. UNIT 3 DECOMMISSIONING WATERSHED C		
5–28–15					
PR	BWW		DRAWING	SHEET	REV
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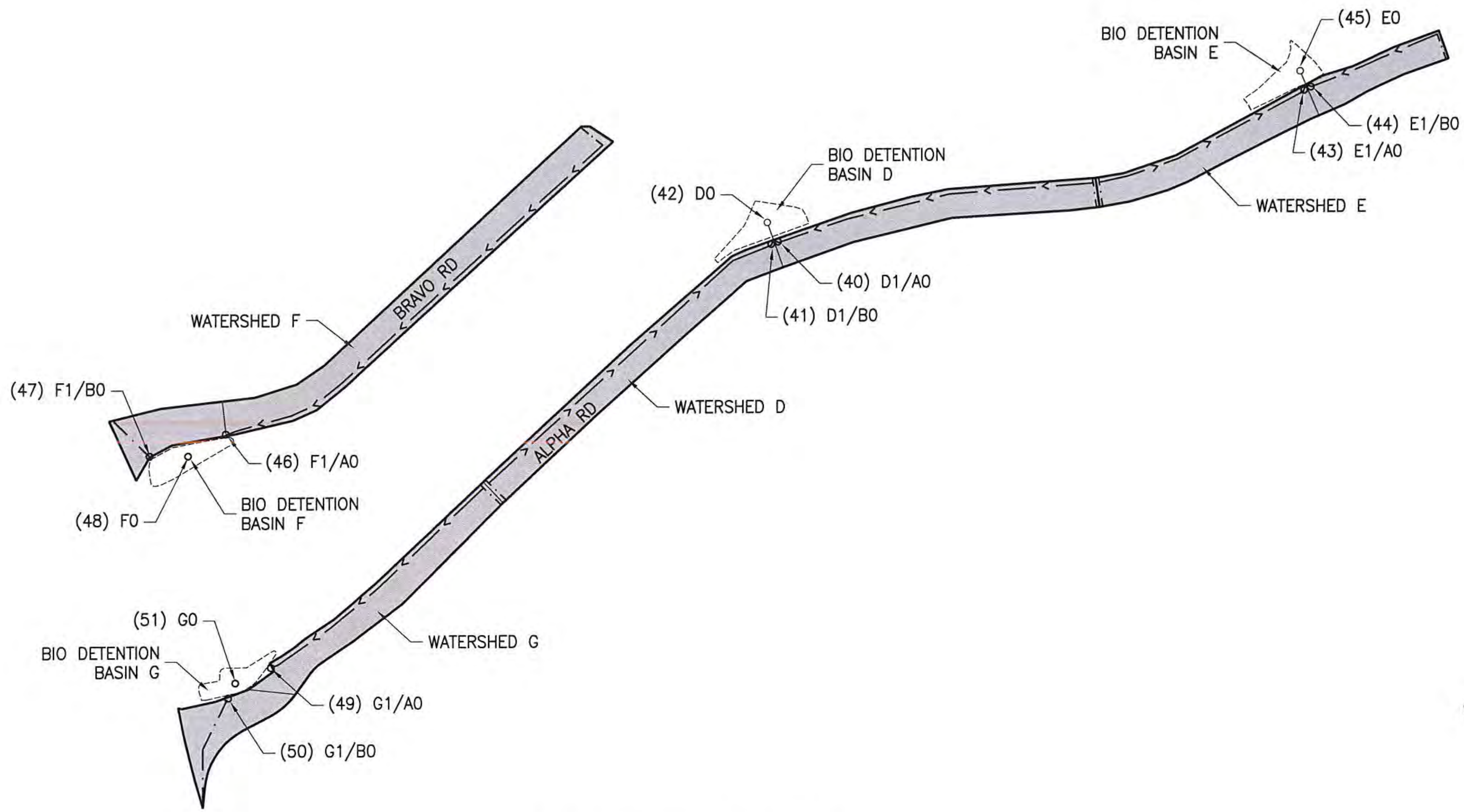
AP	JTL				



FLOW TYPE LEGEND

- SHEET FLOW
- SHALLOW CONCENTRATED FLOW
- SWALE
- SD — SD — SD — STORM DRAIN PIPE

COVER TYPES LEGEND

- IMPERVIOUS
- GRAVEL
- GRASS
- (##) X#–X# — POINT OF CONCENTRATION

WATERSHED ID# WATERSHED NAME

WATERSHED D, E, F, & G

SCALE: 1"=100'

0 50' 100' 200'

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1 INCH

PRELIMINARY

DATE	REVISION DESCRIPTION	HUMBOLDT BAY POWER PLANT – PG&E CO. UNIT 3 DECOMMISSIONING WATERSHED D, E, F, & G		
5-28-15		DRAWING	SHEET	REV
PR BWW		4108684	5 of 10	A
CK DPT				
AP JTL				

ABBREVIATIONS

L —WATER COURSE LENGTH (ft.)
S —WATER COURSE SLOPE
Cv —SURFACE WHICH CARRIES FLOW
T —TIME OF CONCENTRATION (min.)

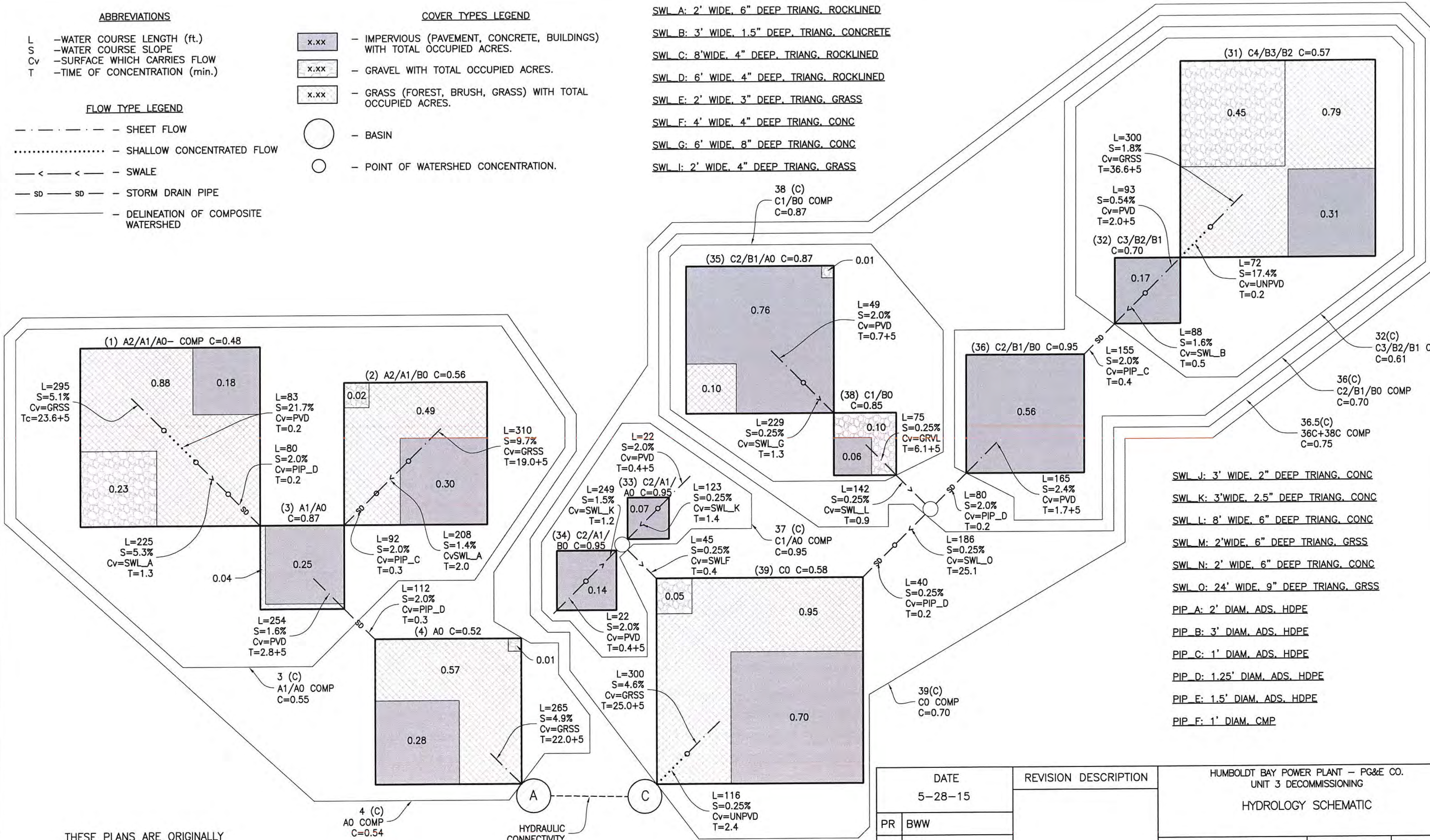
FLOW TYPE LEGEND

— SHEET FLOW
— SHALLOW CONCENTRATED FLOW
— SWALE
— SD — SD — STORM DRAIN PIPE
— DELINEATION OF COMPOSITE WATERSHED

COVER TYPES LEGEND

x.xx — IMPERVIOUS (PAVEMENT, CONCRETE, BUILDINGS) WITH TOTAL OCCUPIED ACRES.
x.xx — GRAVEL WITH TOTAL OCCUPIED ACRES.
x.xx — GRASS (FOREST, BRUSH, GRASS) WITH TOTAL OCCUPIED ACRES.
— BASIN
— POINT OF WATERSHED CONCENTRATION.

SWL_A: 2' WIDE, 6" DEEP TRIANG, ROCKLINED
SWL_B: 3' WIDE, 1.5" DEEP, TRIANG, CONCRETE
SWL_C: 8'WIDE, 4" DEEP, TRIANG, ROCKLINED
SWL_D: 6' WIDE, 4" DEEP, TRIANG, ROCKLINED
SWL_E: 2' WIDE, 3" DEEP, TRIANG, GRASS
SWL_F: 4' WIDE, 4" DEEP TRIANG, CONC
SWL_G: 6' WIDE, 8" DEEP TRIANG, CONC
SWL_I: 2' WIDE, 4" DEEP TRIANG, GRASS



SWL_J: 3' WIDE, 2" DEEP TRIANG, CONC
SWL_K: 3'WIDE, 2.5" DEEP TRIANG, CONC
SWL_L: 8' WIDE, 6" DEEP TRIANG, CONC
SWL_M: 2'WIDE, 6" DEEP TRIANG, GRSS
SWL_N: 2' WIDE, 6" DEEP TRIANG, CONC
SWL_O: 24' WIDE, 9" DEEP TRIANG, GRSS
PIP_A: 2' DIAM, ADS, HDPE
PIP_B: 3' DIAM, ADS, HDPE
PIP_C: 1' DIAM, ADS, HDPE
PIP_D: 1.25' DIAM, ADS, HDPE
PIP_E: 1.5' DIAM, ADS, HDPE
PIP_F: 1' DIAM, CMP

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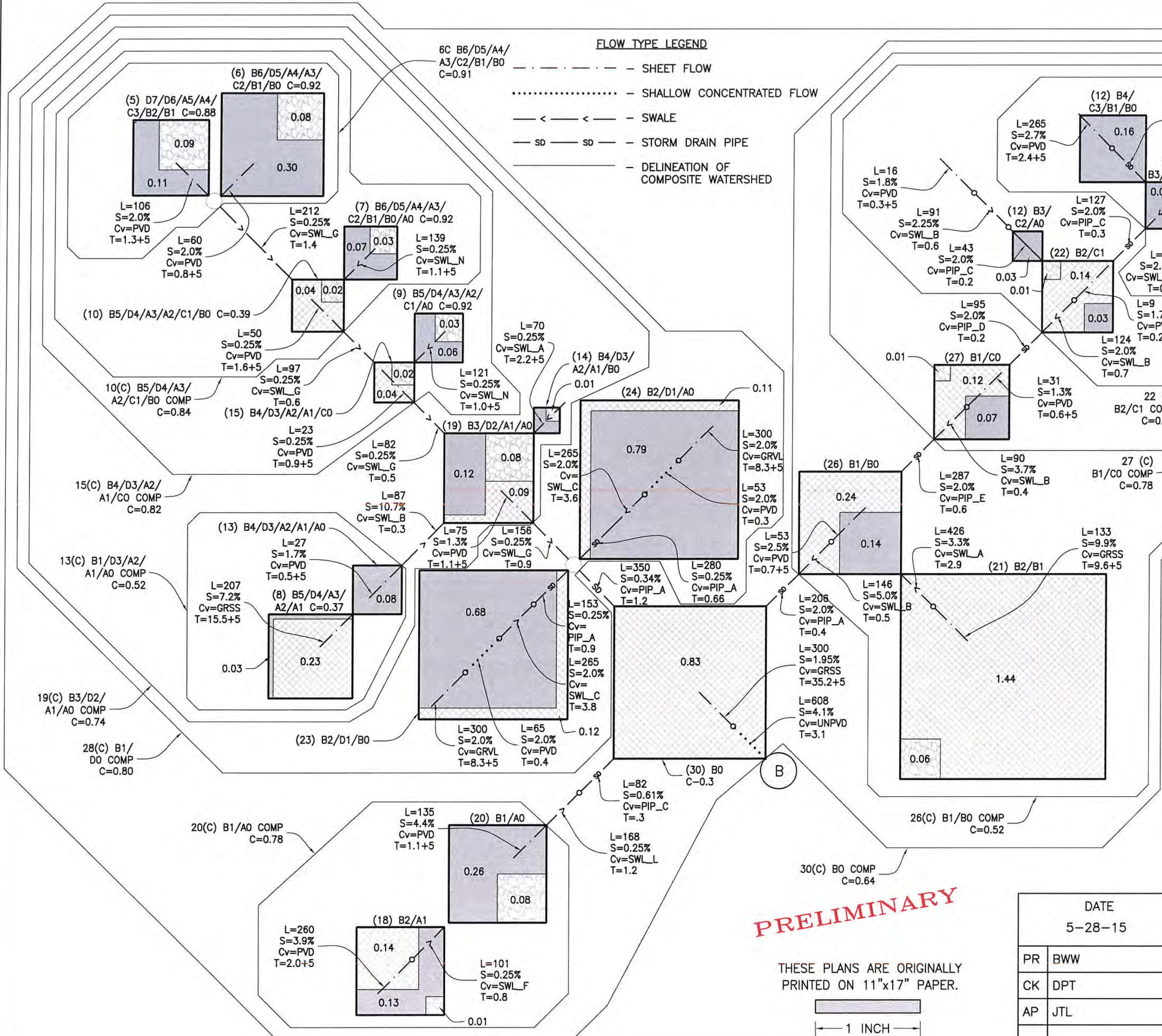
PRELIMINARY

DATE		REVISION DESCRIPTION	HUMBOLDT BAY POWER PLANT – PG&E CO. UNIT 3 DECOMMISSIONING HYDROLOGY SCHEMATIC		
5-28-15					
PR	BWW		DRAWING	SHEET	REV
CK	DPT		4108684	6 of 10	A
AP	JTL				



SWL_A: 2' WIDE, 6" DEEP TRIANG, ROCKLINED

SWL_B: 3' WIDE, 1.5" DEEP, TRIANG, CONCRETE

SWL_C: 8'WIDE, 4" DEEP, TRIANG, ROCKLINED

SWL_D: 6' WIDE, 4" DEEP, TRIANG, ROCKLINED

SWL_E: 2' WIDE, 3" DEEP, TRIANG, GRASS

SWL_F: 4' WIDE, 4" DEEP TRIANG, CONC

SWL_G: 6' WIDE, 8" DEEP TRIANG, CONC

SWL_I: 2' WIDE, 4" DEEP TRIANG, GRASS

SWL_J: 3' WIDE, 2" DEEP TRIANG, CONC

SWL_K: 3'WIDE, 2.5" DEEP TRIANG, CONC

SWL_L: 8' WIDE, 6" DEEP TRIANG, CONC

SWL_M: 2'WIDE, 6" DEEP TRIANG, GRSS

SWL_N: 2' WIDE, 6" DEEP TRIANG, CONC

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PIP_D: 1.25' DIAM, ADS, HDPE

PIP_E: 1.5' DIAM, ADS, HDPE

PIP_F: 1' DIAM, CMP

ABBREVIATIONS

L -WATER COURSE LENGTH (ft.)
S -WATER COURSE SLOPE
Cv -SURFACE WHICH CARRIES FLOW
T -TIME OF CONCENTRATION (min.)

COVER TYPES LEGEND

x.xx – IMPERVIOUS (PAVEMENT, CONCRETE, BUILDINGS) WITH TOTAL OCCUPIED ACRES.

x.xx – GRAVEL WITH TOTAL OCCUPIED ACRES.

x.xx – GRASS (FOREST, BRUSH, GRASS) WITH TOTAL OCCUPIED ACRES.

– BASIN

– POINT OF WATERSHED CONCENTRATION.

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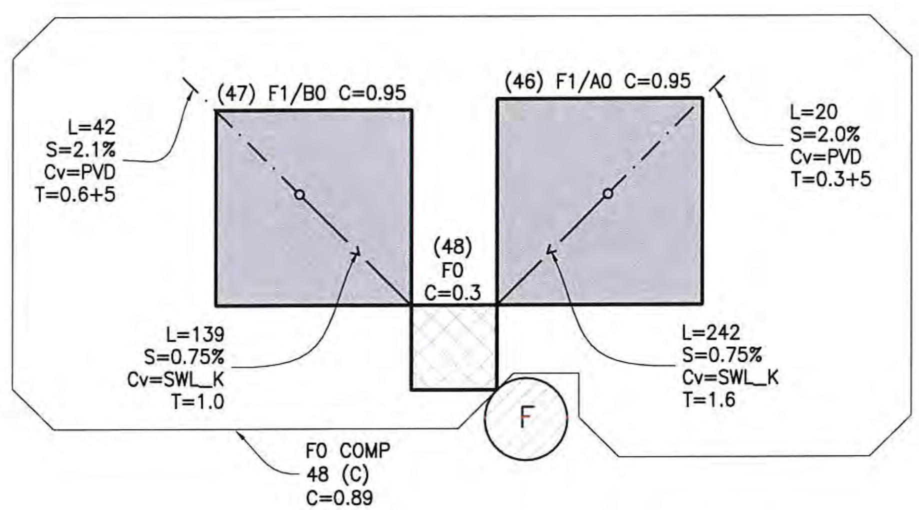
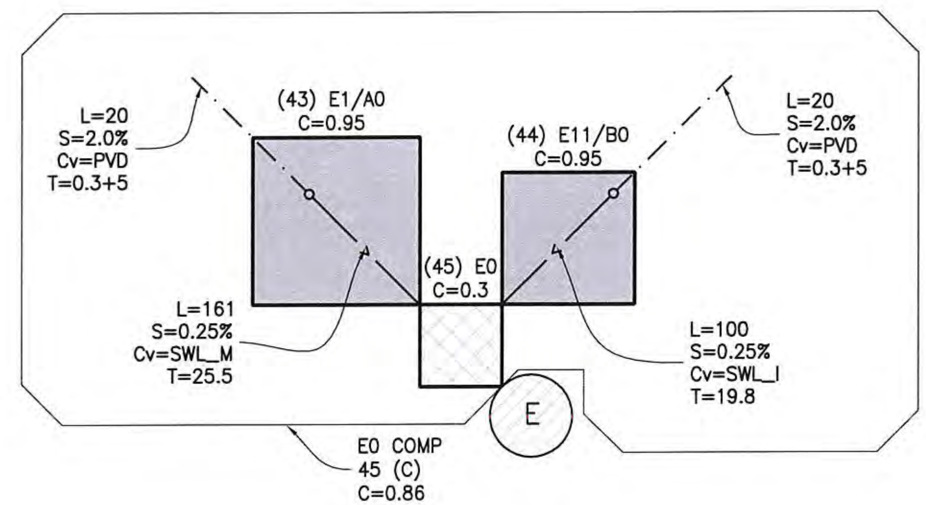
CK	DPT				
AP	JTL		4108684	7 of 10	A





FLOW TYPE LEGEND

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- SWALE
- STORM DRAIN PIPE
- DELINEATION OF COMPOSITE WATERSHED

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SWL_C: 8'WIDE, 4" DEEP, TRIANG, ROCKLINED
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SWL_E: 2' WIDE, 3" DEEP, TRIANG, GRASS
SWL_F: 4' WIDE, 4" DEEP TRIANG, CONC
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SWL_K: 3'WIDE, 2.5" DEEP TRIANG, CONC
SWL_L: 8' WIDE, 6" DEEP TRIANG, CONC
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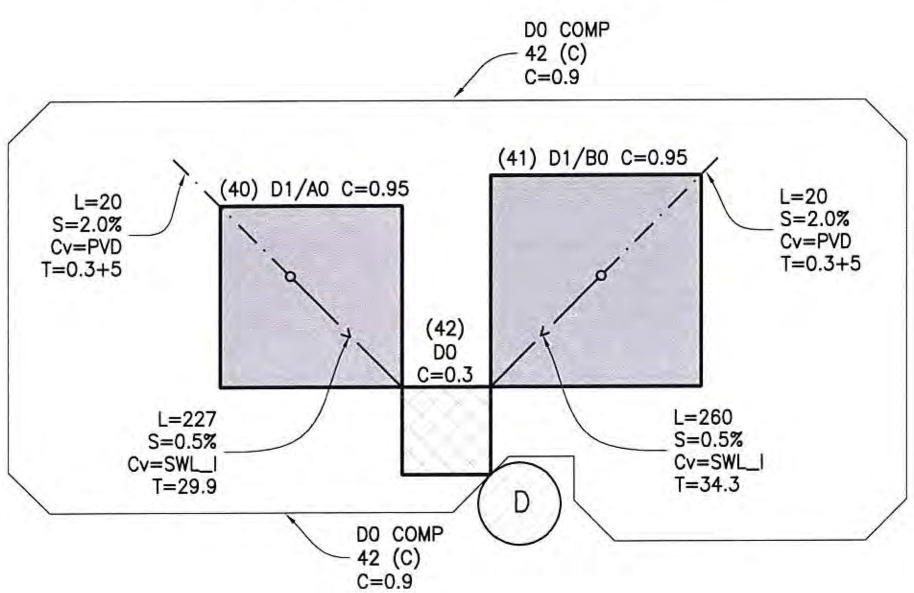

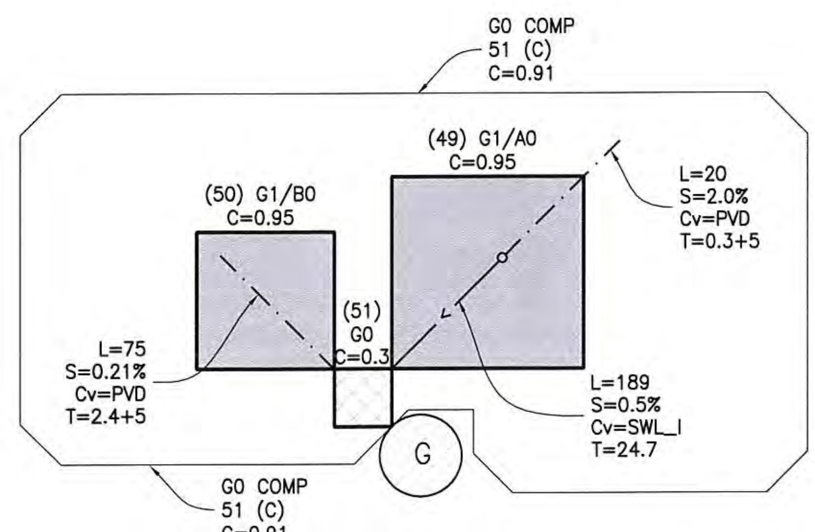

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- x.xx – GRAVEL WITH TOTAL OCCUPIED ACRES.
- x.xx – GRASS (FOREST, BRUSH, GRASS) WITH TOTAL OCCUPIED ACRES.
- – BASIN
- – POINT OF WATERSHED CONCENTRATION.

PRELIMINARY

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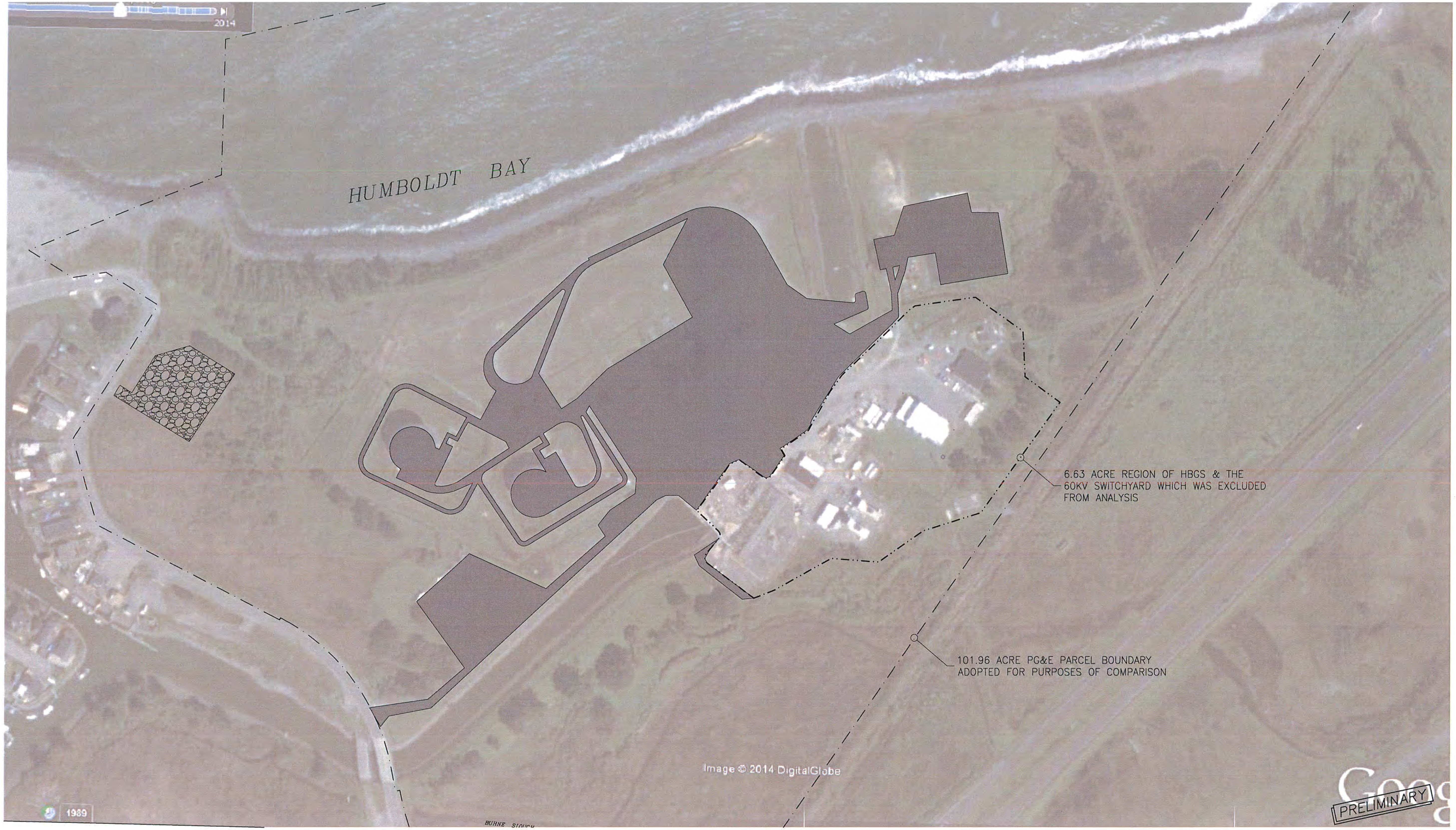
CK DPT				
AP JTL				



PAVED	*±9.65 AC;*	*CN=98*
GRAVELED	*±0.71 AC;*	*CN=91*
GRASS	*±84.97 AC;*	*CN=78*

TOTAL SITE 101.96–95.33 COMPOSITE CN=80.1

SITE PLAN
SCALE: 1"= 100'

TRUE NORTH
PLANT GRID

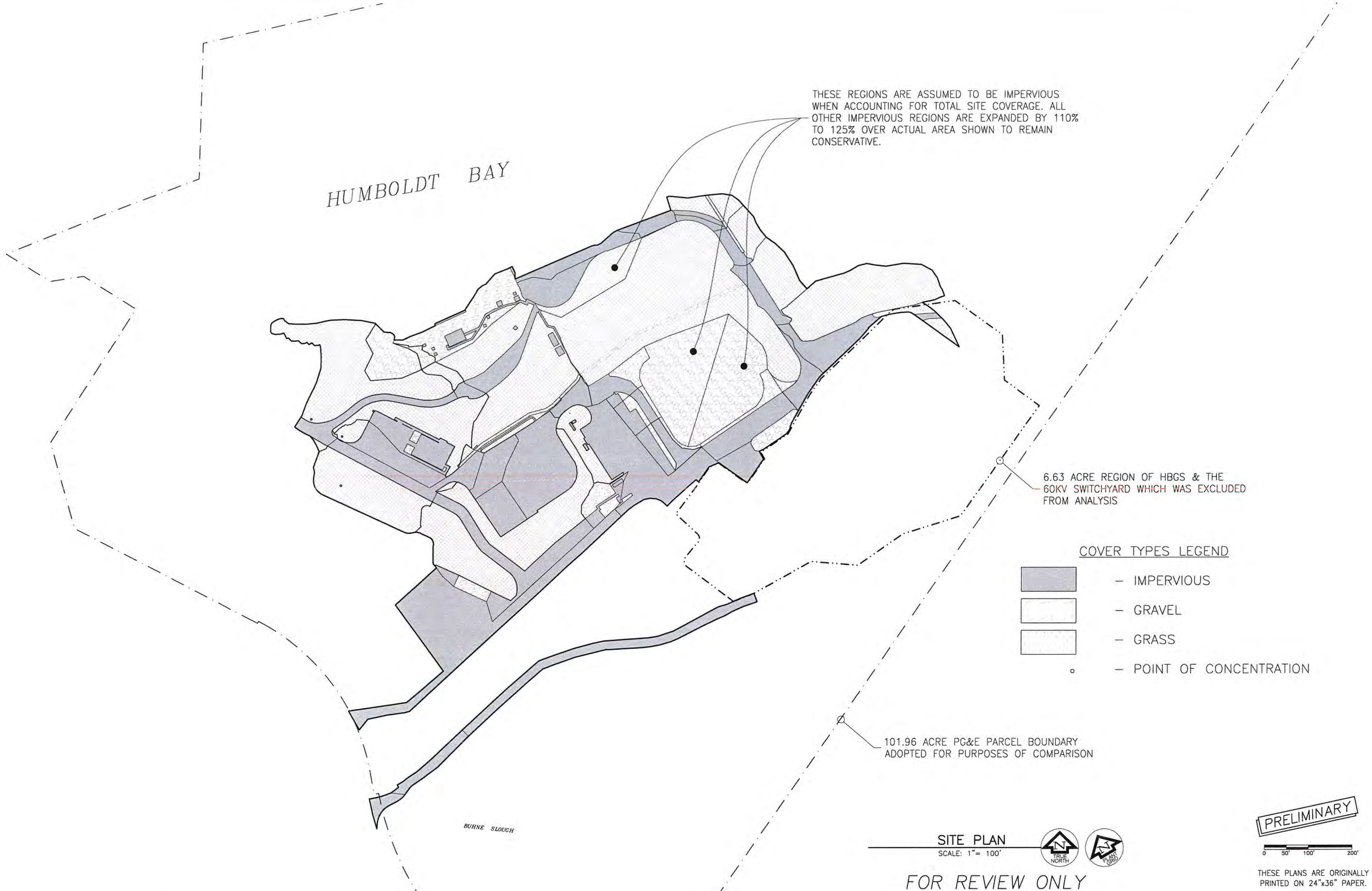
FOR REVIEW ONLY
NOT FOR SUBMITTAL

0 50' 100' 200'

THESE PLANS ARE ORIGINALLY PRINTED ON 24"x36" PAPER.

1 INCH

DATE 5-28-15		REVISION DESCRIPTION	2003 COVER TYPES	DWG SCALE:		
PR	BWW			BILL OF MAT'L.:		
CK	DPT			SUPSDS:		
AP	JTL		PACIFIC GAS & ELECTRIC COMPANY HUMBOLDT BAY POWER PLANT EUREKA, CALIFORNIA	SUPSD BY:		
				DRAWING 4108684	SHEET 9 of 10	REV A

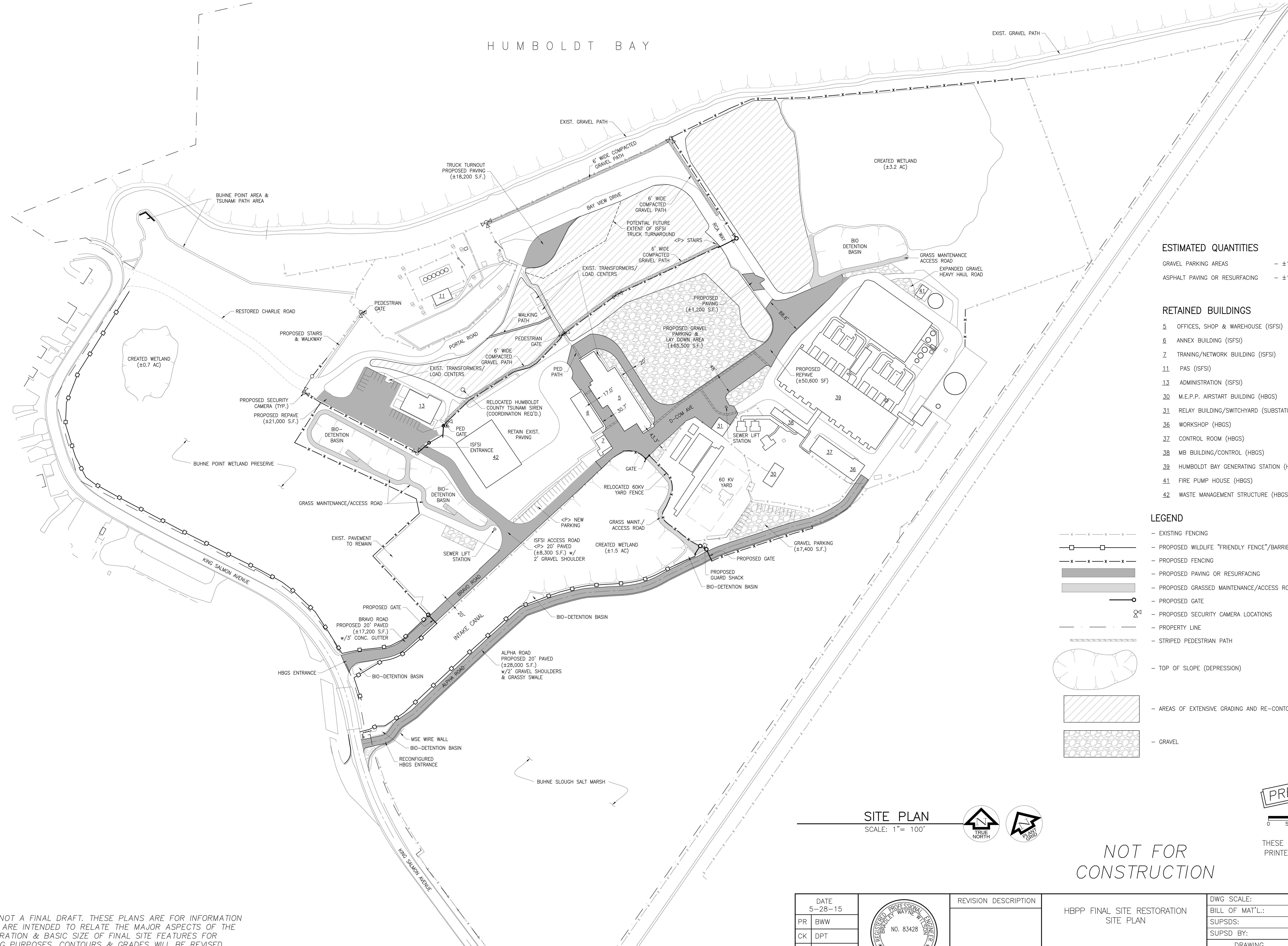


FOR REVIEW ONLY
NOT FOR SUBMITTAL

TOTAL PAVED =8.85 ACRES CN=98
TOTAL GRAVELED =1.39 ACRES CN=91
TOTAL GRASS =85.9 ACRES CN=78
TOTAL SITE =101.96−6.63=95.33 ACRES COMPOSITE CN=80.0

DATE 5–28–15		REVISION DESCRIPTION	2018 COVER TYPES	DWG SCALE:		
PR	BWW			BILL OF MAT'L.:		
CK	DPT			SUPSDS:		
AP	JTL		PACIFIC GAS & ELECTRIC COMPANY HUMBOLDT BAY POWER PLANT EUREKA, CALIFORNIA	SUPSD BY:		
				DRAWING	SHEET	REV
				4108684	10 of 10	A

Final Site Restoration Drawings



ESTIMATED QUANTITIES

GRAVEL PARKING AREAS – ±150,900 S.F. (3.4 AC ±0.1 AC)

ASPHALT PAVING OR RESURFACING – ±130,000 S.F. (3.0 AC ±0.1 AC)

RETAINED BUILDINGS

5 OFFICES, SHOP & WAREHOUSE (ISFSI)
6 ANNEX BUILDING (ISFSI)
7 TRANING/NETWORK BUILDING (ISFSI)
11 PAS (ISFSI)
13 ADMINISTRATION (ISFSI)
30 M.E.P.P. AIRSTART BUILDING (HBGS)
31 RELAY BUILDING/SWITCHYARD (SUBSTATION)
36 WORKSHOP (HBGS)
37 CONTROL ROOM (HBGS)
38 MB BUILDING/CONTROL (HBGS)
39 HUMBOLDT BAY GENERATING STATION (HBGS)
41 FIRE PUMP HOUSE (HBGS)
42 WASTE MANAGEMENT STRUCTURE (HBGS)

LEGEND

– EXISTING FENCING
– PROPOSED WILDLIFE "FRIENDLY FENCE"/BARRIER
– PROPOSED FENCING
– PROPOSED PAVING OR RESURFACING
– PROPOSED GRASSED MAINTENANCE/ACCESS ROAD (±0.40 AC ±0.1 AC)
– PROPOSED GATE
– PROPOSED SECURITY CAMERA LOCATIONS
– PROPERTY LINE
– STRIPED PEDESTRIAN PATH
– TOP OF SLOPE (DEPRESSION)
– AREAS OF EXTENSIVE GRADING AND RE-CONTOURING (5.0 AC ±0.1 AC)
– GRAVEL

PRELIMINARY

0 50' 100' 200'

SITE PLAN
SCALE: 1"= 100'

TRUE NORTH
PLANT GRID

NOT FOR CONSTRUCTION

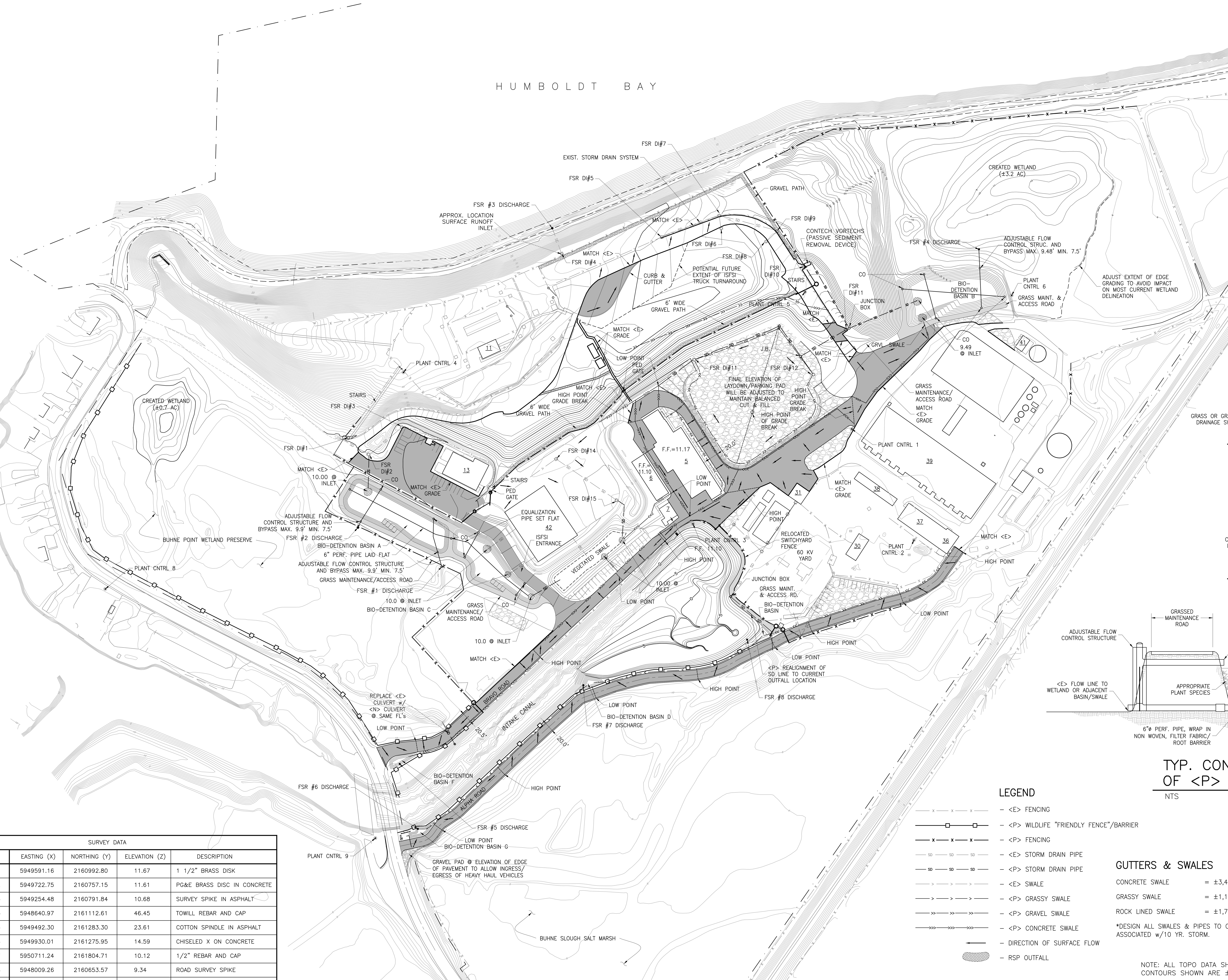
THESE PLANS ARE ORIGINALLY PRINTED ON 24"x36" PAPER.

1 INCH

THIS IS NOT A FINAL DRAFT. THESE PLANS ARE FOR INFORMATION ONLY & ARE INTENDED TO RELATE THE MAJOR ASPECTS OF THE CONFIGURATION & BASIC SIZE OF FINAL SITE FEATURES FOR PLANNING PURPOSES. CONTOURS & GRADES WILL BE REVISED DURING CONSTRUCTION DOCUMENT DETAILING.

DATE 5-28-15		REVISION DESCRIPTION	HBPP FINAL SITE RESTORATION SITE PLAN	DWG SCALE:		
PR	BWW			BILL OF MAT'L.:		
CK	DPT			SUPSDS:		
AP	JTL			SUPSD BY:		
			PACIFIC GAS & ELECTRIC COMPANY HUMBOLDT BAY POWER PLANT EUREKA, CALIFORNIA	DRAWING 4108682	SHEET 1 of 6	REV H

REGISTERED PROFESSIONAL ENGINEER BRADLEY WAYNE WILSON NO. 83428 CIVIL STATE OF CALIFORNIA



SURVEY DATA

MONUMENT ID	EASTING (X)	NORTHING (Y)	ELEVATION (Z)	DESCRIPTION
PLANT CONTROL 1	5949591.16	2160992.80	11.67	1 1/2" BRASS DISK
PLANT CONTROL 2	5949722.75	2160757.15	11.61	PG&E BRASS DISC IN CONCRETE
PLANT CONTROL 3	5949254.48	2160791.84	10.68	SURVEY SPIKE IN ASPHALT
PLANT CONTROL 4	5948640.97	2161112.61	46.45	TOWILL REBAR AND CAP
PLANT CONTROL 5	5949492.30	2161283.30	23.61	COTTON SPINDLE IN ASPHALT
PLANT CONTROL 6	5949930.01	2161275.95	14.59	CHISELED X ON CONCRETE
PLANT CONTROL 7	5950711.24	2161804.71	10.12	1/2" REBAR AND CAP
PLANT CONTROL 8	5948009.26	2160653.57	9.34	ROAD SURVEY SPIKE
PLANT CONTROL 9	5948581.51	2160125.59	10.45	TOWILL REBAR AND CAP
PLANT CONTROL 10	5949382.91	2158905.95	13.52	ROAD SURVEY SPIKE

GRADING & DRAINAGE PLAN

SCALE: 1"= 100'

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NOT FOR CONSTRUCTION

PRELIMINARY

TRUE NORTH — PLANT GRID

0 50' 100' 200'

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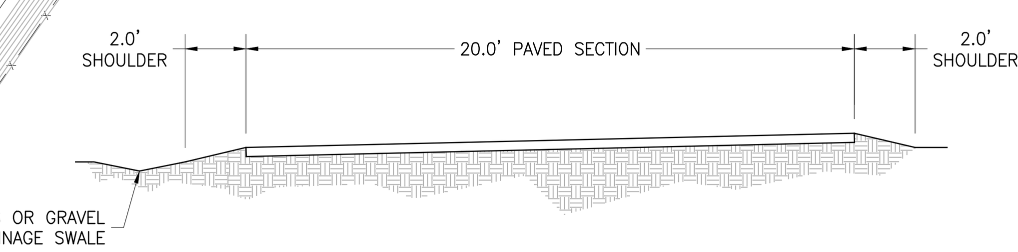
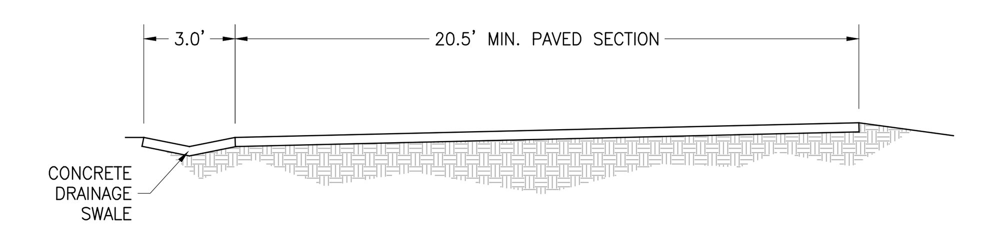
1 INCH



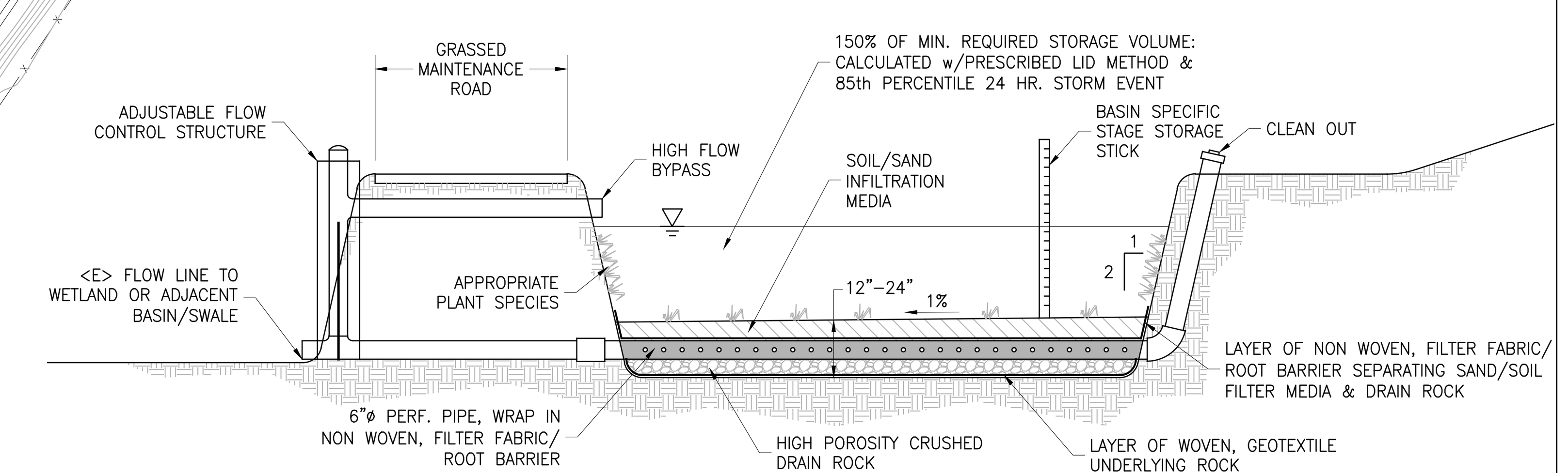
TYP. ALPHA ROAD SECTION

NTS



TYP. BRAVO ROAD SECTION

NTS



TYP. CONCEPTUAL CROSS SECTION OF <P> BIO-DETENTION BASINS

NTS

LEGEND

- <E> FENCING
- <P> WILDLIFE "FRIENDLY FENCE"/BARRIER
- <P> FENCING
- <E> STORM DRAIN PIPE
- <P> STORM DRAIN PIPE
- <E> SWALE
- <P> GRASSY SWALE
- <P> GRAVEL SWALE
- <P> CONCRETE SWALE
- DIRECTION OF SURFACE FLOW
- RSP OUTFALL

GUTTERS & SWALES

CONCRETE SWALE	= ±3,450 L.F. TOTAL
GRASSY SWALE	= ±1,150 L.F. TOTAL
ROCK LINED SWALE	= ±1,750 L.F. TOTAL

*DESIGN ALL SWALES & PIPES TO CONVEY THE MIN. FLOWS ASSOCIATED w/10 YR. STORM.

CUT & FILL QUANTITIES

CUT	FILL	NET IMPORT
54.8 KCYDS	72.7 KCYDS	17.9 KCYDS

ADJUST FINAL ELEVATIONS OF LAYDOWN PADS & SLOPES OF FILL BANKS TO MINIMIZE IMPORT.

VALUES REPRESENT IN-PLACE VOLUMES

NOTE: ALL TOPO DATA SHOWN IS DERIVED FROM THE AERIAL TOPO PERFORMED ON 11/2013 CONTOURS SHOWN ARE ±6". ALL GRADES TO BE VERIFIED DURING CONSTRUCTION DETAILING.

DATE	5-28-15
PR	BWW
CK	DPT
AP	JTL

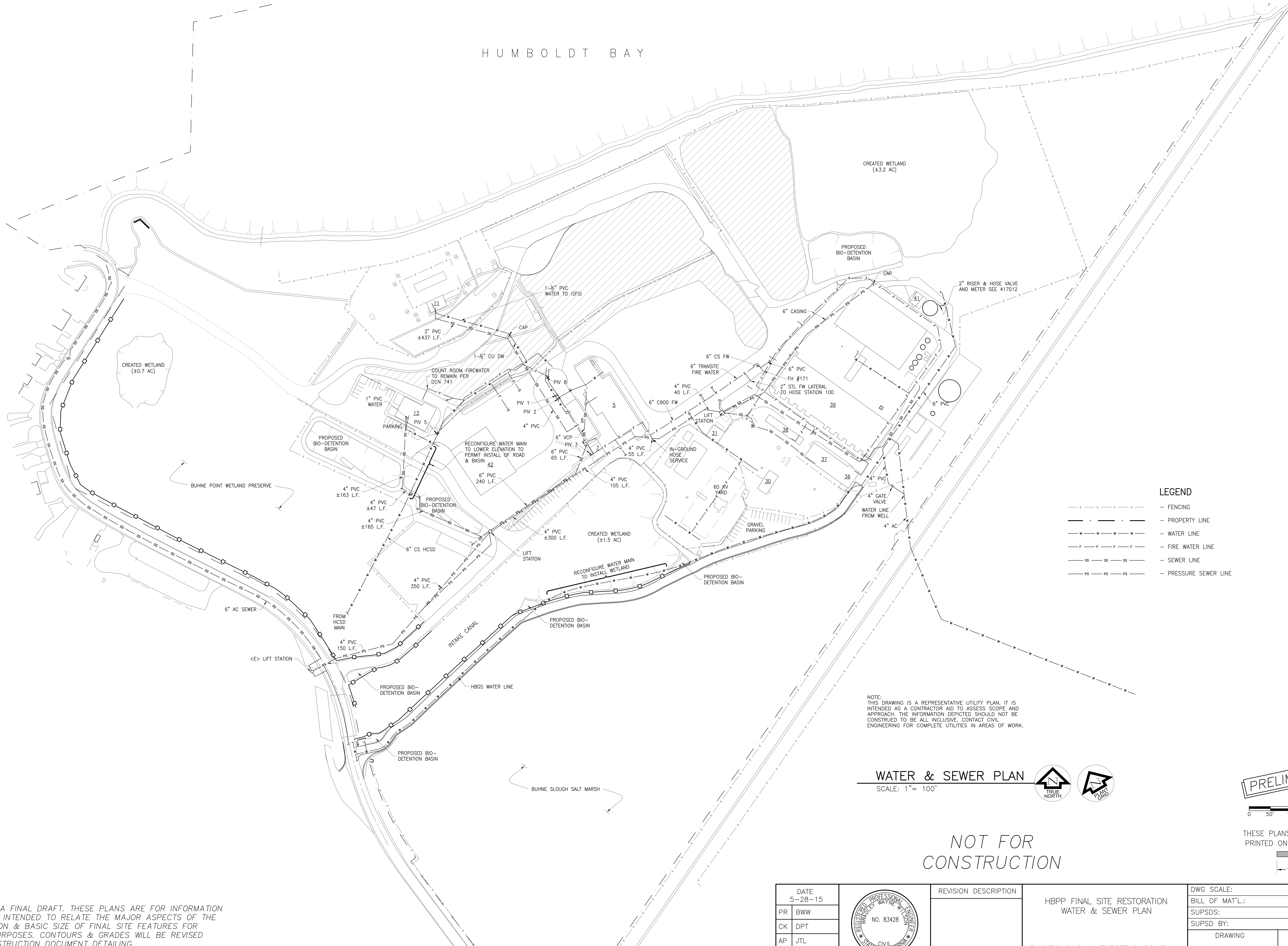
REGISTERED PROFESSIONAL ENGINEER BRADLEY WAYNE WILLSON NO. 83428 CIVIL STATE OF CALIFORNIA

REVISION DESCRIPTION

HBPP FINAL SITE RESTORATION GRADING & DRAINAGE PLAN

PACIFIC GAS & ELECTRIC COMPANY
HUMBOLDT BAY POWER PLANT EUREKA, CALIFORNIA

DWG SCALE:		
BILL OF MAT'L.:		
SUPSDS:		
SUPSD BY:		
DRAWING	SHEET	REV
4108682	2 of 6	H



LEGEND

- FENCING
- PROPERTY LINE
- WATER LINE
- FIRE WATER LINE
- SEWER LINE
- PRESSURE SEWER LINE

NOTE:
THIS DRAWING IS A REPRESENTATIVE UTILITY PLAN. IT IS INTENDED AS A CONTRACTOR AID TO ASSESS SCOPE AND APPROACH. THE INFORMATION DEPICTED SHOULD NOT BE CONSTRUED TO BE ALL INCLUSIVE. CONTACT CIVIL ENGINEERING FOR COMPLETE UTILITIES IN AREAS OF WORK.

WATER & SEWER PLAN

SCALE: 1"= 100'

TRUE NORTH

PLANT GRID

PRELIMINARY

0 50' 100' 200'

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1 INCH

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DATE 5-28-15	
PR	BWW
CK	DPT
AP	JTL

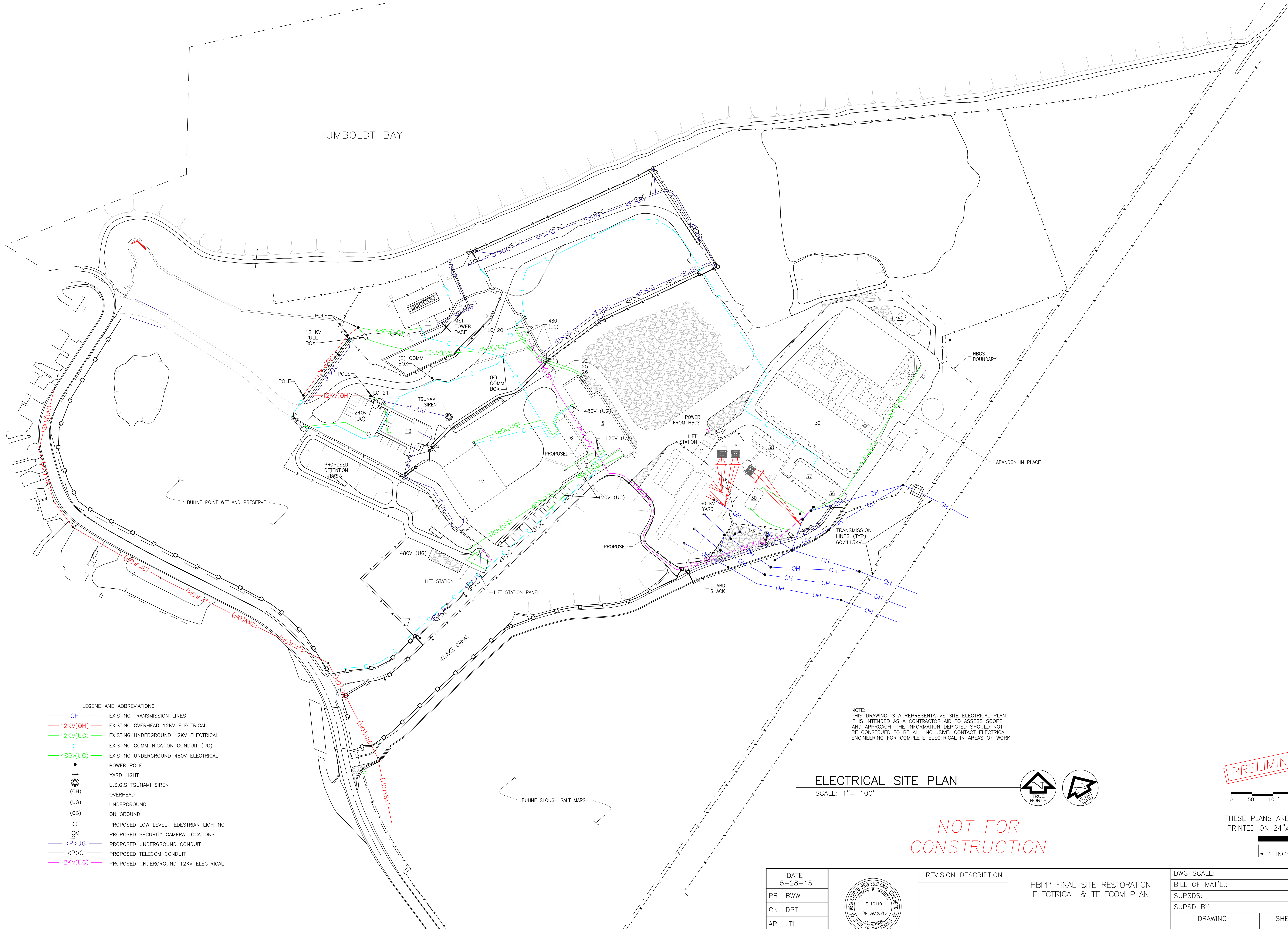
REGISTERED PROFESSIONAL ENGINEER BRADLEY WAYNE WILLSON NO. 83428 CIVIL STATE OF CALIFORNIA

REVISION DESCRIPTION

HBPP FINAL SITE RESTORATION
WATER & SEWER PLAN

PACIFIC GAS & ELECTRIC COMPANY
HUMBOLDT BAY POWER PLANT EUREKA, CALIFORNIA

DWG SCALE:		
BILL OF MAT'L.:		
SUPSDS:		
SUPSD BY:		
DRAWING	SHEET	REV
4108682	3 of 6	H



LEGEND AND ABBREVIATIONS

—— OH ——	EXISTING TRANSMISSION LINES
—— 12KV(OH) ——	EXISTING OVERHEAD 12KV ELECTRICAL
—— 12KV(UG) ——	EXISTING UNDERGROUND 12KV ELECTRICAL
—— C ——	EXISTING COMMUNICATION CONDUIT (UG)
—— 480v(UG) ——	EXISTING UNDERGROUND 480V ELECTRICAL
●	POWER POLE
	YARD LIGHT
	U.S.G.S TSUNAMI SIREN
(OH)	OVERHEAD
(UG)	UNDERGROUND
(OG)	ON GROUND
	PROPOSED LOW LEVEL PEDESTRIAN LIGHTING
	PROPOSED SECURITY CAMERA LOCATIONS
—— <P>UG ——	PROPOSED UNDERGROUND CONDUIT
—— <P>C ——	PROPOSED TELECOM CONDUIT
—— 12KV(UG) ——	PROPOSED UNDERGROUND 12KV ELECTRICAL

NOTE:
THIS DRAWING IS A REPRESENTATIVE SITE ELECTRICAL PLAN. IT IS INTENDED AS A CONTRACTOR AID TO ASSESS SCOPE AND APPROACH. THE INFORMATION DEPICTED SHOULD NOT BE CONSTRUED TO BE ALL INCLUSIVE. CONTACT ELECTRICAL ENGINEERING FOR COMPLETE ELECTRICAL IN AREAS OF WORK.

ELECTRICAL SITE PLAN

SCALE: 1"= 100'

0 50' 100' 200'

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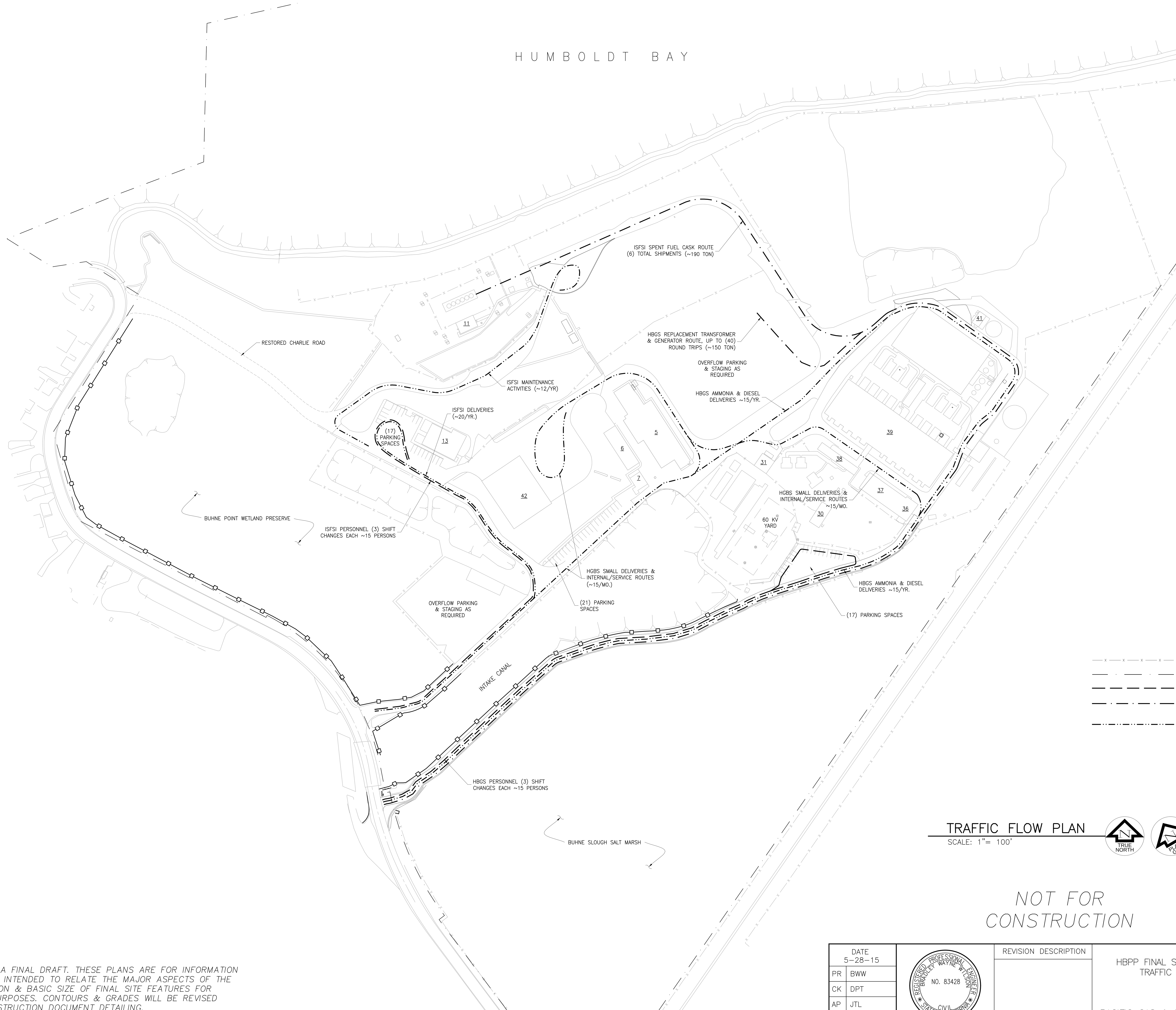
1 INCH

NOT FOR CONSTRUCTION

DATE 5-28-15		REVISION DESCRIPTION	HBPP FINAL SITE RESTORATION ELECTRICAL & TELECOM PLAN	DWG SCALE:		
PR	BWW			BILL OF MAT'L.:		
CK	DPT			SUPSDS:		
AP	JTL			SUPSD BY:		
			PACIFIC GAS & ELECTRIC COMPANY HUMBOLDT BAY POWER PLANT EUREKA, CALIFORNIA	DRAWING 4108682	SHEET 4 of 6	REV H

REGISTERED PROFESSIONAL ENGINEER EDWIN R. KAHLER E 10110 Exp. 09/30/15 ELECTRICAL STATE OF CALIFORNIA



RETAINED BUILDINGS

5 OFFICES, SHOP & WAREHOUSE (ISFSI)
6 ANNEX BUILDING (ISFSI)
7 TRANING/NETWORK BUILDING (ISFSI)
11 PAS (ISFSI)
13 ADMINISTRATION (ISFSI)
30 M.E.P.P. AIRSTART BUILDING (HBGS)
31 RELAY BUILDING/SWITCHYARD (SUBSTATION)
36 WORKSHOP (HBGS)
37 CONTROL ROOM (HBGS)
38 MB BUILDING/CONTROL (HBGS)
39 HUMBOLDT BAY GENERATING STATION (HBGS)
41 FIRE PUMP HOUSE (HBGS)
42 WASTE MANAGEMENT STRUCTURE (HBGS)

LEGEND

- FENCING
- PROPERTY LINE
- DAILY PERSONNEL ROUTES
- CASK & HBGS REPLACEMENT COMPONENT ROUTE (HEAVY HAUL ROAD)
- TRUCK/DELIVERY ROUTE

PRELIMINARY

0 50' 100' 200'

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DATE 5-28-15	
PR	BWW
CK	DPT
AP	JTL

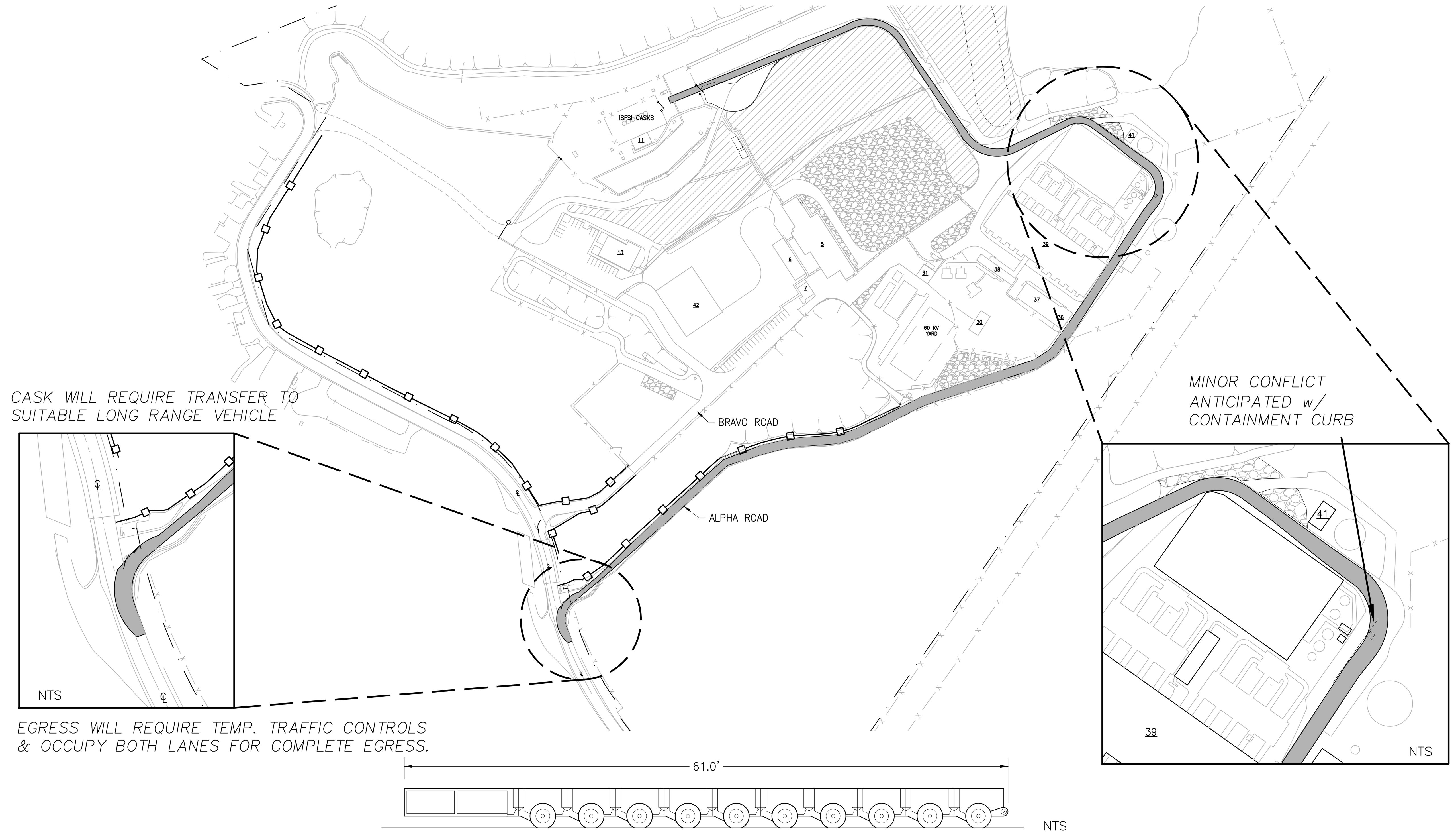
REGISTERED PROFESSIONAL ENGINEER BRADLEY WAYNE WILSON NO. 83428 CIVIL STATE OF CALIFORNIA

REVISION DESCRIPTION

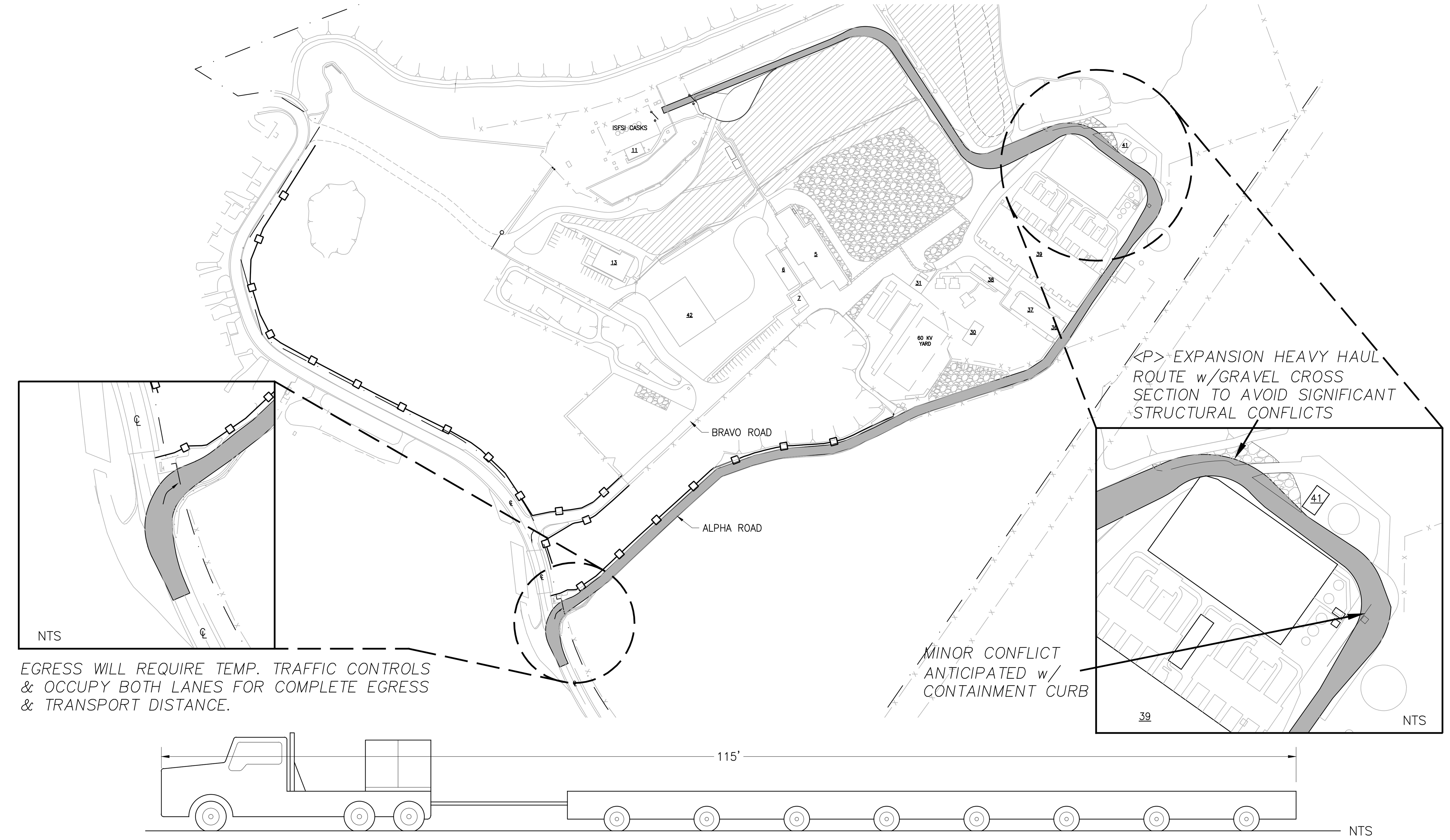
HBPP FINAL SITE RESTORATION
TRAFFIC FLOW PLAN

PACIFIC GAS & ELECTRIC COMPANY
HUMBOLDT BAY POWER PLANT EUREKA, CALIFORNIA

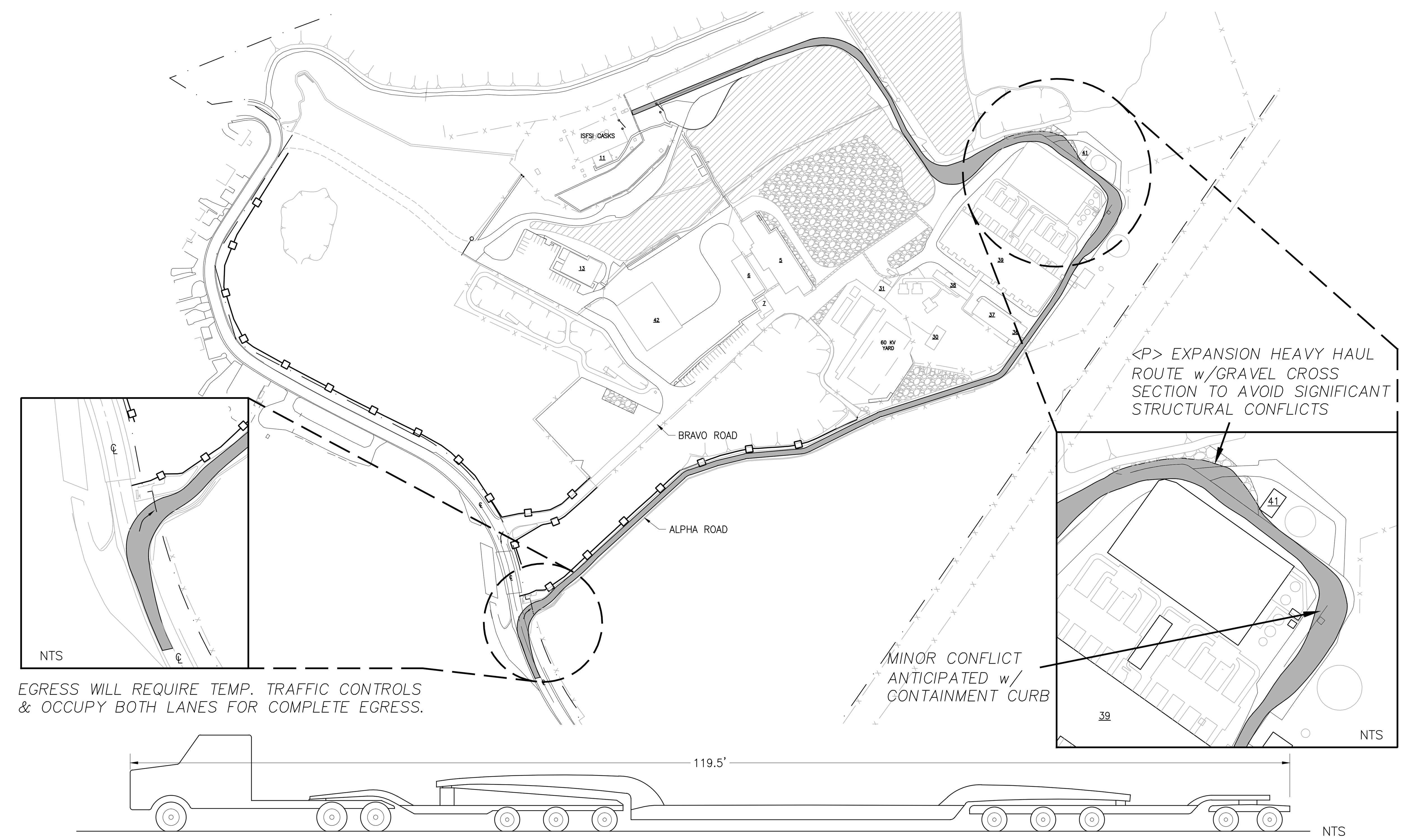
DWG SCALE:		
BILL OF MAT'L.:		
SUPSDS:		
SUPSD BY:		
DRAWING	SHEET	REV
4108682	5 of 6	H



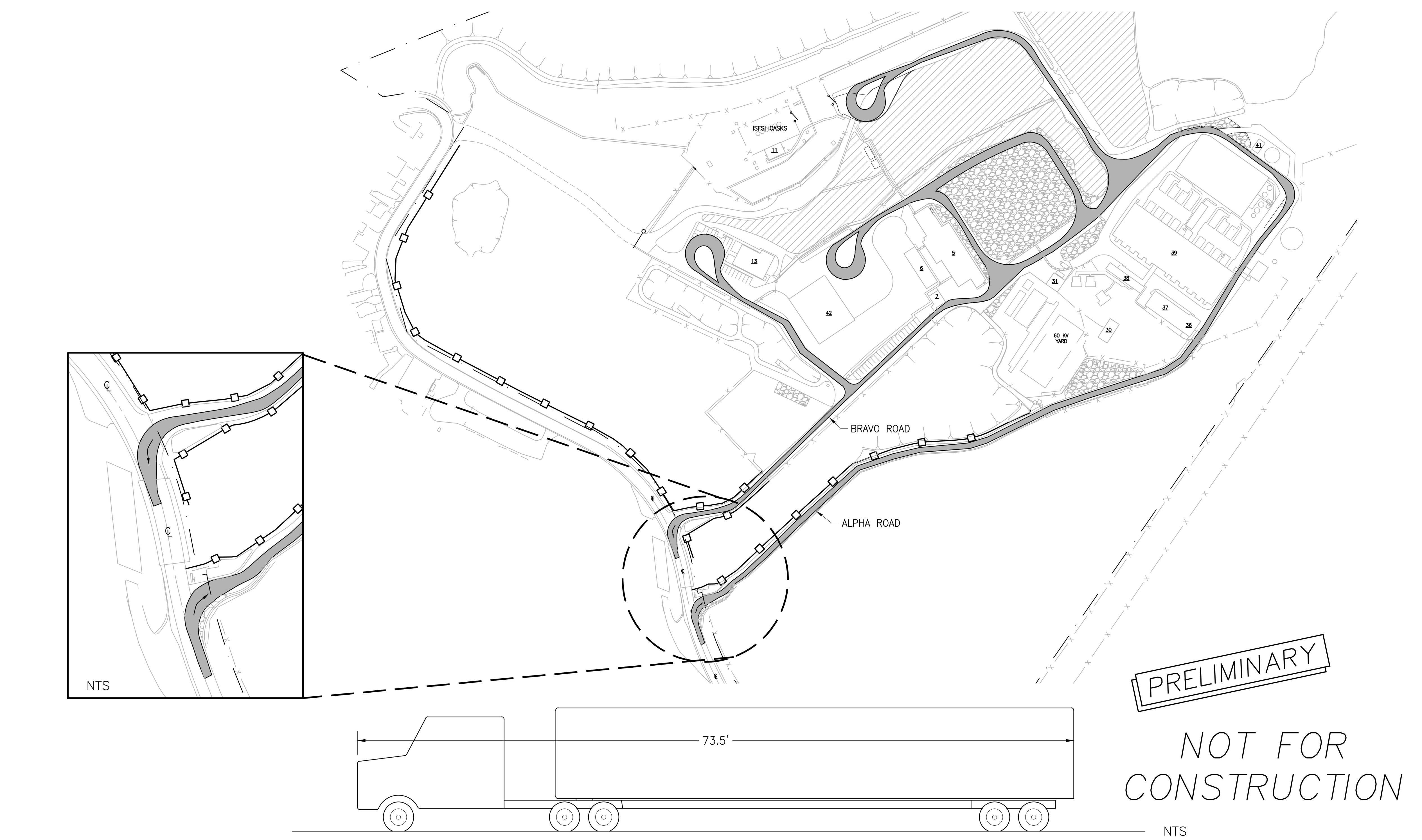
TRANSPORT #1 ALTERNATE – 10 LINE, 10' WIDE, SELF PROPELLED–MAX. EXTENTS OF OCCUPIED TRAVEL PATH



TRANSPORT #2 ALTERNATE – 8 LINE, 18' WIDE, THP–MAX. EXTENTS OF OCCUPIED TRAVEL PATH



TRANSPORT #3 ALTERNATE – 8' WIDE STEERABLE TRAILER–MAX. EXTENTS OF OCCUPIED TRAVEL PATH



WORST CASE AASHTO DESIGN TRUCK–MAX. EXTENTS OF OCCUPIED TRAVEL PATH

PRELIMINARY

NOT FOR CONSTRUCTION

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0 100' 200' 400'

THESE PLANS ARE ORIGINALLY PRINTED ON 24"x36" PAPER.

1 INCH

TRAFFIC FLOW PLAN

SCALE: 1"= 200'

TRUE NORTH — PLANT GRID

THESE PLANS ARE TO INFORM CONCEPTUAL LEVEL PLANNING OF TRANSPORT EFFORTS & REPRESENT THE TRANSPORT OPTIONS AVAILABLE AT THE TIME OF COMPOSITION. FINAL TRANSPORT OPTIONS WILL NEED TO BE RE EVALUATED IN GREATER DETAIL & INCLUDE CONSIDERATIONS BEYOND PG&E PROPERTY.

DATE	5-28-15
PR	BWW
CK	DPT
AP	JTL

REGISTERED PROFESSIONAL ENGINEER — BRADLEY WAYNE WILSON — NO. 83428 — CIVIL — STATE OF CALIFORNIA

REVISION DESCRIPTION

HBPP FINAL SITE RESTORATION
TRAFFIC FLOW PLAN

PACIFIC GAS & ELECTRIC COMPANY
HUMBOLDT BAY POWER PLANT EUREKA, CALIFORNIA

DWG SCALE:		
BILL OF MAT'L.:		
SUPSDS:		
SUPSD BY:		
DRAWING	SHEET	REV
4108682	6 of 6	H