DOCKETED

Docket Number:	01-AFC-19C
Project Title:	SMUD Cosumnes Power Plant - Compliance
TN #:	244288-50
Document Title:	1993 Geological Assessment from Rancho Seco ISFSI - Appendix 8_15A - 22 pages
Description:	N/A
Filer:	Patty Paul
Organization:	Ch2mhill/Carrier
Submitter Role:	Applicant Consultant
Submission Date:	8/1/2022 5:08:59 PM
Docketed Date:	8/1/2022

APPENDIX 8.15A

1993 Geological Assessment from Rancho Seco ISFSI

John Hicks

GEOTECHNICAL STUDY
FOR PROPOSED
INDEPENDENT SPENT FUEL STORAGE INSTALLATION
RANCHO SECO NUCLEAR GENERATING STATION
SACRAMENTO COUNTY, CALIFORNIA
FOR

SACRAMENTO MUNICIPAL UTILITY DISTRICT

JOB NO. ES-519/E306-01
JUNE 1, 1993

EGC

ENVIRONMENTAL GEOTECHNICAL CONSULTANTS, INC.

4229 NORTHGATE BOULEVARD, SUITE #3, SACRAMENTO, CALIFORNIA 95834
TELEPHONE (916) 925-4789 · FAX (916) 925-5973

Job No ES-519/E306-01
June 1, 1993

Sacramento Municipal Utility District
P.O. Box 15830 (Mail Stop 50)
Sacramento, California 95852-1830

Attention: Mr. John Bell

SUBJECT: Geotechnical Study for Proposed Independent Spent Fuel Storage Installation, Rancho Seco Nuclear Generating Station, Sacramento County, California

Dear Mr. Bell:

We have completed the geotechnical study for the proposed Independent Spent Nuclear Fuel Storage Installation (ISFSI) at Rancho Seco Nuclear Generating Station in Sacramento County, California. The accompanying geotechnical report presents the results of the subsurface exploration and our recommendations concerning the design and construction of earthworks and foundations for this project.

Per your request, the work was performed in general accordance with EGC's proposal EPS-881, dated February 8, 1993. In summary, the exploratory borings drilled at the site encountered natural clay and silt soils. The natural soils appeared suitable for support of the proposed structural mat. Recommendations are provided in the attached report.

We appreciate the opportunity to be of service to you in this project. If you have any questions concerning this report, or if we can be of further assistance, please contact EGC at your convenience.

REGISTERED PROFESSIONAL ENGINEER
DONALD R. POINDEXTER
GE-690
EXP 12/31/93
GEOTECHNICAL
STATE OF CALIFORNIA

Very truly yours

ENVIRONMENTAL GEOTECHNICAL CONSULTANTS, INC.

Reviewed by

Don R. Poindexter

Don R. Poindexter, P.E.
Geotechnical Engineer
GE-690, Exp. 12/31/93

John M. Phillips, P.E.
Senior Project Engineer

TABLE OF CONTENTS

List of Figures

1.0 INTRODUCTION

This report presents the results of our geotechnical study for the proposed ISFSI at Rancho Seco Nuclear Generating Station in Sacramento County, California (Figure 1). The purpose of this report is to describe the subsurface conditions encountered in the borings, analyze and evaluate test data and make recommendations regarding design and construction of earthworks, foundations and pavements.

1.1 Proposed Development

The site is located on the west side of the existing nuclear generating facilities at Rancho Seco Nuclear Generating Station, Sacramento County, California (Figures 1 and 2). The site slopes downward from west to east with an average slope of 12:1 (horizontal:vertical) and a maximum difference in elevation of approximately 35-feet. The site is covered with grass.

EGC understands that the proposed ISFSI will be a reinforced concrete structural mat with plan dimensions of approximately 225 by 175-feet. The central portion of the mat will be an area about 40-feet wide, oriented parallel to the sides of the mat and will have a thickness of approximately 2-feet. Up to 22 Horizontal Storage Modules (HSMs) will be stored back-to-back in two parallel rows running along the central portion of the mat. Each HSM is approximately 10-feet wide and 20-feet long, and weighs 250-tons fully loaded with spent fuel and shielding. In addition, two NUHOMS-MP187, multipurpose casks will be stored on the pad at a distance of approximately 20-feet from the HSMs. Each multi-purpose cask weighs 110-tons and has plan diameter of 8-feet.

An access road will be constructed to the ISFSI. We understand that previous practice at Rancho Seco is to convert existing railroad tracks to roads by topping with asphalt and leaving the railroad tracks in place. A portion of the access road will be constructed in this manner and will run along an existing railroad track before turning directly towards the ISFSI. The remainder of the road will be constructed on new fill. The road will support a loaded transporter with a total weight of about 135-tons. The remainder of the pad not occupied by the HSMs must support the transportation system.

The finished grade of the ISFSI will be 175-feet above Mean Sea Level (MSL) with a subgrade elevation of 173-feet above MSL. Cuts will be on the order of 20-feet at the west end of the central mat area and 10-feet at the eastern end. Maximum cut will occur near the northwest corner of the mat and will be on the order of 28-feet, while at the south eastern corner, cuts will be on the order of 6-feet. An earth berm will be constructed along portions of the southern side of the ISFSI. The berm will have a maximum height of about 20-feet. We also understand that minor sloughing of the cut slopes and berm followed by periodic maintenance is acceptable.

1.2 Scope of Work

The scope of work included performing soil borings on the subject property and obtaining soil samples for soils laboratory analysis. The results of the sampling and testing are presented in this report.

2.0 FIELD AND LABORATORY ASSESSMENT

A site visit was made by a representative of EGC on April 27, 1993 and as required by the above referenced proposal, two borings were drilled on the property to obtain relatively undisturbed samples of subsurface soils (see Figure 2). The borings were advanced using 8-inch diameter hollow stem augers powered by a rubber-tired, Mobile B-53 drill rig. Soils encountered were logged by a field engineer who observed all cuttings and collected soil samples. The soil samples were collected using a 2.5-inch O.D. California barrel sampler driven by blows from a 140 pound hammer free falling 30 inches per blow. Blows necessary to advance the sampler 1 foot were recorded on the field logs as blow counts. The blow counts are presented on exploratory boring logs (Figures 3 and 4). Values obtained using the California barrel sampler have been correlated to Standard Penetration Test (SPT) values and are used as a indicator of soil strength.

One 62-foot deep boring and one 75-foot deep boring were drilled at the west and east ends respectively, of the central portion of the proposed ISFSI.

The boring locations were laid out on the site by the EGC representative under the supervision of Mr. Wayne Hawley, SMUD. Distances from these locations to the reference features indicated in figure 2 are approximate and were measured by pacing. Right angles for the boring location measurements were estimated. Ground surface elevations indicated on the boring logs are approximate and were obtained by interpolation from plan contours. The locations and elevations of the borings should be considered accurate only to the degree implied by the means and methods used to define them. True surface elevations at these locations could differ due to interpolation and other differences could occur from superposing approximate boring locations on the topographic plan.

2.1 Soils Laboratory Testing

Selected samples were tested to determine in-place moisture content and dry density. These tests were conducted to quantify the physical characteristics of the site soils for engineering purposes. Moisture content and dry density laboratory test data are presented on the logs. A bulk sample was also obtained and subjected to Atterberg Limits Tests and particle size distribution analyses to determine the suitability of on-site soils as fill material. In addition, field testing of selected soil samples using a hand penetrometer were performed and these results are also presented on the logs. The hand penetrometer was used to provide supplemental information for classification purposes. The calibrated hand penetrometer has been correlated with unconfined compression tests and provides a better estimate of soil consistency than visual examination alone.

3.0 GEOLOGIC HAZARDS

Our studies included evaluation of a number of potentially hazardous geologic conditions. The following sections evaluate the relative levels of potential impact resulting from slope stability and seismic hazards.

3.1 Slope Stability

No obvious signs of instability were observed on the natural slopes in the area of the proposed ISFSI. The areas of potential slope instability are limited to the proposed cut slopes and earth berm. Sloping requirements are given below.

3.2 Seismicity

The subject site is considered subject to the probability of seismic activity. The following statements summarize the potential impact of the seismic setting upon development of the subject property.

No known active faults cross the area of the proposed development. The nearest fault system is the Foothill Fault System located approximately 10-miles east of the proposed ISFSI. The Foothill Fault System has been inactive since the Jurassic Period. The nearest known active faults are the Hayward and San Andreas Faults located approximately 70 and 90-miles west of the site, respectively.

The project site is susceptible to ground shaking in the event of an earthquake from the Hayward and San Andreas Faults. The seismic risk to a structure depends on the distance to the epicenter; the characteristics of the earthquake; the geologic, groundwater and soil conditions underlying the structure and its vicinity; and the nature of the construction. Ground shaking at the site is possible during the life of the project. Due to the sites distance from the above faults and the subsurface conditions, maximum ground accelerations would be on the order of 0.05 to 0.1g.

3.3 Ground Failure

Ruptures along the surface trace of a fault tend to occur along lines of previous faulting. Since the evidence indicates that a potentially active fault trace does not pass through the site, the primary hazard of surface rupture is negligible. The potential for seismically induced liquefaction was evaluated during our study and is discussed below.

3.4 Liquefaction Potential

Soil liquefaction is a phenomenon in which saturated cohesionless soils are subject to a temporary but essentially total loss of shear strength under the reversing cyclic shear stresses associated with earthquakes. Based on the above-mentioned seismic characteristics and geohydrological conditions discussed below, there is low potential for significant damage due to liquefaction.

4.0 SUBSURFACE SOIL CONDITIONS

Conditions encountered in each boring are indicated on the individual boring logs. Stratification boundaries on the boring logs represent the approximate location of changes in soil types as the transition between materials may be gradual. Based on the results of the borings, subsurface conditions on the project site can be generalized as follows.

Underlying an approximately 6-inch root zone in both borings, alternating layers of clays, sands and silts were encountered. In boring B-1, a layer of silty clay extended to a depth of about 10.5-feet below grade where it was underlain by a hardpan layer with a thickness of approximately 2.5-feet. The hardpan was not encountered in boring B-2. Beneath the root zone in boring B-2, a layer of sandy clay extended to a depth of about 9-feet below grade. Underlying the sandy clay in boring B-2 and the hardpan in boring B-1, a sand layer was encountered which varied in thickness from 3 to 7-feet. Based on a finished subgrade elevation of 173-feet above MSL, the bottom of mat of the ISFSI would occur at roughly the same elevation as the sand layer.

The sand layer was underlain in both borings by a 11.5 to 16.5-feet thick layer of sandy silt. A second sand layer occurred beneath the sandy silt, and varied in thickness from 4.5 to 7-feet. Beneath the second sand layer, layers of silt and sandy clay soils extended to the termination depth of 62-feet in boring B-1 In boring B-2, layers of silt, sand and clay extended to the termination depth of 75-feet below grade.

The clay and silt soils exhibited dry densities ranging from 77 to 87 pounds per cubic foot and moisture contents from about 12 to 18 percent. Their consistencies were typically hard, with blow counts ranging from 50/5-inches to 25/0.5-inch. The sands and gravel exhibited relative densities that were typically very dense, with the exception of the first sand layer in boring B-2 which was medium dense. The medium dense result may be due in part to the presence of a limited amount of perched water in the sand layer which is discussed below.

5.0 GROUNDWATER OBSERVATIONS

The borings were monitored while drilling and after completion of the borings to observe the presence of groundwater. Groundwater was not encountered in the borings. According to Department of Water Resources Maps, available at Sacramento County Environmental Management Department (SCEMD) offices and reviewed as part of this study, groundwater in the area of Rancho Seco is on the order of 150-feet below grade.

However, short-term perched water conditions can occur due to seasonal variations in the amount of rainfall, runoff, and other factors not evident at the time the borings were performed. Perched groundwater conditions may occur near the lower elevations of the existing slopes and should be considered when developing the design and construction plans for the project.

6.0 DISCUSSION AND RECOMMENDATIONS

6.1 General

In general the primary design considerations for this project are as follows:

Bearing capacity and settlement criteria for structural mat, as well as mat thickness, reinforcing, subgrade preparation and modulus of subgrade reaction (k).

Slope stability of cut slopes, erosion control, vegetation and construction.

Slope stability of berms, compaction, erosion, vegetation and keying/benching into underlying natural slopes.

Access road construction, preparation of subgrade and compaction of sub-base.

Site drainage.

The following sections address the above listed issues in greater detail.

6.2 Site Preparation and Grading

Site preparation for the project should include removal of any vegetation, organic soil and otherwise unsuitable material from the proposed structural mat, berm and access road areas. We anticipate a stripping depth of about 6 to 9 inches will probably be required to remove the root zone in the project area. Deeper stripping depths may be required in localized areas where organic soils or fill are present. The stripping depth should be evaluated at the time of construction by the geotechnical engineer or his representative.

Prior to placement of fill in areas below design grade and after rough grading is completed in other areas, the subgrade should be proof-rolled. Proof-rolling aids in providing a firm base for compaction of fill and delineating soft and disturbed areas that may exist below subgrade level. Unsuitable areas observed at this time should be improved by compaction or by undercutting and placement of suitable, compacted fill. Proof-rolling may be accomplished with a fully loaded tandem-axle dump truck or other equipment providing an equivalent subgrade loading. A minimum gross weight of 25 tons is recommended for the proof-rolling equipment.

Fill placed within structure areas and areas to be paved should consist of approved materials which are free of organic matter and debris. The fill should be placed and compacted in lifts of 8 inches or less in loose thickness. Fill placed for support of the structural mat or access roads should be compacted to at least 95 percent of the material's maximum modified Proctor dry density in accordance with ASTM D 1557-78 test procedures. Fill should be placed at a moisture content within 3 per cent of the materials optimum moisture content as defined by the modified Proctor test. Based on Atterberg Limits Test results and observation of the on-site soils, the on-site soils appear suitable for use as fill. Results of tests performed on the on-site material, revealed the following properties

Maximum Modified Proctor dry density - 104.5 pcf

Optimum moisture content - 19.5 per cent

Plasticity Index - 10.6

- Per cent passing #200 sieve - 53 per cent

6.3 Structural Mat Support

Based on the results of the subsurface exploration, it appears that the natural soils could be used for support of the structural mat for the proposed ISFSI. A mat bearing on dense sand or very stiff clay or silt, natural soil or on new fill extending to suitable natural soils could be designed using a maximum net allowable soil bearing pressure of 4000 pounds per square foot. This is the maximum pressure that should be transmitted to the bearing soils in excess of the minimum surrounding over-burden pressure. The bearing capacity can be increased by 1/3 for short wind or seismic loads. A vertical subgrade modulus of 300 psf can be used for design of the structural mat.

The edge of the structural mat should be embedded a minimum of 1-foot below outside grade. Lateral loads may be resisted by a passive pressure equivalent to that generated by a fluid weighing 300 pcf.

For a mat designed in accordance with the above recommendations, we estimate total settlement should be about 1.5 inches or less. Differential settlement of 1/2 to 2/3 of the total settlement is possible.

Bearing soils in all excavations should be observed by the geotechnical engineer during construction. If loose or soft soil is present at or below bottom of structural mat bearing level, corrective measures will be required which would include over-excavation and backfill with controlled structural fill. It may also be possible to densify loose sands in-place by means of vibratory compaction equipment.

The over-excavation and backfill treatment should consist of over-excavating below design footing level to very stiff or dense natural soil. Excavation should be extended laterally at least 5-feet beyond the edge of the mat in all directions. New fill should be placed and compacted in lifts no greater than 8 inches in loose thickness and the water content should be near the optimum value determined by the modified Proctor test. Fill should be compacted to at least 95 percent of the material's maximum modified Proctor dry density.

The base of the mat excavation should be free of water and loose soil prior to placing concrete. Concrete should be placed as soon as possible after excavation to minimize bearing soil disturbance. Should the soils at bearing level become disturbed, the affected soils should be removed or compacted prior to placing concrete.

Should a perched groundwater table occur during construction, some water seepage in the excavation could occur. Any seepage into the anticipated excavation should be minor and dewatering, if required, should be possible with sump pits and pumps.

6.4 Mat Thickness and Reinforcement

The central portion of the structural mat should have a minimum thickness of 2-feet. Remaining portions of the mat should have a minimum 1-foot thickness. Reinforcing should consist at a minimum of #4 bars spaced at 18-inches in both directions, top and bottom. Review of these design guidelines which are made based on soil conditions, should be performed by a licensed structural engineer.

6.5 Slope Stability: Cut Slopes

Utilizing a limit analysis of slope stability and soil strength values obtained from test data, and a safety factor calculated on the basis of critical slope height, cut slopes with an inclination of 1.5:1 (horizontal:vertical) would have a factor of safety of at least 1.5. Minor sloughing of the slopes may occur and periodic maintenance will be required. In areas where the cut slope exposes layers of the sand soils encountered during the subsurface exploration, the slope should be observed by the geotechnical engineer and may require flattening to a 2:1 slope.

6.6 Slope Stability: Berm

Based on a limit analysis of slope stability, the soil types encountered and test data, berm slopes constructed in accordance with the recommendations of this report and with an inclination of 1.5:1 (horizontal:vertical) would have a factor of safety of at least 1.5. The slopes should be overbuilt a minimum of 2-feet and then cutback to their final configuration. Fill placed in the berm should be placed in lifts of a maximum 8-inch loose thickness and compacted to at least 90 per cent of the materials maximum modified Proctor dry density. Minor sloughing of the slopes may occur and periodic maintenance will be required.

Prior to construction of the berm, the underlying soils should be prepared following the procedures outlined in section 6.2 above and scarified to a depth of at least 9-inches and recompacted. In areas where the natural slopes are steeper than 6:1, the natural slope should be benched before construction of the berm. Benching would consist of cutting steps, no higher than 2-feet and approximately as wide as the blade of the construction equipment used to construct the bench.

6.7 Pavements

Detailed pavement design including minimum section recommendations based upon R-value tests were beyond the scope of work for this project; However, following are general guidelines for fill placement within pavement areas.

Fill placed within 12 inches of pavement subgrade level should be compacted to a minimum of 95 percent of maximum modified Proctor dry density (ASTM D-1557). Fill placed below this level should be compacted to a minimum of 90 percent of the materials maximum modified Proctor dry density. In cut areas or transitional cut/fill areas near final grade, the subgrade should be scarified and recompacted to at least 95 percent of maximum modified Proctor dry density to a depth of at least 12 inches. Fill should extend a minimum of 5-feet or at a slope of 3:1 (horizontal:vertical), whichever is greater, from edge of access roads.

Pavements should be sloped to provide rapid drainage of surface water. Water allowed to pond on or adjacent to the pavement could saturate the subgrade and contribute to premature pavement deterioration.

6.7 Site Drainage

Positive drainage away from the structural mat should be provided for a distance of at least 5-feet in all directions. A toe drain should be installed at the toe of all cut and berm slopes to provide positive drainage away from the excavated area. The toe drain should consist of at a minimum, an 18-inch wide, 3-foot deep trench, with a 6-inch diameter ABS SDR 23.5 or equivalent perforated pipe placed in the center of the trench and approximately 6-inches above the bottom of the trench. The trench should be backfilled with 1/2 to 3/4-inch, clean drain rock. The trench should be lined with a suitable non-woven geotextile. The drain pipe should be placed with the perforations downward and with a minimum slope of one per cent toward the outlet side for flow. All pavements and the structural mat should be sloped to provide drainage

7.0 LIMITATIONS

This report has been prepared for the exclusive use of Sacramento Municipal Utility District (SMUD) and their consultants for the specific application to the proposed development. In the event that any changes in the nature of design location or configuration to the proposed development, the conclusions and recommendations contained herein shall not be considered valid unless the changes are reviewed or revised by our firm. Once the changes have been reviewed additional confirming borings may be necessary.

The analysis, opinions, conclusions and recommendations submitted in this report are based in part on the referenced materials, site reconnaissance and subsurface exploration. The nature and extent of variation among exploratory borings may not become evident until construction. If variations then appear, it will be necessary to re-evaluate or revise recommendations made in this report.

The recommendations presented in this report are contingent on an adequate testing and monitoring program during construction of the proposed development. Unless the construction monitoring and testing program is provided by or coordinated with our firm, Environmental Geotechnical Consultants, Inc. will not be held responsible for compliance with design recommendations presented in this report and other supplemental reports submitted as part of this report.

Our services have been provided in accordance with generally accepted geotechnical engineering practices. No warranties are made, express or implied, as to the professional opinions or advice provided. Recommendations contained in this report are valid for a period of one year, after which time they must be reviewed by this firm to determine whether or not they are still applicable.

8.0 REFERENCES

Sacramento Municipal Utility District, Rancho Seco ISFSI Environmental Report, Revision 1, April 9, 1993

Geocon Inc., Geotechnical Engineering Investigation, ISFSI, Rancho Seco Nuclear Generating Station, August 1991

Sacramento Municipal Utility District, Rancho Seco ISFSI Design Criteria, Revision 1, March 10, 1993

9.0 GENERAL NOTES

DESCRIPTIVE SOIL CLASSIFICATION:

Soil Classification is based on the Unified Soil Classification System and ASTM Designations D-2487 and D-2488. Coarse Grained Soils have more than 50% of their dry weight retained on a #200 sieve; they are described as: boulders, cobbles, gravel or sand. Fine Grained Soils have less than 50% of their dry weight retained on a #200 sieve; they are described as: clays, if they are plastic, and silts if they are slightly plastic or non-plastic. Major constituents may be added as modifiers and minor constituents may be added according to the relative proportions based on grain size. In addition to gradation, coarse grained soils are defined on the basis of their relative in-place density and fine grained soils on the basis of their consistency. Example: Clay with sand, trace gravel, stiff (CL); silty sand, trace gravel, medium dense (SM).

CONSISTENCY OF FINE-GRAINED SOILS:

Unconfined Compressive Strength, Qu, psf	Consistency
< 500	Very Soft
500 - 1,000	Soft
1,001 - 2,000	Medium
2,001 - 4,000	Stiff
4,001 - 8,000	Very Stiff
8,001 - 16,000	Hard
> - 16,000	Very Hard

RELATIVE DENSITY OF COARSE-GRAINED SOILS:

N-Blows/ft.	Relative Density
0-3	Very Loose
4-9	Loose
10-29	Medium Dense
30-49	Dense
50-80	Very Dense
80+	Extremely Dense

RELATIVE PROPORTIONS OF SAND AND GRAVEL:

Descriptive Term(s) (of Components Also Present in Sample)	Percent of Dry Weight
Trace	< 15
With	15 - 29
Modifier	> 30

RELATIVE PROPORTIONS OF FINES:

Descriptive Term(s) (of Components Also Present in Sample)	Percent of Dry Weight
Trace	< 5
With	5 - 12
Modifier	> 12

GRAIN SIZE TERMINOLOGY:

Major Component of Sample	Size Range
Boulders	Over 12 in. (300mm)
Cobbles	12 in. to 3 in. (300mm) to 75mm)
Gravel	3 in. to #4 sieve (75mm to 4.75mm)
Sand	#4 to #200 sieve (4.75mm to 0.075mm)
Silt or Clay	Passing #200 sieve (0.075mm)