

**FINAL REPORT
GEOTECHNICAL INVESTIGATION
OF ALTERNATE INDEPENDENT
SPENT FUEL STORAGE INSTALLATION (ISFSI)**

**Prepared for
Nuclear Engineering Design Organization
Civil Engineering**

**By
Southern California Edison Company
E P E & C Geotechnical Group**

November 1995

GEOTECHNICAL INVESTIGATION OF ALTERNATE INDEPENDENT SPENT FUEL STORAGE INSTALLATION (ISFSI)

CONTENTS

PAGE NO.

EXECUTIVE SUMMARY

PROJECT DESCRIPTION-----2

SECTION 1.0 - SITE CHARACTERIZATION

1.1	INTRODUCTION	2
1.2	PROJECT DESCRIPTION AND SITE CONDITIONS	2
1.3	FIELD AND LABORATORY TESTING	3
1.4	SUBSURFACE CHARACTERIZATION	4
1.5	CONCLUSIONS	4

SECTION 2.0 - TSUNAMI

2.1	INTRODUCTION	4
2.2	TSUNAMI CALCULATIONS	4
2.2.1	SONGS 1 CALCULATIONS	4
2.2.2	SONGS 2 & 3 CALCULATIONS	5
2.3	MAXIMUM WAVE RUNUP	5
2.4	CONCLUSIONS	6

SECTION 3.0 - DESIGN BASIS EARTHQUAKE

3.1	INTRODUCTION	6
3.2	DBE RESPONSE SPECTRA	7
3.2.1	PROXIMITY TO EXISTING FACILITIES	7
3.2.2	PREVIOUS DESIGN CRITERIA	7
3.2.3	SITE CHARACTERIZATION	7
3.3	AVERAGE RETURN PERIOD VALUES ASSOCIATED WITH THE DBE	8
3.4	CONCLUSIONS	8

SECTION 4.0 - SLOPE STABILITY

4.1	INTRODUCTION	8
4.2	SITE (SLOPE) CONDITIONS	8
4.3	APPROACH	9
4.4	RESULTS	10
4.5	CONCLUSIONS	11

SECTION 5.0 - LIQUEFACTION

5.1	INTRODUCTION	11
5.2	SITE CONDITIONS	12
5.3	APPROACH	12
5.4	RESULTS	13
5.5	CONCLUSIONS	13

APPENDIX A

SECTION 6.0 - REFERENCES

6.1	SITE CHARACTERIZATION	14
6.2	TSUNAMI	14
6.3	DBE RESPONSE SPECTRA	15
6.4	SLOPE STABILITY	15
6.5	LIQUEFACTION	15

APPENDIX A BORING LOGS - FIELD INVESTIGATION

APPENDIX B GRADATION CURVES (LABORATORY TESTING)

FIGURES

FINAL REPORT
GEOTECHNICAL INVESTIGATION OF ALTERNATE
INDEPENDENT SPENT FUEL STORAGE INSTALLATION (ISFSI)

EXECUTIVE SUMMARY

The purpose of this study was to evaluate the geotechnical aspects of siting an Independent Spent Fuel Storage Installation at San Onofre Nuclear Generation Station in the area of the existing Unit 1. The characteristics evaluated included Site Characterization, Evaluation of Design Ground Motion, Tsunami Potential, Slope Stability Analysis, and Liquefaction Potential. The selection of each of these topics was based on a review of the USNRC Information Notice 95-28 which discusses the emplacement of spent fuel dry storage cask pads.

The information discussed in the report was derived from both historic reports as well as current, ongoing studies including data collected specifically for this study. The report has been organized with each topic as a separate Section: 1.0 through 5.0. Section 6.0 includes all references for the report. Following Section 6 are Figures (no figures for Section 2.0, Tsunami), Appendices A, Boring Logs - Field Investigation, B, Gradation Curves (Laboratory Testing), and Figures 1.1 through 4.3 (Figure No. correspond to the pertinent section).

Based on the findings of this report, there are no limiting geotechnical factors in the siting of an ISFSI at this proposed location. The specific conclusions from the study are as follows:

- Site Characterization - All the data collected during this study are consistent with the information gathered at the site over the past 25-30 years. The results of the comparison indicate that the previous extensive characterization efforts can be reasonably applied to this project for siting of the ISFSI.
- Tsunami - Based on the maximum breaking wave height calculated, it is estimated that any wave uprush will be intercepted by the intervening plant facilities, and will have no effect on the cask or pads.
- Design Basis Earthquake - The DBE response spectra with a zero period acceleration of 0.67g provides a reasonable design basis for the ISFSI.
- Slope Stability - Based on the Makdisi-Seed procedure for calculating permanent slope displacement, it is estimated that only minor sloughing of the surficial slope materials is likely to occur during the DBE-level ground shaking. This will not adversely affect the ISFSI project which is located greater than about 60 feet from the toe of the slope.
- Liquefaction - The San Mateo sand will not experience liquefaction during the DBE-level shaking. This finding is consistent with all previous findings for the San Mateo sand.

PROJECT DESCRIPTION

An Independent Spent Fuel Storage Installation (ISFSI) is proposed for the San Onofre Nuclear Generating Station to be located immediately adjacent to the northeast corner of Unit 1. The proposed location is east of Unit 1 Diesel Generating Building and north of the AWS Building. The facility would be used to house spent fuel casks which would be stored on concrete pads.

The purpose of the this study was to evaluate several geotechnical aspects of siting the ISFSI at this location. The specific geotechnical topics include Site Characterization, Design Ground Motion, Tsunami Potential, Slope Stability and Liquefaction Potential. These topics were selected based on a review of the USNRC Information Notice 95-28 which discusses the emplacement of spent fuel dry storage cask pads.

In order to prepare this report, a number of sources were utilized. These included additional field work and laboratory analysis, discussions with consultants, and review of existing geotechnical information and reports appropriate to these topics.

Based on the evaluation of each of the topics, there are no limiting geotechnical factors which would preclude siting of an ISFSI at this proposed location.

1.0 SITE CHARACTERIZATION

1.1 INTRODUCTION

This section summarizes a geotechnical investigation conducted to characterize the subsurface conditions for the proposed Independent Spent Fuel Storage Installation (ISFSI) at the San Onofre Nuclear Generating Station. The evaluation included a review of SONGS Unit 1 project files related to subsurface conditions, drilling of two rotary wash borings, laboratory testing, and direct comparisons with previous characterization efforts for facilities at Unit 1 and Units 2&3.

1.2 PROJECT DESCRIPTION AND SITE CONDITIONS

The project is located east of the Unit 1 Diesel Generating Building and north of the AWS Building as shown on the attached aerial photo, Figure 1.0. The proposed slab configuration for the proposed ISFSI is shown on Figure 1.1. The surface topography in the project area is generally flat, at Elev. +20 feet, and slopes slightly toward the Pacific Ocean to the west. However, the project is located near the base of existing cut slopes made during construction of Unit 1 with the closest distance between the base of cut slopes and the edge of the cask storage foundation slab to be about 60 feet. The height of the cut slopes above the Unit 1 site grade at the location of the proposed cask storage slabs ranges from about 65 to 85 feet and the slope ratio is about 1/2:1 (horizontal:vertical) although near-horizontal benches typically exist at various locations along the slopes. As shown on Figure 1.1, the toe of the slopes, in plan, is generally

straight for cut slopes located to the east of the proposed cask storage slabs but the footprint of the slope is both concave and convex for slopes that exist to the north.

Previous borings made in the general vicinity to the proposed ISFSI include fourteen borings by Southern California Edison for Unit 1 investigations and two borings for the Unit 1 Diesel Generating Building (Woodward-Clyde Consultants 1975). Detailed mapping of the cut slopes to the east was performed as part of the geotechnical study for the AWS Building (Woodward-McNeill 1974b).

Sands of the San Mateo formation are known to be the foundation material throughout the main plant area at SONGS. Locally, San Mateo sand underlies terrace deposits below elevations of about 50 to 60 feet and extending down to an estimated elevation of -850 feet. The San Mateo formation is of Pleistocene age and the upper units generally consist of a very dense, well-graded sand with apparent cohesion due to efficiency in grain-packing. The physical and engineering properties of this material have been studied extensively (e.g., Woodward-McNeill 1973; Woodward-McNeill 1974a; and Woodward-McNeill 1974b).

Numerous underground utilities are reported to exist within the project area, including a 42-inch-diameter storm drain pipe. Therefore, compacted backfill is expected to exist in trenches along utility lines. Structural backfill is known to exist around the control/administration building to the southeast of the Unit 1 containment sphere and various buildings to the south and west of the Unit 1 Containment Building as well as the Diesel Storage Tanks and the 42" storm drain pipe.

1.3 FIELD AND LABORATORY TESTING

A field investigation was conducted at the project site on September 23, 1995. The field work consisted of drilling two borings using mud-rotary wash techniques to depths of 57.5 and 17.3 feet at locations shown on Figure 1.1. A Standard Penetration Test (SPT) was performed at five-foot intervals to measure driving resistance and obtain samples for laboratory testing. The boring logs and a detailed description of the SPT are presented in Appendix A.

The borings were made through hand-dug "potholes" which extended to depths of about 6 feet beneath the plant grade under the direction of Bechtel Construction personnel. These potholes were used to avoid encountering buried utility lines. A third boring was begun at the location shown on Figure 1.1 but was abandoned because gravel fill was encountered to a depth of 6-1/2 feet suggesting that a utility line might be present.

Soil samples obtained from the field investigation were tested to measure gradation and moisture content. The results of the laboratory tests are presented in Appendix B. For convenience, some of the laboratory test results are also indicated on the boring logs presented in Appendix A.

1.4 SUBSURFACE CHARACTERIZATION

San Mateo sand was encountered below asphaltic pavement and base coarse fill to the full depth of the two borings. A comparison among seven gradation tests performed on SPT samples and a typical range developed for San Mateo sand from previous studies is shown on Figure 1.2. Some of the new samples have a trace of gravel and a slightly larger percentage of fines than previously measured but overall the comparison is close. The close comparison between the typical range and the new data shown on Figure 1.2 indicates that the foundation material is San Mateo sand.

1.5 CONCLUSIONS

The SPT blowcounts were very high, typically being 70 blows for 2 to 5 inches of seating penetration. These blowcounts confirm that the San Mateo sand is in a very dense, compact condition. These blowcounts are fairly consistent with the range of SPT blowcounts measured previously in San Mateo sand. The results of the comparison of gradations and blowcounts indicate that previous extensive characterization efforts for San Mateo sand can be reasonably applied to this project.

Groundwater was encountered at a depth of about 18 feet (estimated elev. + 2 ft) in boring B-1. This level is consistent with recent groundwater measurements made at the plant by others (1995, verbal communication, H. Gene Hawkins). The groundwater is known to fluctuate between elev. + 2 and +6 feet.

2.0 TSUNAMI

2.1 INTRODUCTION

During the past 30 years, several studies have been performed to investigate the potential for tsunami effects on the SONGS area (Marine Advisers, 1965, 1966; Wilson, 1972; Woodward Clyde Consultants, 1986). The scope of these studies included: an evaluation of the susceptibility of coastal southern California to tsunamis, an analysis of the magnitude of potential tsunami at SONGS, and an estimate of wave forces on the SONGS 1 seawall.

2.2 TSUNAMI CALCULATIONS

2.2.1 SONGS 1 CALCULATIONS

In their 1965 report, Marine Advisers discussed tsunami potential from both remote and local origins. Their results indicated that the maximum height of tsunami run-up would be on the order of 6 ft., which exceeds the highest ever recorded along coastal southern California. By combining tsunami effects, tide and storm surge, they estimated that the maximum run-up elevation would be on the order of +13 feet, mllw. For tsunamis of local origin, they based their analysis on local borderland faulting and landsliding.

They noted that a magnitude 8.25 earthquake (1906) on the submarine extension of the San Andreas Fault did not cause any measurable tsunami effects. They concluded that local tsunamis would not exceed remote tsunamis. Therefore, the design basis of +13 feet mllw for the run-up would be conservative.

2.2.2 SONGS 2&3 CALCULATIONS

In 1972, after general agreement had been reached on the SONGS 2&3 site seismicity, Dr. Basil Wilson investigated a tsunami generated by a fault rupture along the offshore zone of deformation. He based his study on a magnitude 7.5 earthquake occurring 5 miles offshore along the shelf edge. He further assumed that the vertical component of ground displacement would be about 7.1 ft. Based on these parameters he estimated that the maximum still water run-up likely to occur at SONGS would be 15.6 ft. mllw. This includes a tsunami run-up of 7.6 feet, a 7-foot extreme high tide, and a 1-foot storm surge. The tsunami generated crest is referred to as the maximum still water run-up because the period of the tsunami wave is extremely long resulting in a slow rising, nonbreaking water surface.

2.3 MAXIMUM WAVE RUNUP

To determine the worst-case effects of a tsunami on the project, no credit was taken for the existing seawall. It was assumed that the seawall was not present, but that the configuration of the existing structures would remain much as they are today.

The proposed ISFSI is located on the east side of the site as shown on Figure 1. Lying between it and the ocean are other plant facilities such as the containment building, power block and the plant perimeter security wall. These structures, along with other plant facilities located between the ocean and the ISFSI site, would provide a barrier to the wave runup if it were possible for the upsurge to reach this far inland. However, based on the calculations and site configuration, it is not possible for the wave runup to reach the ISFSI site. As stated earlier, the maximum still water elevation of the tsunami is +15.6 ft. mllw. Plant grade of the ISFSI site is about +20 ft. mllw. The cask foundation pad would therefore be about 4.4 feet higher than the proposed wave runup. There is no threat to the casks and pads from inundation by the tsunami.

To assess the potential dynamic effects of wave action, storm waves were superimposed on the tsunami-generated still water run-up. Combining a low probability-of-occurrence storm wave with the other extreme events associated with the tsunami results in a maximum incredible wave.

From previous calculations it was determined that the maximum breaking wave that could impact on the plant occurs at the beach walkway. The walkway is raised above the beach and is protected on the seaward side by a rock riprap embankment. The depth of water at the toe of the riprap, used to calculate the maximum breaking wave, is 8.6 feet and the depth of water at the top of the beach walkway is 1.6 feet. The maximum breaking wave height is a function of water depth. Therefore, the highest wave will break at this location, that is, at the toe of the riprap. It was determined in the SONGS 1 seawall calculations (SCE, Design Calculations) that the maximum breaking wave to impinge upon the seawall was 8.8 feet high. This is the highest wave that can break at this location based on the shoreline geometry. The distance between the walkway and the pad site is about 425 feet and the elevation of the pad is 6 feet higher than the walkway. Assuming that the seawall is no longer present, the energy and momentum of the braking wave will be dissipated as it moves toward the ISFSI location. The wave will be dissipated by distance, elevation and the intervening plant facilities that will interfere with the runup. A mass balance was used to determine the height of wave runup from a breaking wave at the riprap. A volumetric calculation of the wedge of water with a height of 8.8 feet by 66 feet long (at right angles to the shore direction) (WCC, 1986), would fill the area from the riprap to the site with water up to elevation 18.8 ft. mllw which is 1.2 ft. below plant grade for the ISFSI slab.

2.4 CONCLUSIONS

In summary, previous reports and calculations indicate that the maximum run-up on the seawall from a tsunami with superimposed storm waves will be less than 27 ft. Assuming that the seawall is not present during a tsunami plus a storm runup, the maximum height of water entering the plant area would be to elevation 18.8 ft. mllw. This is 1.2 ft. below ISFSI plant grade. Therefore the tsunami with superimposed storm waves has no impact on the project.

3.0 DESIGN BASIS EARTHQUAKE

3.1 INTRODUCTION

This section presents the basis for the appropriateness of the Design Basis Earthquake (DBE) response spectra for the SONGS Units 2 and 3 as the design response spectra for the ISFSI. Average return period values associated with the DBE response spectra are based on the recent Individual Plant Examination of External Events (IPEEE) ground motion study (Risk Engineering, 1994). The results are then evaluated for the appropriateness of the DBE in comparison with previous studies done at SONGS.

3.2 DBE RESPONSE SPECTRA

The DBE response spectrum for SONGS Units 2 and 3 at a peak ground acceleration of 0.67g previously developed for 2 percent damping is shown on Figure 3.1. The corresponding response spectra for other damping values were obtained, also previously, by scaling the plateau of the 2 percent response spectrum (i.e., the constant spectral acceleration for period range of 0.2 to 1 second) by a scaling factor as recommended by Newmark and Hall (1969). The resulting DBE response spectra for various other damping values are also shown on Figure 3.1. The following sections discuss the appropriateness of the response spectrum based on proximity to the existing facilities, distance to the earthquake source and the nature of the foundation materials.

3.2.1 PROXIMITY TO EXISTING FACILITIES

The proposed site is located within the existing SONGS 1 area and about 200 yards north of the existing SONGS 2/3 facilities. The proposed site would be located on the same foundation materials as Units 1, 2 and 3, and about the same elevation. Studies conducted since the early 1960's indicate that there are no significant differences between the location of the various units. In fact, over time, the more conservative design for SONGS 2/3 was adopted for SONGS 1 during its' reevaluation in the 1970's and 1980's. So, the seismic criteria for all three units is essentially the same. That is, the seismic criteria are based on a major earthquake, M=7, located some 5 miles offshore. The zero-period acceleration for all three units is now at 0.67g.

3.2.2 PREVIOUS DESIGN CRITERIA

A major portion of the design criteria for SONGS 2/3 was an evaluation of the seismic criteria in accordance with 10CFR100 Appendix A. These criteria were revisited a number of times over the past twenty years to insure that there have been no significant changes to the geologic understanding of the area. As detailed in the Safety Evaluation Report (USNRC, 1980), the DBE (SSE) is based on a zone of deformation about 5 miles offshore "which extends from the Newport-Inglewood fault zone to the north, to the Rose Canyon fault zone to the south." (USNRC, p. 2.5-1) The zero-period acceleration is equal to 0.67g. Subsequent studies done at SONGS both by SCE and others continue to support those conclusions.

3.2.3 SITE CHARACTERIZATION

Section 1.0, Site Characterization, of this report concluded that the earlier characterization of the site, including its' foundation materials was essentially unchanged during the past 30 years of construction and

operation at SONGS. The proposed site should perform much the same throughout the life of the new facilities.

3.3 AVERAGE RETURN PERIOD VALUES ASSOCIATED WITH THE DBE

Figure 3.2 shows hazard curves at 5 percent damping calculated for two periods, 0.033 second (or peak ground acceleration) and 1 second, in the recent IPEEE study (Risk Engineering, 1994) for the site. Using these figures, it is possible to infer a value of average return period associated with a given spectral acceleration value at a given period. In particular, Figure 3.2 indicates values of average return period associated with the DBE response spectrum at 5 percent damping at periods of 0.033 second and 1 second based on the 5 percent DBE spectral values at these periods shown on Figure 3.1. The two data points shown on Figure 3.2 translate into the two data points shown on Figure 3.3, which shows the values of average return period associated with the DBE response spectrum as a function of period.

3.5 CONCLUSIONS

It is concluded that the DBE response spectra with a zero period acceleration of 0.67g provides a reasonable design basis for the ISFSI. This conclusion is based on the following observations:

- The proposed site is located on the existing property of SONGS 1,2&3 which utilized a 2/3g design response spectrum.
- .67g was considered a reasonably conservative design value for the previous units.
- There have been essentially no changes to the site during the past 30 years with respect to soil type, distance to faults or site response.
- The return period for the DBE would exceed about 3300 years.

4.0 SLOPE STABILITY

4.1 INTRODUCTION

This section summarizes a study to estimate permanent displacements in cut slopes in the vicinity of the proposed Cask Storage Project that could result from ground shaking associated with the Design Basis Earthquake (DBE). The evaluation included using the results of the site characterization study, a review of SONGS Unit 1 project files related to idealized slope and subsurface conditions, and an analysis to estimate permanent slope displacement using a simplified method of analysis.

4.2 SITE (SLOPE) CONDITIONS

The proposed slab configuration for the proposed Cask Storage Project is shown on Figure 1.1. The project is located east of the Unit 1 Diesel Generating Building and north of the AWS Building as shown on Figure 1. The surface topography in the plant area is generally flat, at Elev. 20 feet, and sloped slightly

toward the Pacific Ocean to the west. However, the project is located near the base of existing cut slopes made during construction of Unit 1 with the closest distance between the base of cut slopes and the edge of the cask storage foundation slab to be about 60 feet. The height of the cut slopes above the Unit 1 site grade at the location of the proposed cask storage slabs ranges from about 65 to 85 feet and the slope ratio is about 1/2:1 (horizontal-vertical) although near-horizontal benches typically exist at various locations along the slopes. As shown on Figure 1.1, the toe of the slopes, in plan, is generally straight for cut slopes located to the east of the proposed cask storage slabs but the footprint of the slope is both concave and convex for slopes that exist to the north.

Based upon our observations of slopes not covered with gunite, the present slope conditions appear to be slightly weathered with small erosional features such as rilling and minor channeling. These conditions are consistent with conditions noted by Woodward-McNeill (1974a) over twenty years ago and are not considered to increase the potential for slope failure to occur. No tension cracks were observed near the top of slopes.

The stratigraphy of the cut slopes generally consists of terrace deposits overlying dense sand of the San Mateo formation. Each of these materials has been studied in detail, including engineering properties related to slope stability (e.g., Woodward-McNeill 1973; Woodward-McNeill 1974b). The stratigraphy of the slopes to the east was carefully mapped and idealized material properties established in a previous study for the AWS Building (Woodward-McNeill 1974c). In this area, the terrace deposits consist of layers of clay, clayey sand, sandy clay, and gravel. The engineering properties for each of the idealized materials were assumed based upon previous investigations for Unit 1 and Units 2&3 facilities as summarized in Table 1. Groundwater was assumed to be at an elevation of about +2 feet based upon the site characterization effort for this study and the range of known groundwater levels at the site.

4.3 APPROACH

To evaluate the effects of ground shaking for DBE motions on the cut slopes, a simplified permanent displacement analysis procedure proposed by Makdisi and Seed (1977) was conducted. This procedure is based on a sliding block analogy. The procedure involves using earthquake magnitude-dependent charts that are a function of the ratio of yield acceleration to the maximum average acceleration which is unique for each sliding surface. To obtain the ratio of yield to maximum average acceleration, the following approach was used:

- 1) Select representative slope profiles;
- 2) Evaluate critical slip surfaces from static stability analyses;
- 3) Calculate the yield acceleration, k_y , for critical slip surfaces using applied horizontal forces and static slope stability analyses;
- 4) Estimate the maximum average acceleration within the material above the slip surface, k_{max} , and
- 5) Calculate the ratio of k_y / k_{max} .

Four profiles (A-A' through D-D'), considered to be the representatives of the most critical conditions with respect to the proposed cask storage slabs based on slope geometry and closeness to the cask storage slabs, were selected for static slope stability analysis, as located on Figure 1.1. For each profile, two critical slip surfaces were selected for analysis based upon the results of limit equilibrium analyses: (1) the slip surface that provided the minimum static factor of safety; and (2) a slip surface that provided a minimum static factor of safety and encompassed the entire slope height. A global search was made for the surface associated with a minimum static factor of safety. The toe of this surface for all four profiles was located at or slightly below the contact between Terrace Deposits and San Mateo Sand. A deeper slip surface was also selected for each profile by applying constraints to the automatic search routines in order to evaluate the potential displacements for a much larger area of the slope. Both circular and non-circular limit equilibrium analysis procedures were used. The yield acceleration was calculated for each slip surface by finding that seismic coefficient applied to the center of the sliding wedge above the slip surface that results in the static factor of safety is unity. The yield acceleration is equal to that coefficient in g's.

Three-dimensional effects of the slope configuration to the north may be an issue in the slope stability evaluation. However, a study by Duncan (1992) concluded that factors of safety from three-dimensional analyses are greater than factors of safety from two-dimensional analyses. Therefore, the results of two-dimensional slope stability analyses were considered to be reasonable and conservative for curved slopes in the vicinity of this project.

The maximum average acceleration, k_{max} , was estimated to equal the peak ground acceleration for the DBE (0.67g). This estimate of k_{max} is considered to be conservative for the shallow slip surfaces and even more conservative for the deep slip surfaces because the level of ground shaking is known to decrease with depth.

4.4 RESULTS

The locations of the slip surfaces on each of the four profiles A-A' through D-D' and corresponding factors of safety are shown on Figures 4.2 through 4.5, respectively. A summary of results is presented in Table 2. The minimum static factor of safety is 1.77 for a shallow surface at profile section C-C' as indicated from the results in Table 2 and shown on Figure 4.4. This factor of safety indicates that the slopes are considered to be stable under static loading conditions. The other factors of safety range up to 3.16. The yield accelerations range from 0.58 to 0.87g for shallow slip surfaces and from 0.54 to 0.74g for deeper slip surfaces as listed in Table 2 and shown on Figures 4.2 through 4.5. Also shown on Figures 4.2 through 4.5 are the ratios of k_y/k_{max} (using a k_{max} of 0.67g) used to estimate permanent slope displacements. These k_y/k_{max} ratios range from 0.81 to 1.3:

The estimates of permanent slope displacement using the Makdisi-Seed procedure are shown on Figure 4.6 considering the magnitude 7 earthquake that characterizes the DBE. As indicated on Figure 4.6, considering the range of k_y/k_{max} ratios, the computed displacements range from 0 (where $k_{yield}/k_{max} > 1$) to of the order of 1/2-inch using the lowest k_y/k_{max} ratio (0.81) and the top of the range of displacements for a magnitude 7 event.

4.5 CONCLUSIONS

The small displacements estimated using the Makdisi-Seed procedure as shown on Figure 6 suggest that only minor sloughing of the near slope surface materials is likely to occur during DBE ground shaking. Minor sloughing will not adversely affect the ISFSI which is located at distances greater than about 60 feet from the toe of the slopes. Therefore, slope stability will not be a concern for the ISFSI facility since the 60 feet offset provides a sufficient standoff distance.

Table 1
Material Properties

Index	Material	Classification	Unit Weight (pcf)	Friction Angle (degrees)	Cohesion (psf)
1	Terrace Deposits	CL-SC	130	17	2600
2	"	SC-GP	140	17	1400
3	"	CL-SC	130	17	1700
4	"	GP-SP	132	38	200
5	San Mateo Sand	SW to SW-SM	125	41	800

Table 2
Summary of Static Slope Stability Results

Section	Shallow Slip Surface		Deeper Slip Surface	
	Factor of Safety	Yield Acceleration	Factor of Safety	Yield Acceleration
A-A'	1.91	0.78g	1.94	0.61g
B-B'	1.93	0.83g	2.05	0.54g
C-C'	1.77	0.58g	2.26	0.55g
D-D'	2.13	0.87g	3.16	0.74g

5.0 LIQUEFACTION

5.1 INTRODUCTION

This section summarizes a study to evaluate liquefaction potential for native foundation materials that could result from ground shaking associated with the Design Basis Earthquake (DBE). The evaluation included using the results of the site characterization study, a review of SONGS Unit 1 project files related to idealized subsurface conditions, and a performance-based simplified method of analyzing the potential for liquefaction. All previous evaluations at SONGS have concluded that San Mateo sand is so dense that it is not subject to liquefaction.

APPENDIX A

5.2 SITE CONDITIONS

The proposed slab configuration for the proposed ISFSI is shown on Figure 1.1. The project is located east of the Unit 1 Diesel Generating Building and north of the AWS Building as shown on Figure 1.1. The surface topography in the plant area is generally flat, at Elev. +20 feet, and sloped slightly toward the Pacific Ocean to the west. However, the project is located near the base of existing cut slopes made during construction of Unit 1 with the closest distance between the base of cut slopes and the edge of the proposed cask storage foundation slab to be about 60 feet.

Sands of the San Mateo formation are known to be the foundation material for the ISFSI as determined through a project-specific field and laboratory investigation documented site characterization section. San Mateo sand is the foundation material for all main plant facilities with footing elevations below about 50 to 60 feet. The San Mateo formation likely extends down to an estimated elevation of -850 feet.

The San Mateo formation is of Pleistocene age and the upper units generally consist of a very dense, well-graded sand with apparent cohesion due to efficiency in grain-packing. The physical and engineering properties of this material have been studied extensively (e.g., Woodward-McNeill 1973; Woodward-McNeill 1974b; and Woodward-McNeill 1974c). The range of gradations and moisture contents measured in the site characterization section are consistent with the ranges measured previously in the Unit 1 and Units 2&3 plant area.

5.3 APPROACH

The performance-based method by Seed, et al. (1985) was used to evaluate the potential for liquefaction of the San Mateo sand at the ISFSI. This empirical method was derived by using SPT blowcounts measured at sites that have experienced liquefaction and at sites that have not experienced liquefaction during earthquakes as shown on Figure 5.1. Relationships defining combinations of modified standard penetration blowcounts (a measure of the denseness of the soil) and cyclic stress ratio (a measure of the severity of ground shaking) were interpreted by Seed, et al. (1985) from this data base. These relationships are dependent on the magnitude of the earthquake shaking and the percentage of fines (material passing the #200 sieve). The relationship for a Richter Magnitude 7.5 earthquake and clean sand is shown on Figure 5.1.

Measured SPT blowcounts must be modified to make an accurate evaluation of liquefaction potential. Blowcounts are normalized to an equivalent effective overburden stress of 1 ton per square foot and adjusted, if necessary, for hammer efficiency and other factors that affect blowcount. Total unit weights of 125.5 and 136.5 pcf (nominally this corresponds to a uniform dry unit weight of about 123 pcf) were used for San Mateo sand above and below the water table, respectively, with the water table assumed to be at +2 feet mllw (based on the recent drilling). A hammer energy efficiency ratio of 60 percent was assumed for the drill rig and hammer system used (Seed, 1986). Relationships for a magnitude 7 earthquake and clean sands (5 percent fines or less) were used for comparisons. The use of the clean sand relationship to

represent the San Mateo sand at this project site with fines contents ranging from 7 to 11 percent is conservative for a liquefaction evaluation.

5.4 RESULTS

The modified blowcounts from borings at the ISFSI are generally in the same range and higher than blowcounts previously recorded at SONGS. A histogram and a polynomial fit representing the cumulative distribution of recent data are shown on Figure 5.2. Also shown for comparison on Figure 5.2 is the previous cumulative distribution of modified blowcounts in San Mateo sand at SONGS. These data confirm that the San Mateo sand at this project location is as dense as the San Mateo sand previously tested.

The range of modified blowcounts from borings at the ISFSI were compared with the liquefaction curve shown on Figure 5.2 for Magnitude 7.5 events as shown on Figure 5.3. All recent data are shown although some of the blowcounts are in unsaturated sand and, therefore, could not liquefy. As shown on Figure 5.3, the horizontal scale for blowcounts was modified to allow the full range of modified blowcounts to be represented. Also shown on Figure 5.3 is the equivalent liquefaction curve for a Magnitude 7 event which is representative for the DBE used for the SONGS site and the previous range of modified blowcounts.

5.5 CONCLUSIONS

The comparisons shown on Figures 5.2 and 5.3 indicate that the San Mateo sand (i.e., the pad foundation material) will not experience liquefaction during the DBE-level shaking, because the modified blowcounts, as shown by the range on Figure 5.3, are much larger than the blowcount threshold curve. This conclusion is consistent with all other previous findings for San Mateo sand.

6.0 REFERENCES

6.1 SITE CHARACTERIZATION

Hawkins, H. Gene, 1995, Verbal communication.

Woodward-Clyde Consultants, 1975. "Report of Phase I of Geotechnical Investigation for the Proposed Diesel Generating Facilities, Songs Unit 1, San Onofre, California," Consultants report to Southern California Edison, March 25.

Woodward-McNeill and Associates, 1973. "Stability of Proposed Slopes for Proposed Units 2 and 3, San Onofre Generating Station, San Onofre, California," Consultants report to Southern California Edison, March 27.

Woodward-McNeill and Associates, 1974a. "Dynamic Switchyard Slope Stability Confirmation, San Onofre Generating Station, San Onofre, California," Consultants report to Southern California Edison, October 18.

Woodward-McNeill and Associates, 1974b. "Bearing Capacity and Settlement Evaluation for the Proposed AWS Building, San Onofre, California," Consultants report to Southern California Edison, November 7.

6.2 TSUNAMI

Marine Advisers, 1965, *Examination of Tsunami Potential at the San Onofre Nuclear Generating Station*, prepared for Southern California Edison Co, September 1965.

Marine Advisers, 1966, *A comparison of Offshore Portugal and Southern California*, Report A-203A, prepared for Southern California Edison Company.

Southern California Edison Co., *San Onofre Nuclear Generating Station, Final Safety Analysis Report*, 1975.

Southern California Edison Co., Design Calculations DC-1650, SONGS 1 *Seawall Seismic Study*.

Wilson, B.W., 1972 *Estimate of Tsunami Effect at San Onofre Nuclear Generating Station, Units 2 and 3, California*, prepared for Southern California Edison Co.

Woodward Clyde Consultants, 1986, *Wave Forces, Slings & Seawall*, prepared for Southern California Edison Co.

6.3 DBE RESPONSE SPECTRA

Abrahamson, N.A., 1995 Personal communication.

Newmark, N. M. and Hall, W. J., 1969. "Seismic Design Criteria for Nuclear Reactors," Proc. 4th World Conf. on Earthquake Engineering, Vol. II, Santiago, Chile.

Risk Engineering, Inc., 1994. *Seismic Hazard at San Onofre Nuclear Generating Station*, Prepared for Southern California Edison Company, Report dated December 20, 1994.

US Nuclear Regulatory Commission, 1980, *Safety Evaluation Report (Geology and Seismology) related to the operation of San Onofre Nuclear Generating Station, Units 2 and 3*.

6.4 SLOPE STABILITY

Duncan, J. M., 1992. "State-of-the-Art: Static Stability and Deformation Analysis," Proc., Stability and Performance of Slopes and Embankments-II, Vol I, ASCE.

Makdisi, F. and Seed, H. B. 1977. "Simplified Procedure for Estimating Dams and Embankment Earthquake-Induced Deformations," Journal, Soil Mechanics and Foundation Engineering, ASCE, 104(GT7), pp. 849-867.

Woodward-McNeill and Associates, 1973. "Stability of Proposed Slopes for Proposed Units 2 and 3, San Onofre Generating Station, San Onofre, California," Consultants report to Southern California Edison, March 27.

Woodward-McNeill and Associates, 1974a. "Erosion Study for the Proposed Units 2 and 3, San Onofre Generating Station, San Onofre, California," Consultants report to Southern California Edison, January 30.

Woodward-McNeill and Associates, 1974b. "Dynamic Switchyard Slope Stability Confirmation, San Onofre Generating Station, San Onofre, California," Consultants report to Southern California Edison, October 18.

Woodward-McNeill and Associates, 1974c. "Bearing Capacity and Settlement Evaluation for the Proposed AWS Building, San Onofre, California," Consultants report to Southern California Edison, November 7.

6.5 LIQUEFACTION

Seed, H. B., Tokimatsu, K., Harder, L. F., Jr., Chung, R. M., 1985. "Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations," Journal, Geotechnical Engineer, ASCE, 112(11), pp 1016-1032.

Woodward-McNeill and Associates, 1973. "Stability of Proposed Slopes for Proposed Units 2 and 3, San Onofre Generating Station, San Onofre, California," Consultants report to Southern California Edison, March 27.

Woodward-McNeill and Associates, 1974b. "Dynamic Switchyard Slope Stability Confirmation, San Onofre Generating Station, San Onofre, California," Consultants report to Southern California Edison, October 18.

Woodward-McNeill and Associates, 1974c. "Bearing Capacity and Settlement Evaluation for the Proposed AWS Building, San Onofre, California," Consultants report to Southern California Edison, November 7.

CALC C-296-01-03 SHT A-22
APPENDIX A

APPENDIX A

BORING LOGS - FIELD INVESTIGATION

APPENDIX A: Boring Logs - Field Investigation

A field investigation was performed on September 23, 1995, and consisted of two rotary-wash borings, B-1 and B-2, with soil sampling advanced to depths of about 57.5 and 17.3 feet below existing ground surface, respectively. Figure 1 shows the approximate locations of the borings.

The borings were advanced using a Mobil B-1 drill rig with a 3-1/4-inch tricone drill bit. Bentonite drilling fluid was circulated through each borehole during drilling to remove soil cuttings. The boring logs were initiated in the field by a Woodward-Clyde engineer who recorded soil characteristics, observations, sample locations, and other drilling information. Further details regarding the drilling, sampling, and logging procedures are presented on Figure A-1, the Key to Log of Boring. The final boring logs were prepared based on visual observations of the samples and the results of the laboratory tests. The logs for Borings B-1 and B-2 are presented on Figures A-2 and A-3, respectively.

Groundwater was measured in Boring B-1 at a depth of 18 feet below ground surface approximately 40 minutes after drill completion. The borings were backfilled with a bentonite slurry to the top of the borehole which was at 6 feet below ground surface. The remaining portion of the trench (from 0 to 6 feet) was covered with a steel plate waiting to be slurried by SCE personnel.

Soil Sampling

Samples of soil were obtained using a Standard Penetration Test (SPT) sampler. SPT samples were taken by attempting to drive the sampler approximately 18 inches into the soil at the bottom of the borehole using a 140-pound out-of-hole safety hammer falling approximately 30 inches. The hammer was hoisted by a hydraulic trip-hammer mechanism. The SPT sampler's cutting shoe and barrel have nominal inside diameters of 1-3/8 and 1-1/2 inches, respectively; the barrel's nominal outside diameter is 2 inches; liners were not used.

The SPT samples were placed in plastic bags and the bags secured to help preserve the natural moisture content. Sample identification was written on a label and attached to each bag.

Penetration resistance is reflected by the number of blows required to drive the sampler into the soil. However, for this project, the greatest penetration was only 6 inches. Therefore, the recorded "blowcounts" reflect the number of blows in the seating interval to drive the sampler to specified distances.

The degree of relative density of granular soils is generally described on the boring logs according to the following conventional correlation for sands:

<u>Blow Count/ft</u>	<u>Description</u>
0-4	Very loose
4-10	Loose
10-30	Medium dense
30-50	Dense
>50	Very dense

The relative density and consistency descriptions may deviate from the correlations for a number of reasons, including reliance on other test results or the engineer's judgment based on manual manipulation of the sample, and simplification of the logs to increase readability.

It is widely accepted that the above SPT blow count correlations are overly simplistic and the blow counts should be adjusted for other factors, including effective vertical pressure at the sample depth and the details of the sampling system (such as hammer, rods, sampler, and technique). As mentioned above, the relative density and consistency descriptions are based on the unadjusted SPT blow count recorded in the field and reported on the logs.

Project: SONGS - Cask Storage
 Project Location: San Onofre, California
 Project Number: 954E181

APPENDIX A

Key to Log of Boring

Sheet 1 of 1

Depth, feet	Approximate Elevation, feet	SAMPLES			MATERIAL DESCRIPTION and other remarks	Water Content, %	Dry Unit Weight, pcf	Other Tests
		Type	Number	Blows/foot Graphic Log				
0					Very dense, moist, brown, well-graded SAND (SW).			
		1	55		Standard Penetration Test (SPT) sample	23	117	SA(3)
		2	16 NR		Number of blows required to advance sampler 1 foot (NR indicates no sample was recovered)			
5					Modified California sample			
		3	50/4**		* indicates blow count for seating interval of sampler, refusal after 4 inches			
10								

ABBREVIATIONS FOR "OTHER TESTS" COLUMN

SA(3) - sieve analysis (percent passing No. 200 sieve)

GENERAL NOTES

1. Classifications are based on ASTM D2487-92 and include relative density/consistency, moisture, and color. Field descriptions have been modified to reflect results of laboratory tests where deemed appropriate.
2. Surface elevations are based on topographic maps and estimated locations.
3. Descriptions on these boring logs apply only at the specific boring locations and at the time the borings were made. They are not warranted to be representative of subsurface conditions at other locations or times.



Project: SONGS - Cask Storage
 Project Location: San Onofre, California
 Project Number: 954E181

APPENDIX A

Log of Boring B-1

Sheet 1 of 2

Date(s) Drilled	9/23/95	Logged By	M. Tufenkjian	Checked By	D. Sykora
Drilling Method	Rotary Wash	Drill Bit Size/Type	3-1/4-inch tricone	Total Depth Drilled (feet)	57.5
Drill Rig Type	Mobile B-61	Drilled By	F & C Drilling Inc.	Hammer Weight/Drop (lbs/in.)	140 / 30
Groundwater Level	18 feet below ground surface	Date Measured	9/23/95	Approx. Surface Elevation (feet)	20
Comments	Borehole potholed to 6 ft below ground surface.			Borehole Backfill	Bentonite slurry

Depth, feet	Approximate Elevation, feet	SAMPLES		Graphic Log	MATERIAL DESCRIPTION and other remarks	Water Content, %	Dry Unit Weight, pcf	Other Tests
		Type	Number	Blows/foot				
0	20				2 inches asphaltic concrete.			
					Moist, light yellowish brown, well-graded SAND with SILT (SW-SM), with pieces of asphaltic concrete (Fill).			
					Very dense, moist, light yellowish brown, well-graded SAND with SILT (SW-SM) (San Mateo Formation).			
5	15							
			1	71/5"	↓ Becomes wet.	13		SA(8)
10	10							
			2	75/4"				
15	5							
			3	70/4"		21		SA(10)
20	0							
			4	70/3.5"				
25	-5							
			5	70/2.5"		18		SA(11)
30	-10							

Project: SONGS - Cask Storage <i>APPENDIX A</i>	Log of Boring B-1
Project Location: San Onofre, California	Sheet 2 of 2
Project Number: 954E181	

Depth, feet	Approximate Elevation, feet	SAMPLES				MATERIAL DESCRIPTION and other remarks	Water Content, %	Dry Unit Weight, pcf	Other Tests
		Type	Number	Blows/foot	Graphic Log				
30	-10					Very dense, wet, light yellowish brown, well-graded SAND with SILT (SW-SM) [San Mateo Formation] (continued).			
			6	70/3.5"		↓ Becomes coarser-grained.			
35	-15		7	70/4"			15		SA(9)
40	-20		8	70/4"			16		SA(7)
45	-25		9	70/4.5"					
50	-30		10	70/4.5"					
55	-35		11	103/6"			18		SA(10)
60	-40					Bottom of boring at 57.5 feet. Groundwater measured at 18 feet below ground surface 40 minutes after completion of drilling. Hole grouted up to depth of 8 feet.			
65	-45								



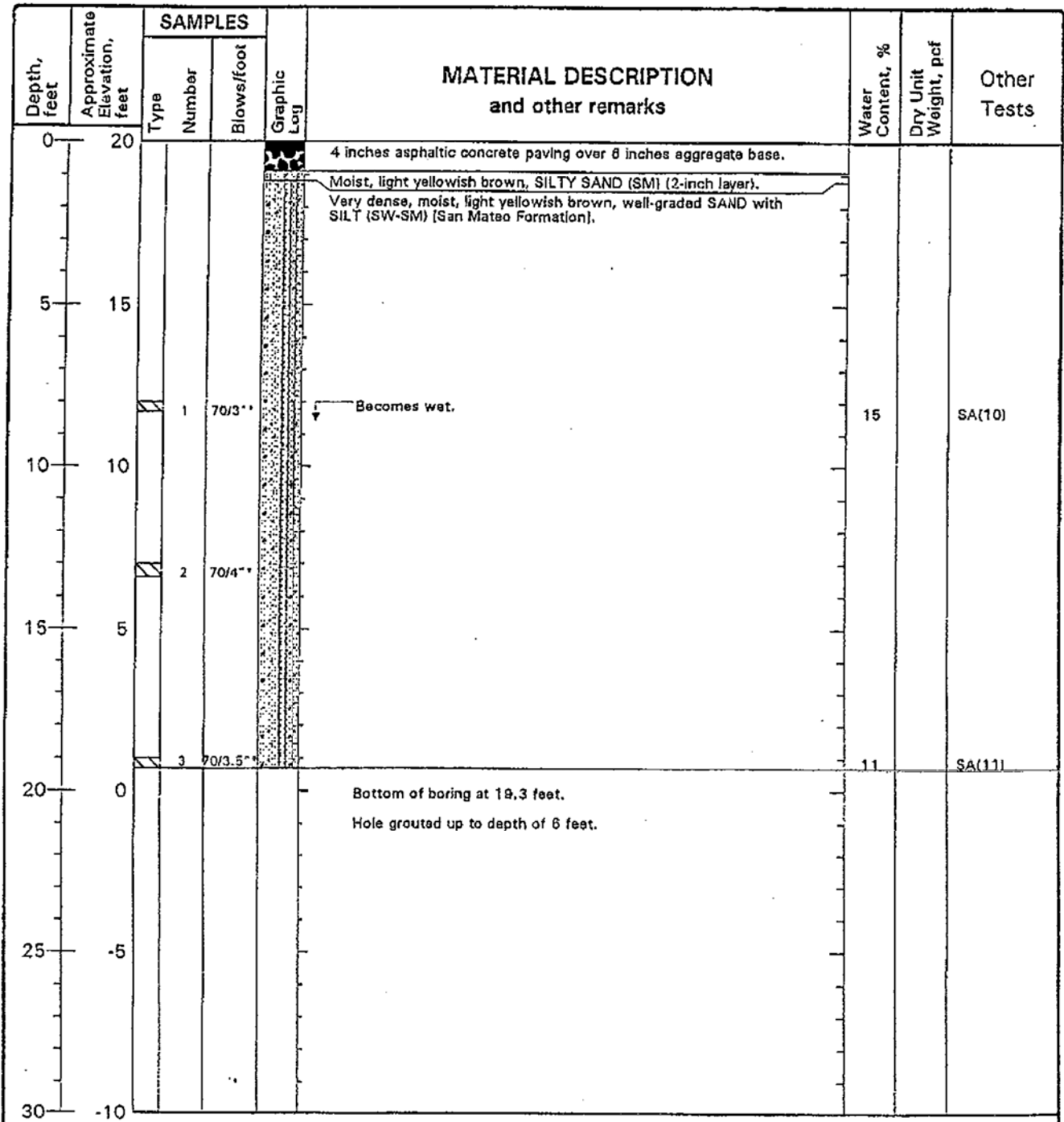
Project: SONGS - Cask Storage
 Project Location: San Onofre, California
 Project Number: 954E181

APPENDIX A

Log of Boring B-2

Sheet 1 of 1

Date(s) Drilled	9/23/95	Logged By	M. Tufankjian	Checked By	D. Sykora
Drilling Method	Rotary Wash	Drill Bit Size/Type	3-1/4-inch tricone	Total Depth Drilled (feet)	19.3
Drill Rig Type	Mobile B-61	Drilled By	F & C Drilling Inc.	Hammer Weight/Drop (lbs/in.)	140 / 30
Groundwater Level	Not established	Date Measured	Not applicable	Approx. Surface Elevation (feet)	20
Comments	Borehole potholed to 6 ft below ground surface.			Borehole Backfill	Bentonite slurry



APPENDIX B

GRADATION CURVES (LABORATORY TESTING)

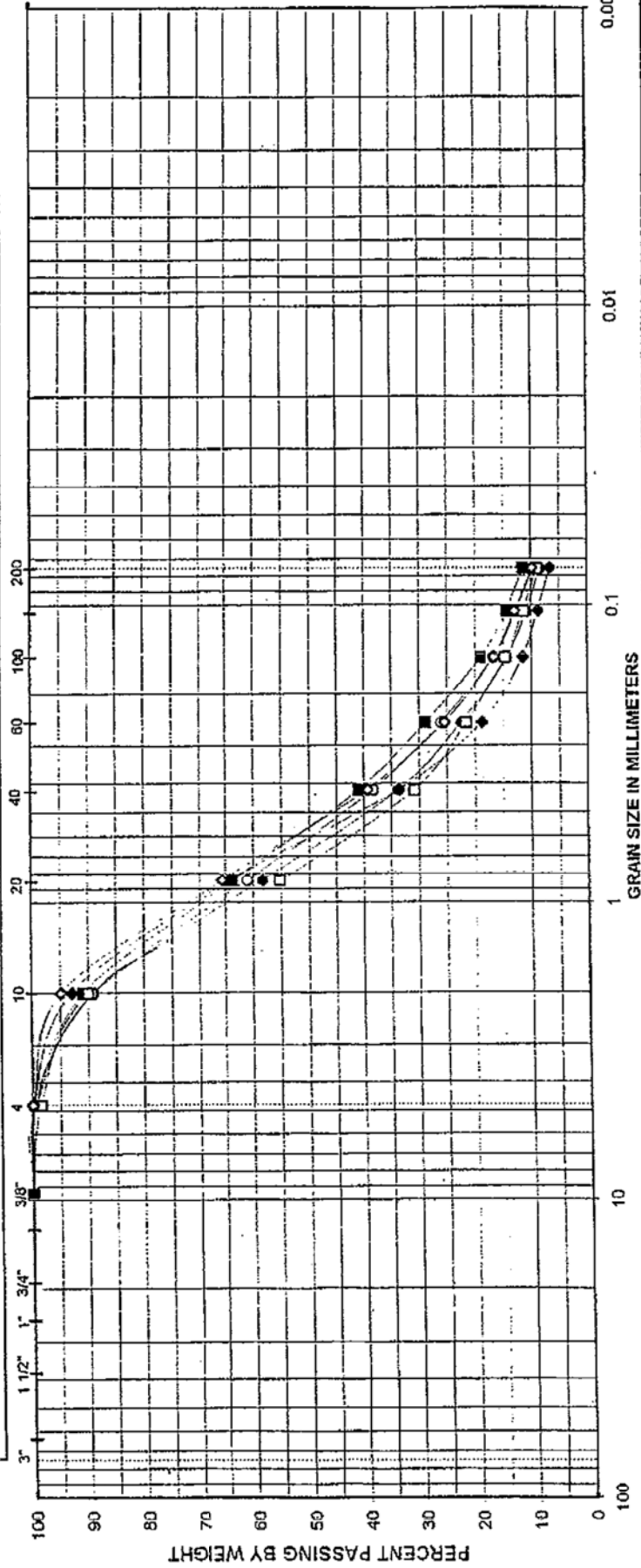
APPENDIX B: Gradation Curves (Laboratory Testing)

The soil samples obtained from borings were visually classified and selected soil samples were tested in our laboratory to determine their classification indices by means of sieve analyses and determining water contents.

The results of the moisture content and sieve analyses (through the No. 200 sieve) are presented on the boring logs (Appendix A) at the depth corresponding to the sample for convenient correlation with the soil profile. The results of the sieve analyses in terms of the grain-size distribution curves are presented on Figures B-2 and B-3.

UNIFIED SOIL CLASSIFICATION

GRAVEL		SAND		SILT OR CLAY
COARSE	FINE	COARSE	FINE	
U. S. STANDARD SIEVE SIZES				
				HYDROMETER



Exploration No.	Sample No.	Depth (ft)	SYMBOL	Wn	LL	PI	Description and Classification	D ₆₀	D ₃₀	D ₁₀	Cu	Cc	CF
B-2b	1	7.0	●	12.7	NT	NT	Brownish yellow well-graded SAND with silt (SW-SM)	0.88	0.38	0.090	9.8	1.8	
B-2b	3	17.0	○	20.8	NT	NT	Brownish yellow well-graded SAND with silt (SW-SM)	0.82	0.30	0.075	10.9	1.5	
B-2b	5	27.0	■	16.4	NT	NT	Brownish yellow well-graded SAND with silt (SW-SM)	0.75	0.27	0.065	11.5	1.5	
B-2b	7	37.0	□	15.3	NT	NT	Brownish yellow well-graded SAND with silt (SW-SM)	0.81	0.30	0.075	10.8	1.5	
B-2b	8	42.0	◆	16.3	NT	NT	Pale yellow well-graded SAND with silt (SW-SM)	0.95	0.40	0.090	10.6	1.9	
B-2b	11	57.0	◇	17.5	NT	NT	Brownish yellow well-graded SAND with silt (SW-SM)	0.70	0.30	0.075	9.3	1.7	

PARTICLE-SIZE DISTRIBUTION CURVES

Figure B-1

PROJECT NAME: SONGS - CASK STORAGE

PROJECT NUMBER: 954E181A

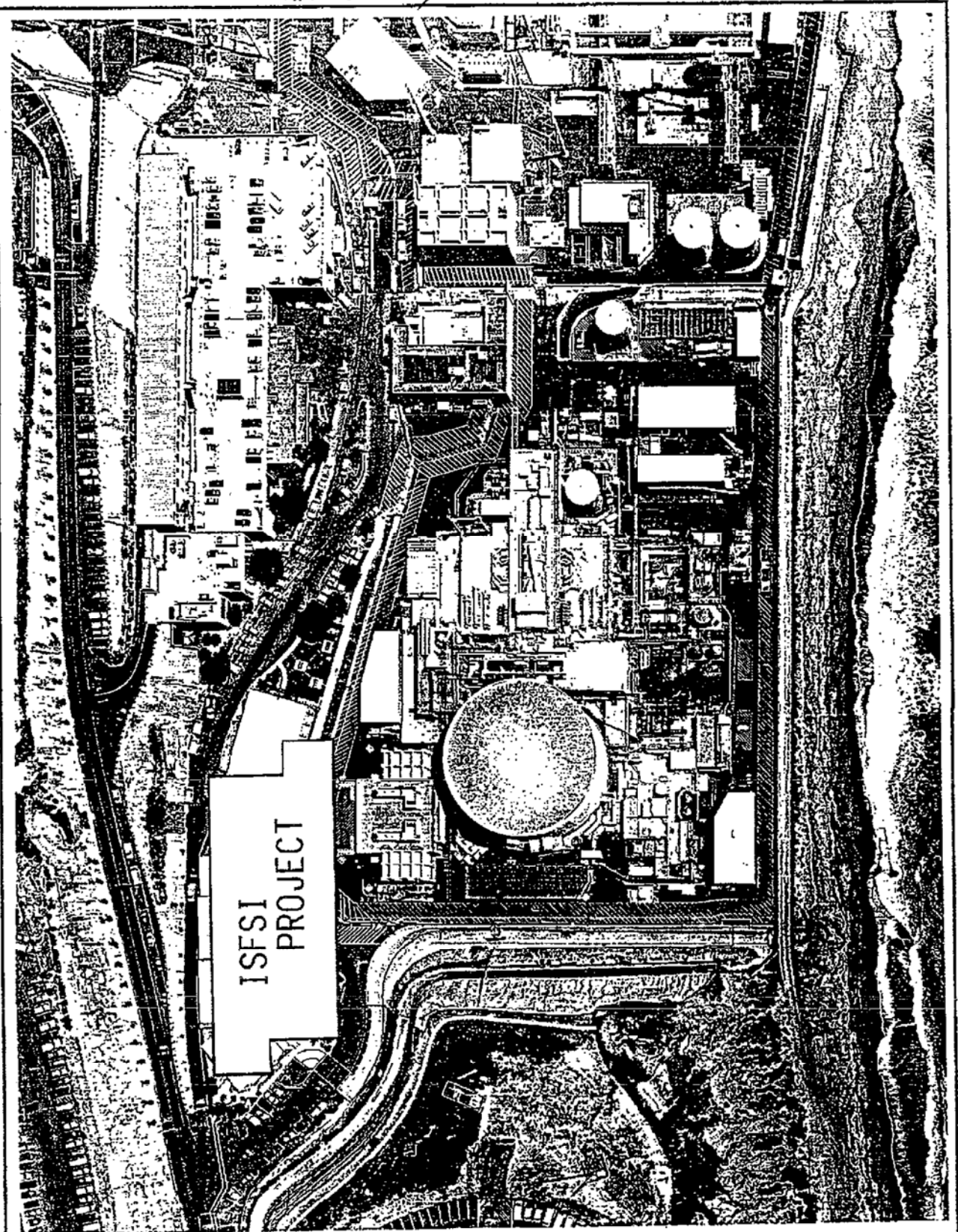
WOODWARD-CLYDE CONSULTANTS

SASONGSA.XLS

(SNA) particle-size (6/95)

CALC C-296-01.03 SAT A-33
APPENDIX A

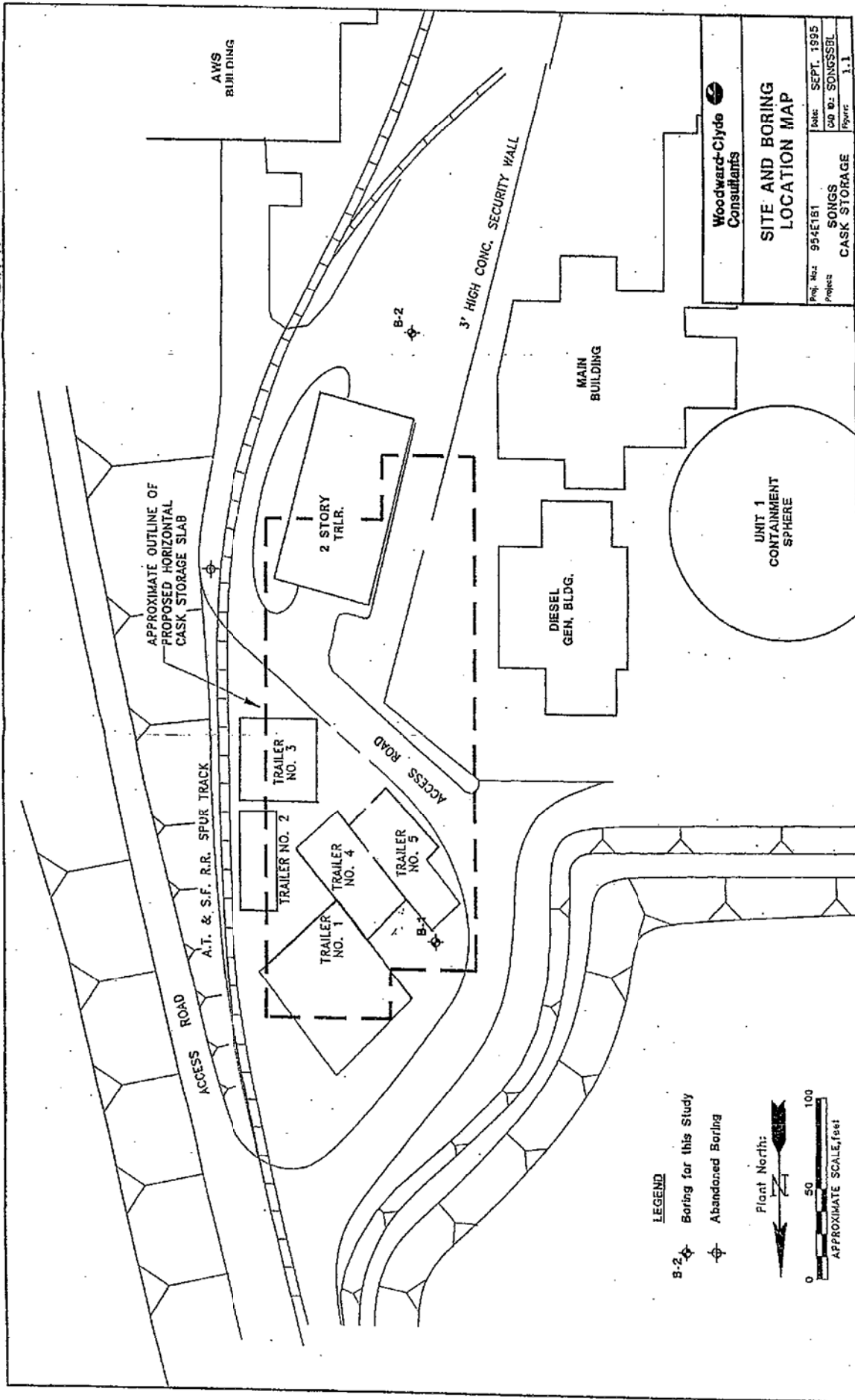
FIGURES



SITE AERIAL PHOTO

FIG. 1.0

CHC C-296-0103 SHT A-35
APPENDIX A



LEGEND

B-2 Boring for this Study

B-1 Abandoned Boring

Plant North:

0 50 100

APPROXIMATE SCALE, feet

Woodward-Clyde
Consultants

**SITE AND BORING
LOCATION MAP**

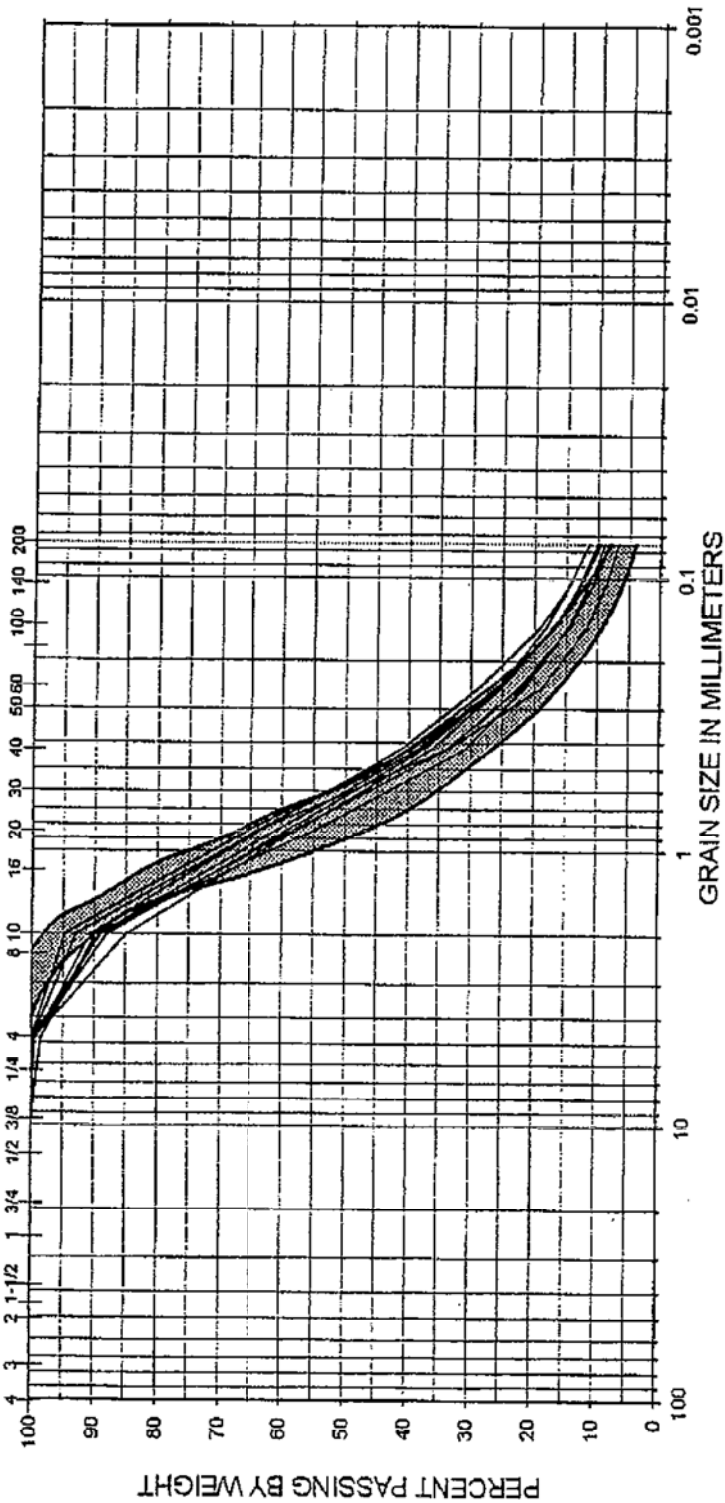
Proj. No. 954E181	Date: SEPT. 1995
Project: SONGS	Drawn by: SONGSBL
CASK STORAGE	Figure: 1.1

APPENDIX A

PARTICLE-SIZE DISTRIBUTION					
COBBLES		GRAVEL		SAND	
COARSE	FINE	COARSE	FINE	COARSE	FINE

SILT OR CLAY

U.S. Standard Sieve Size



SYMBOL DESCRIPTION

Typical Range for San Mateo Sand

Gradation for Task 100 study

sa:jlis
jms:del
gradinc.dwg

d:\songst\stave\grad1.gri

Project No.
954E181

Songs Cask Storage

Woodward-Clyde Consultants



COMPARISON OF MEASURED
PARTICLE SIZE DISTRIBUTION
WITH TYPICAL RANGE FOR
SAN MATEO SAND

Figure
1.2

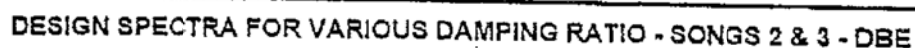
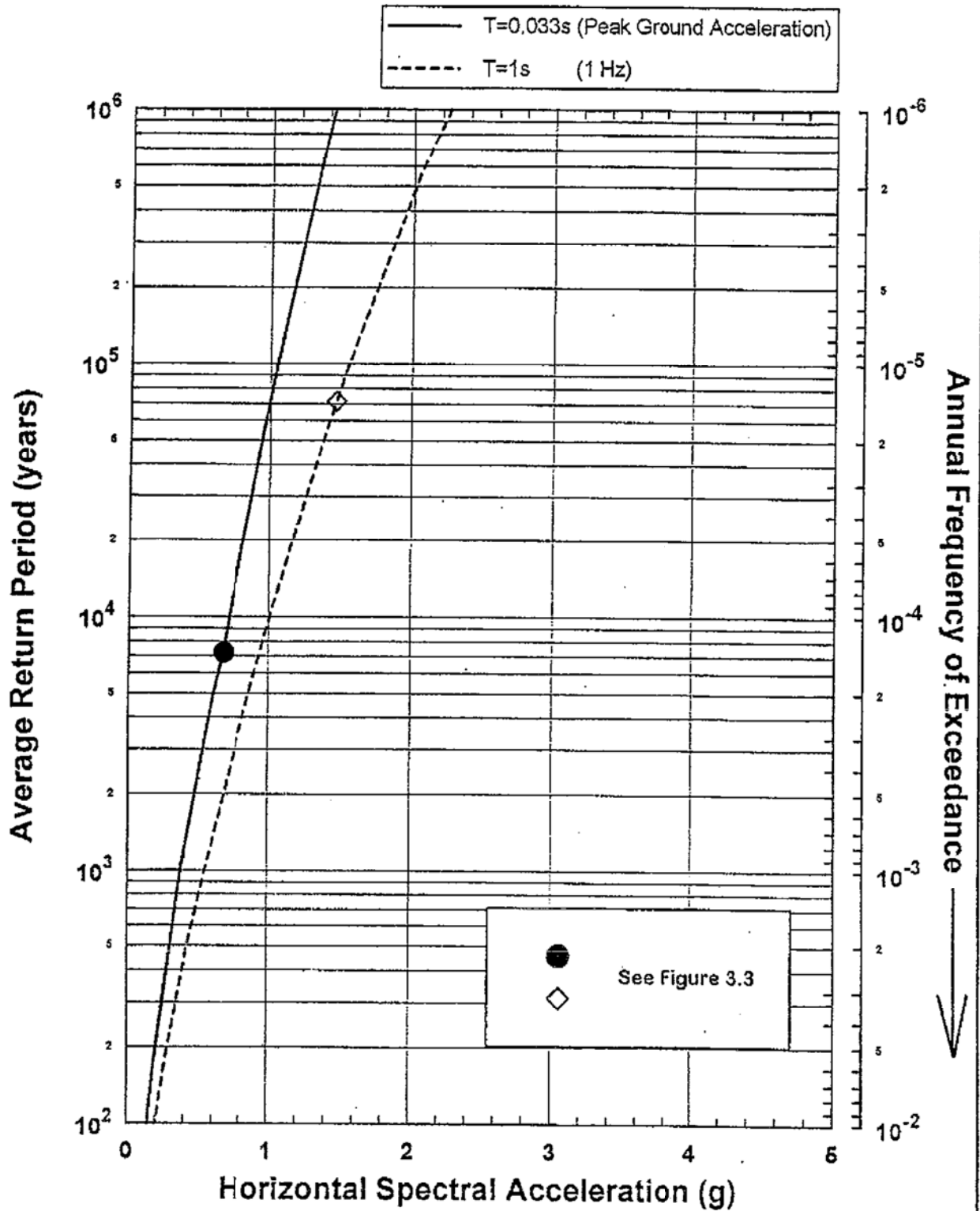


Fig. 3.1

CALC C-296-01.03 SHT A-38
APPENDIX A



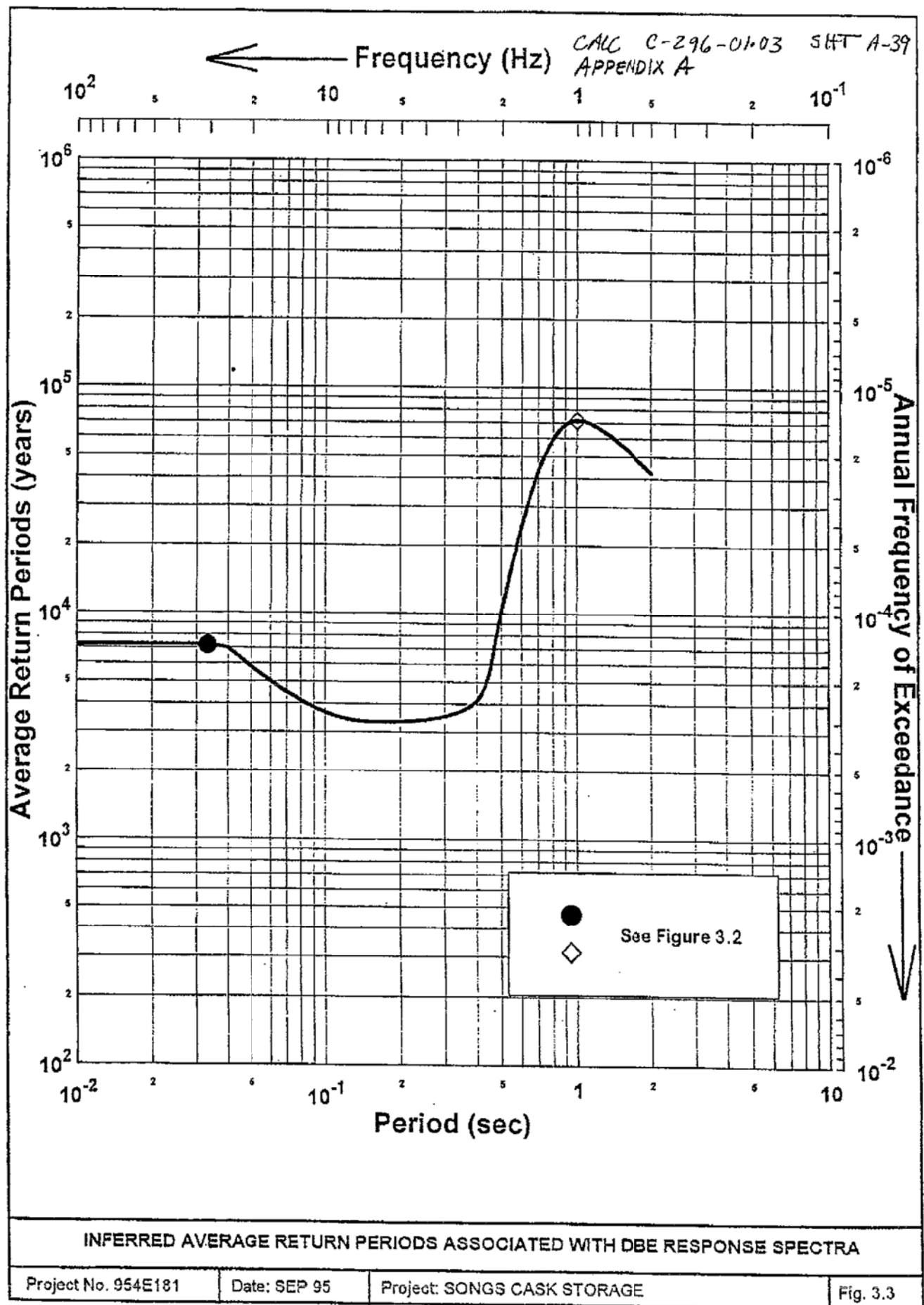
IPEEE HORIZONTAL SPECTAL HAZARD CURVES
 $T=0.033s$ (PEAK GROUND ACCELERATION) AND $T=1s$ (1 Hz)

Project No. 954E181

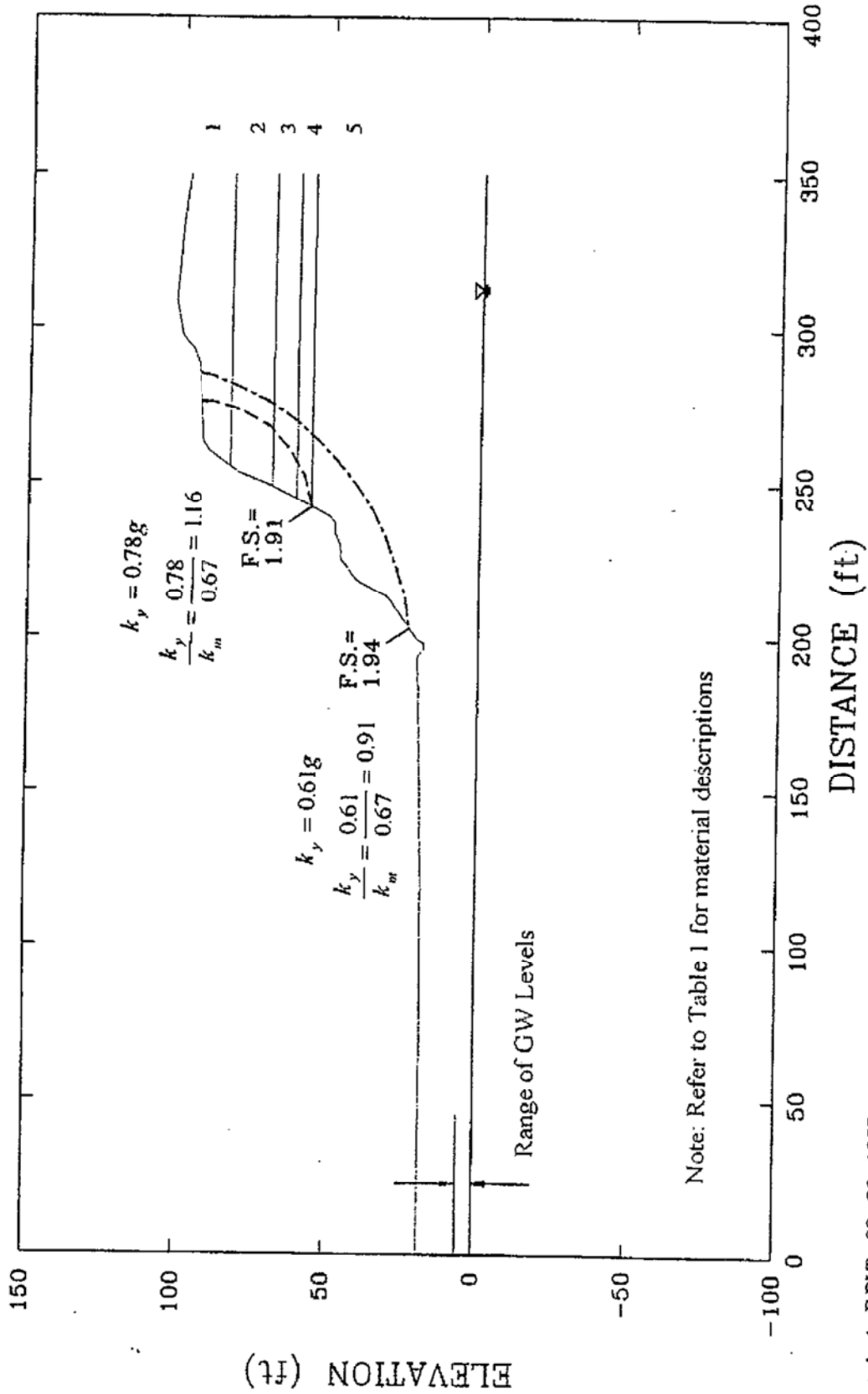
Date: SEP 95

Project: SONGS CASK STORAGE

Fig. 3.2



CALC C-296-01-03 SHT A-40
APPENDIX A



File : A-A-F.PLT, 09-26-1995

Run By: P.TAN

SECTION A-A' - SLOPE STABILITY FACTOR OF SAFETY

Project No.: 954E181

Date: SEP 95

Project:

SONGS CASK STORAGE

Fig. 4.1

Diagram illustrating the cross-section of a bridge deck, showing ELEVATION (ft) on the vertical axis (ranging from -100 to 150) and DISTANCE (ft) on the horizontal axis (ranging from 0 to 400).

The diagram shows five numbered layers (1 to 5) representing different materials and their properties:

- Layer 1: $k_y = 0.83g$, $\frac{k_y}{k_m} = \frac{0.83}{0.67} = 1.24$
- Layer 2: F.S. = 1.93
- Layer 3: F.S. = 2.05
- Layer 4: $k_y = 0.54g$, $\frac{k_y}{k_m} = \frac{0.54}{0.67} = 0.81$
- Layer 5: (Bottom concrete layer)

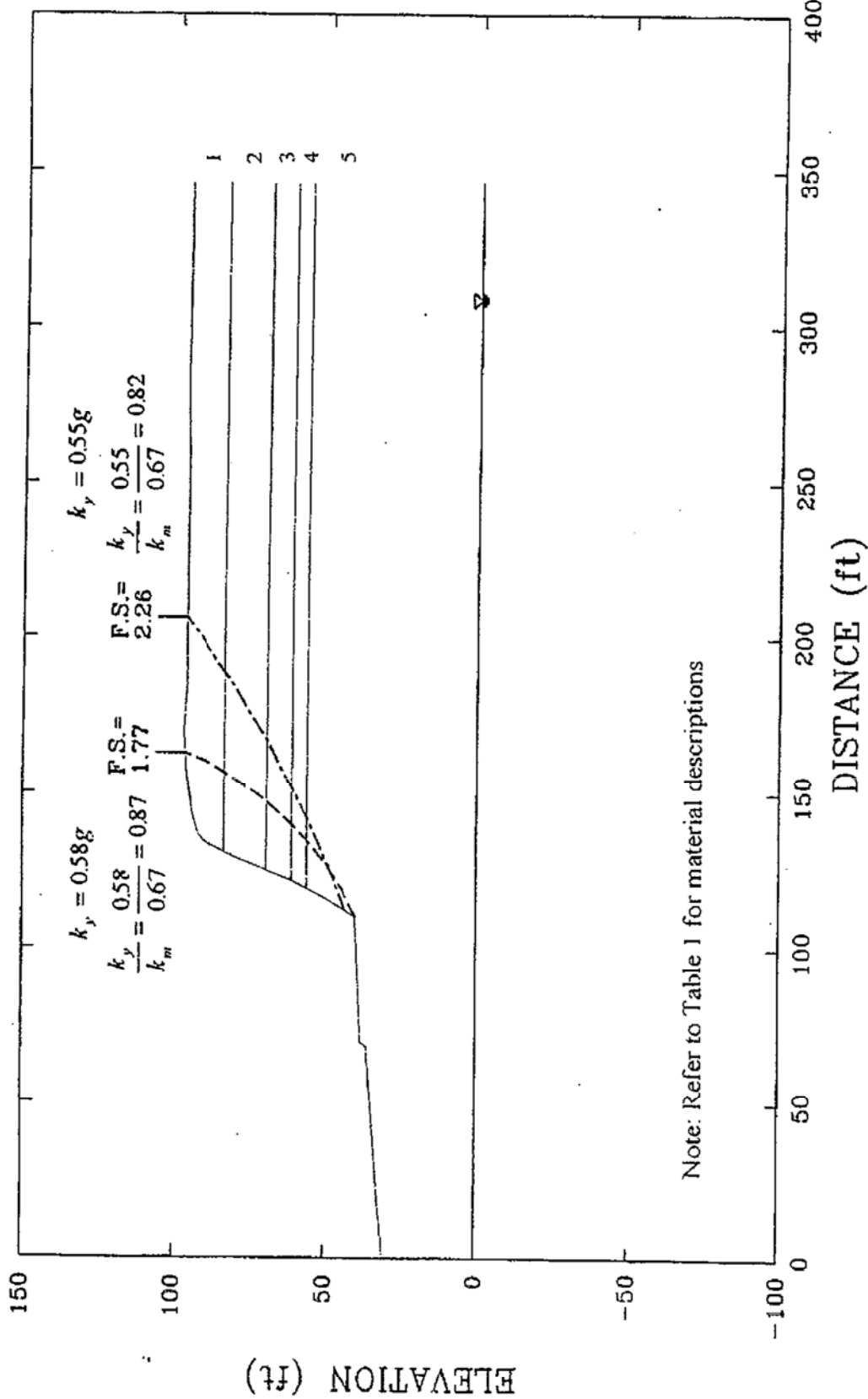
Note: Refer to Table 1 for material descriptions

Run By: P.TAN

SECTION B-B' - SLOPE STABILITY FACTOR OF SAFETY

Fig. 4.2

CALC C-296-01.03 SHT A-42
APPENDIX A



File : C-C1.PLT, 09-28-1995

Run By: P.TAN

SECTION C-C' - SLOPE STABILITY FACTOR OF SAFETY

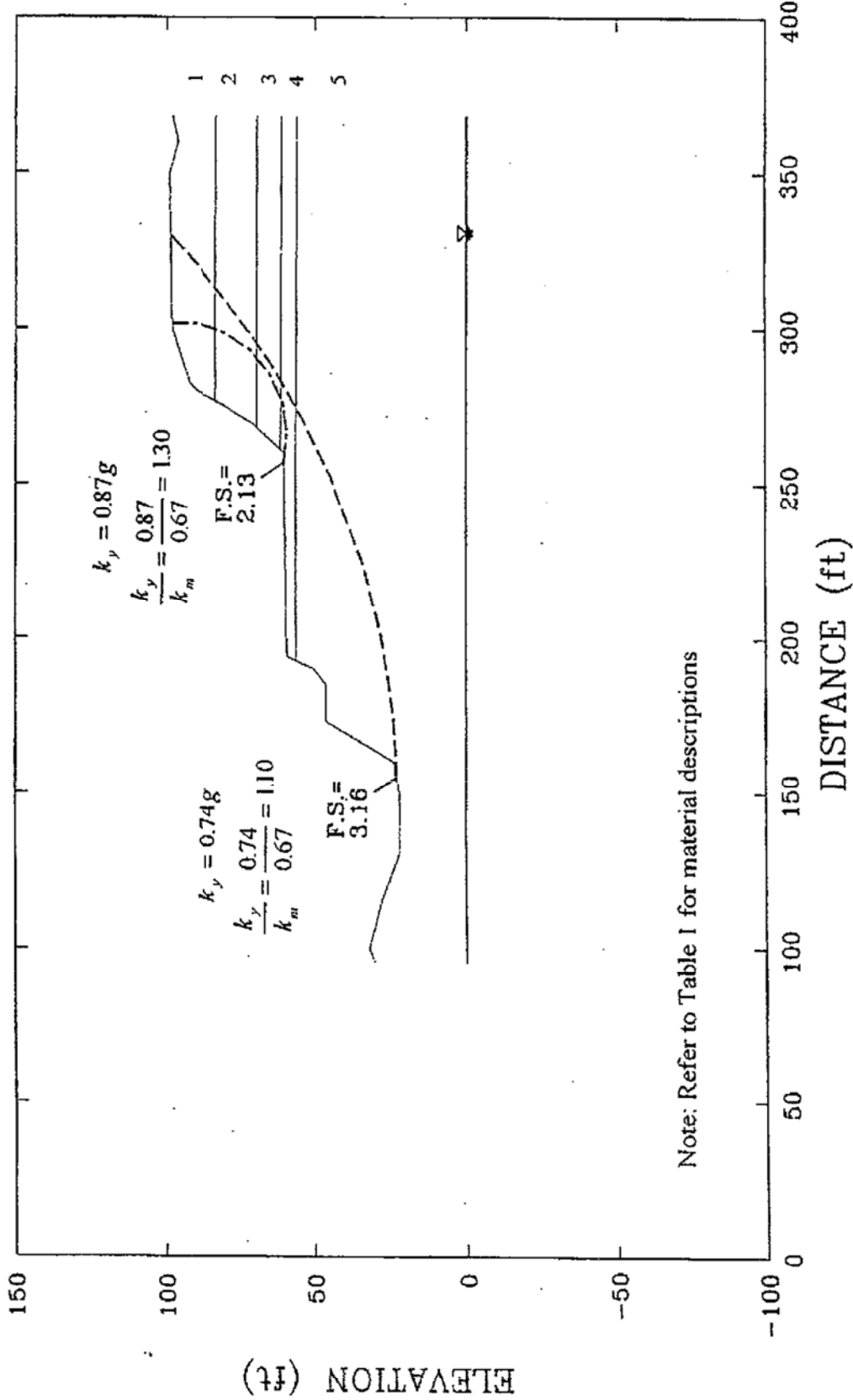
Project No.: 954E181

Date: SEP 95

Project:

SONGS CASK STORAGE

Fig. 4.3



File : D-D4.PLT. 09-26-1995

Run By: P.TAN

SECTION D-D' - SLOPE STABILITY FACTOR OF SAFETY

Project No.: 954E181

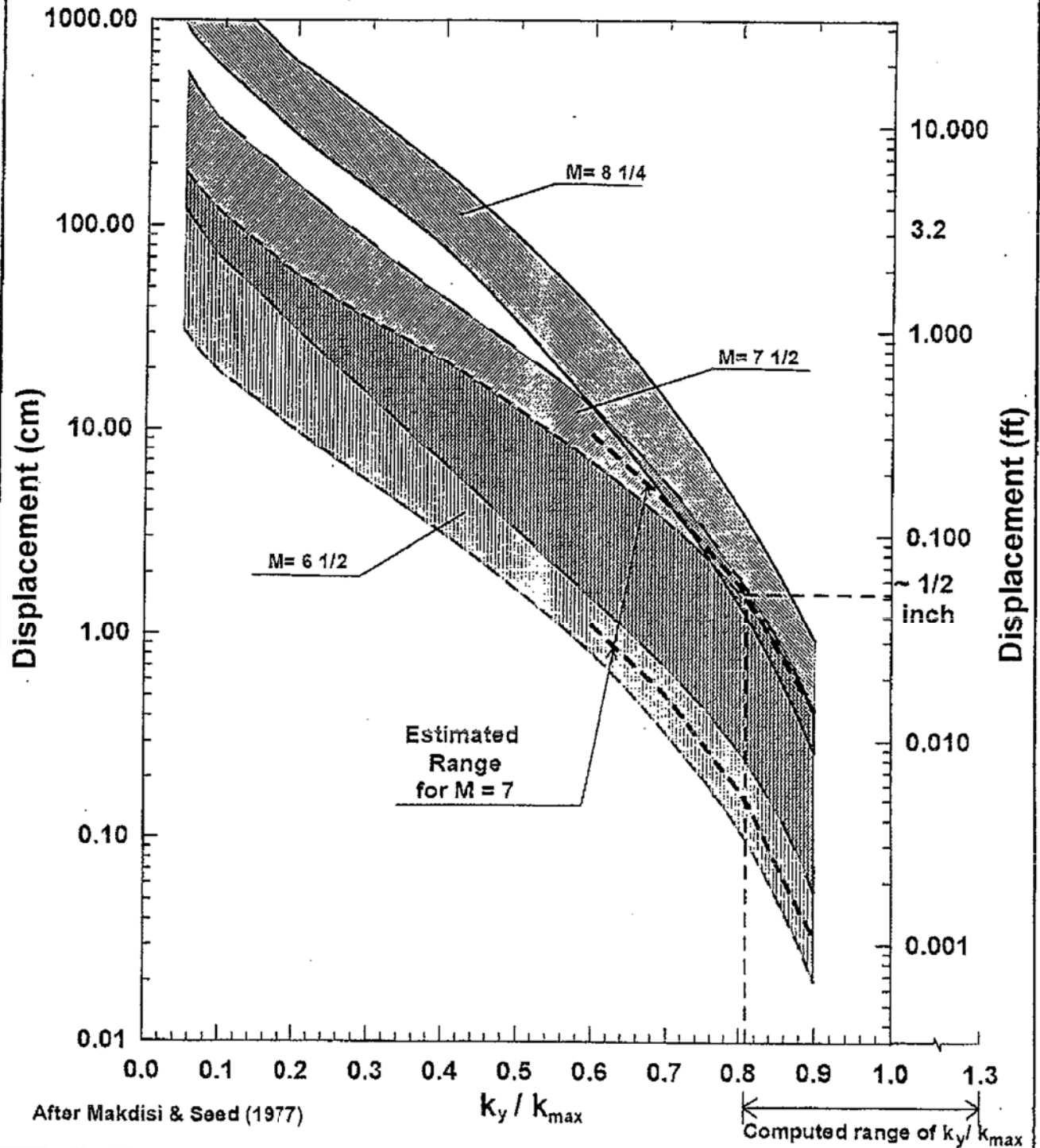
Date: SEP 95

Project:

SONGS CASK STORAGE

Fig. 4.4

CALC C-296-01.03 SHT A-44
APPENDIX A



Empirical Relationships Between Permanent Displacement
of Sliding Block and Ratios of Accelerations

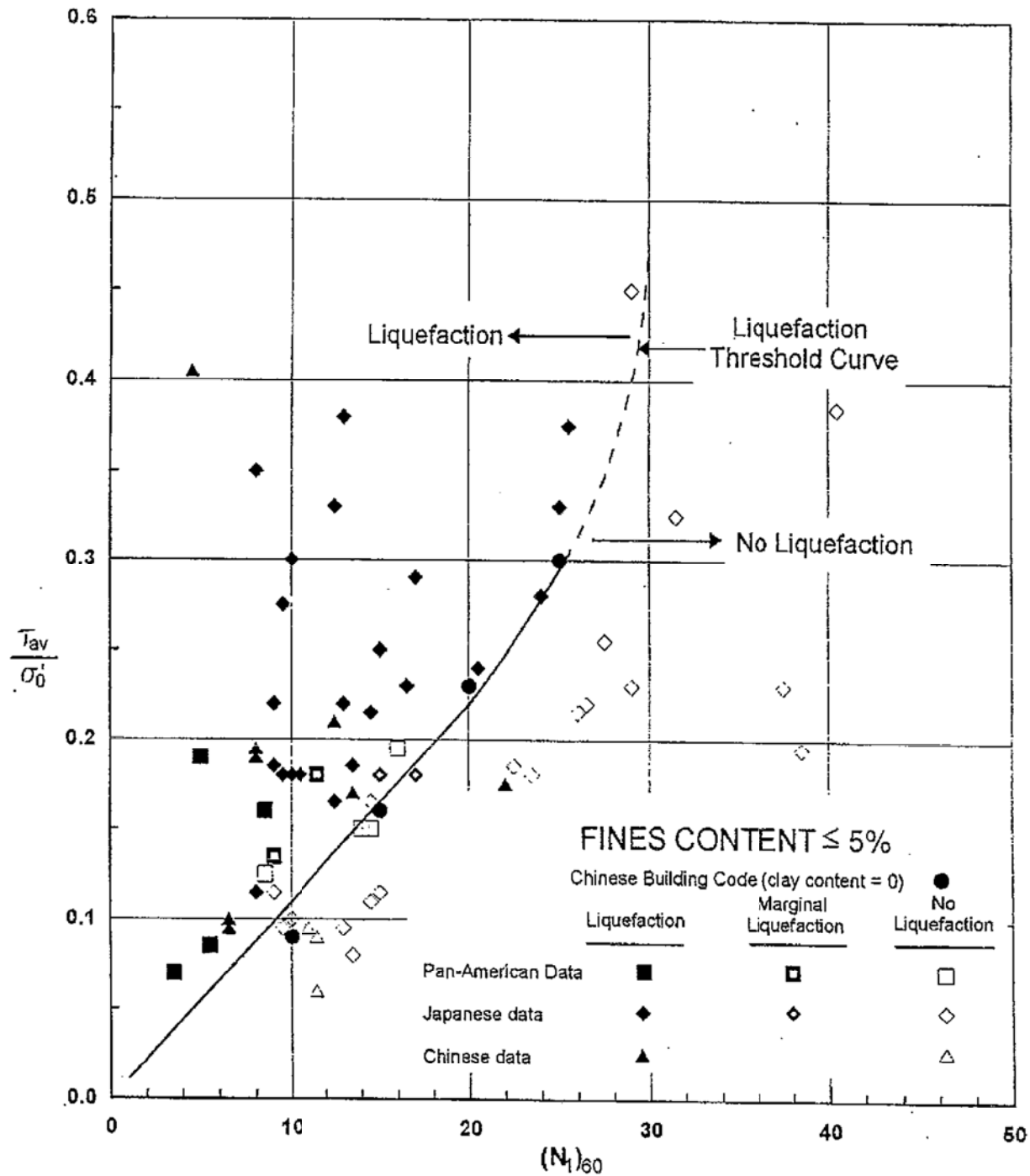
Project No. 954E181

Date: SEPT 1995

Project: SONGS CASK STORAGE

Fig. 4.5

CALC C-296-01.03 SHT A-45
APPENDIX A



Source: Seed et al. (1985)

Relationship Between Stress Ratios Causing Liquefaction and N_1 -Values For Clean Sands For $M=7.1/2$ Earthquakes

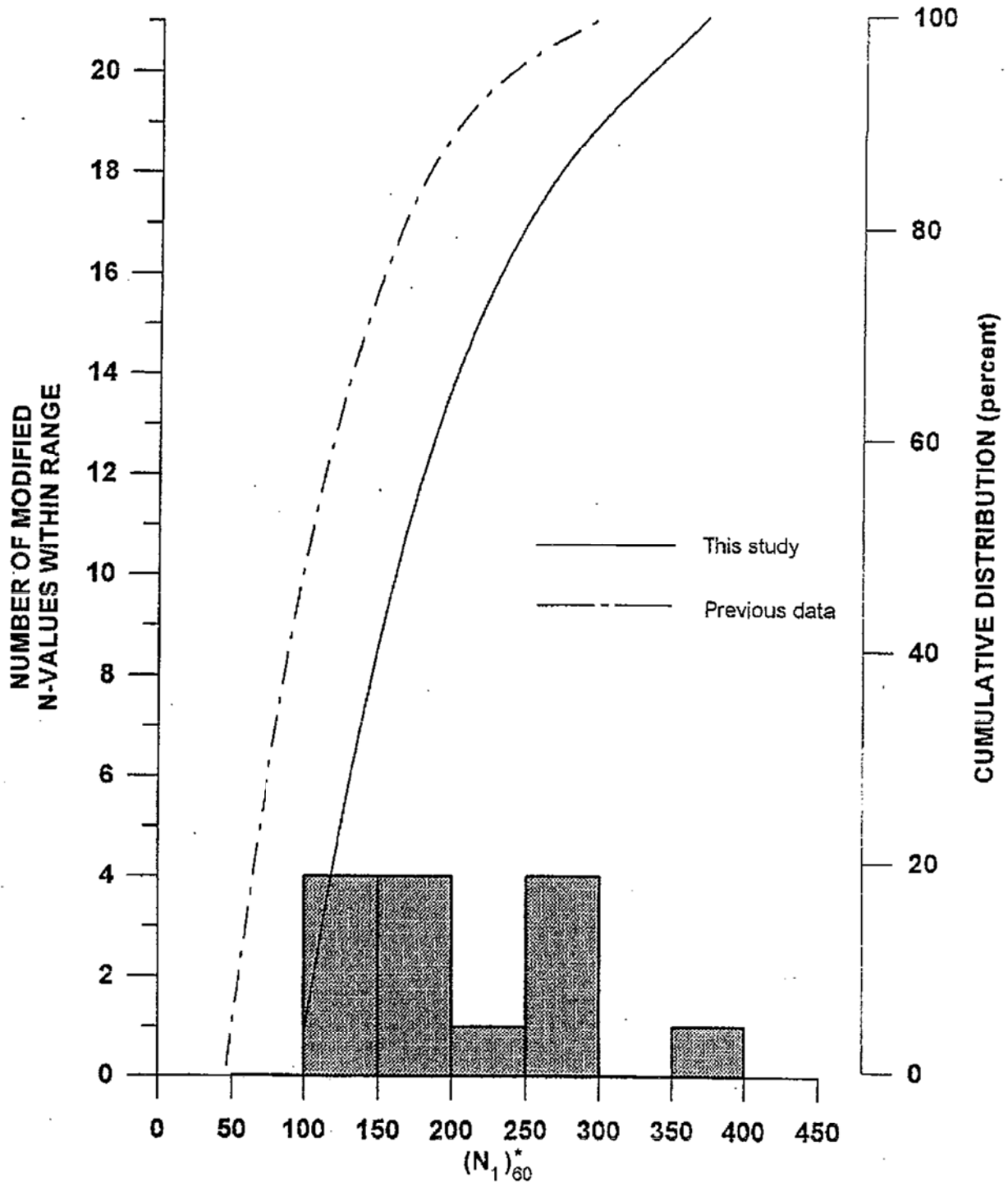
Project No. 954E181

Date: SEPT. 1995

Project: SONGS CASK STORAGE

Fig. 5.1

CALC C-296-01.03 SHT A-46
APPENDIX A



* NOTE: Most blowcounts estimated by linear extrapolation to equivalent penetration of 12 inches.

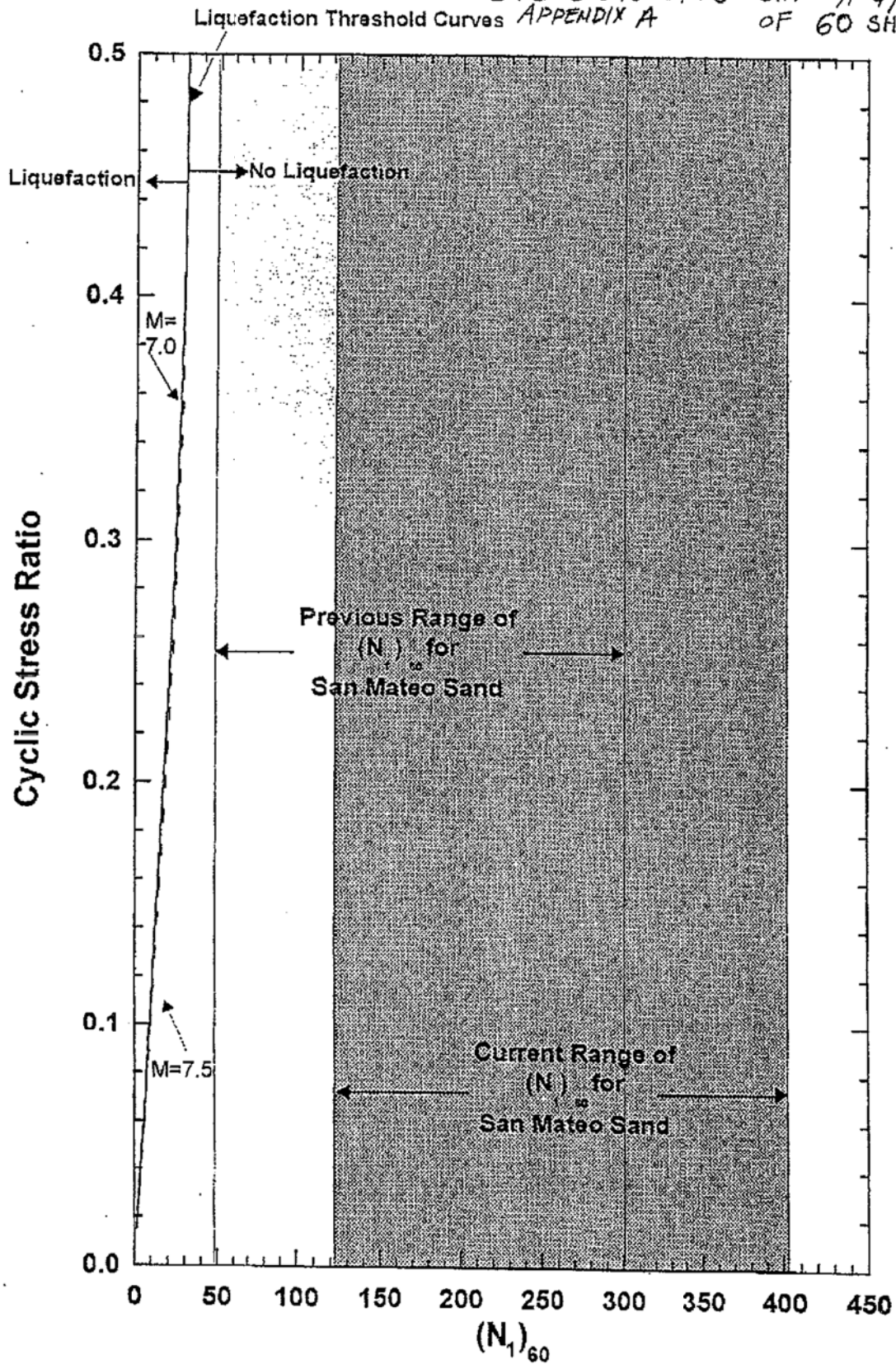
Histogram and Cumulative Distribution of Modified N-values in San Mateo Sand

Project No. 954E181

Date: SEPT. 1995

Project: SONGS CASK STORAGE

Fig. 5.2



Comparison of Cyclic Resistance vs. Liquefaction Threshold for San Mateo Sand

Project No. 954E181

Date: SEPT 1995

Project: SONGS CASK STORAGE

Fig. 5.3