Southern California Edison Company

P. O. BOX 800
2244 WALNUT GROVE AVENUE
ROSEMEAD, CALIFORNIA 91770

K. P. BASKIN
MANAGER OF NUCLEAR ENGINEERING, SAFETY, AND LICENSING

TELEPHONE
(213) 572-1401

April 18, 1983

Director, Office of Nuclear Reactor Regulation
Attention: D. M. Crutchfield, Chief
Operating Reactors Branch No. 5
Division of Licensing
U. S. Nuclear Regulatory Commission
Washington, D.C. 20555

Gentlemen:

Subject: SEP Topic II-4.F
San Onofre Nuclear Generating Station 50-206
Unit 1

In accordance with our letter dated January 31, 1983 enclosed are (1) a revised assessment for SEP Topic II-4.F, Settlement of Structures and Buried Equipment, and (2) revised Sections 1, 2, and 3 and the appendices of our August 17, 1982 report regarding backfill soil conditions. It is anticipated that revised Sections 4 and 5 of our August 17 report and documentation of additional settlement measurements and concrete crack mapping will be submitted by about April 31, 1983.

If you have any questions on this matter, please let us know.

Very truly yours,

K P Baskin

Limited Dist

A035
1/10

2 Copies Advanced
To: Cheng - SEPB
Pearring - SGEB

SYSTEMATIC EVALUATION PROGRAM

TOPIC II-4.F

SAN ONOFRE NUCLEAR GENERATING STATION, UNIT 1

TOPIC: II-4.F, Settlement of Structures and Buried Equipment

I. INTRODUCTION

This topic pertains to the review of plant geotechnical engineering aspects related to the properties and stability of subsurface materials and foundations as they influence the static and seismically induced settlement of critical structures and buried equipment.

II. REVIEW CRITERIA

A. 10 CFR Part 50, Appendix A

1. General Design Criterion 1 - "Quality Standards and Records." This criterion requires that structures, systems and components important to safety shall be designed, fabricated, erected, and tested to quality standards commensurate with the importance of the safety functions to be performed. It also requires that appropriate records of the design, fabrication, erection, and testing of structures, systems and components important to safety shall be maintained by or under the control of the nuclear power plant licensee throughout the life of the plant.

2. General Design Criterion 2 - "Design Bases for Protection Against Natural Phenomena." This criterion requires that safety-related portions of the system shall be designed to withstand the effects of earthquakes, tornadoes, hurricanes, floods, tsunami, and seiches without loss of capability to perform their safety functions.

B. 10 CFR Part 100, Appendix A, "Seismic and Geologic Siting Criteria for Nuclear Power Plants" - These criteria describe the nature of the investigations required to obtain the geologic and seismic data necessary to determine site suitability and identify geologic and seismic factors required to be taken into account in the siting and design of nuclear power plants.

III. RELATED SAFETY TOPICS AND INTERFACES

Geotechnical engineering aspects of slope stability are reviewed under Topic II-4.D. Other interface topics include:

II-3.B	Flooding Potential and Protective Requirements
II-3.C	Safety-Related Water Supply (Ultimate Heat Sink)
II-4.E	Dam Integrity
III-3.A	Effects of High Water Level on Structures

III-3.C In-Service Inspection of Water Control Structures
III-6 Seismic Design Considerations
IX-3 Station Service and Cooling Water Systems
XVI Technical Specifications

IV. REVIEW GUIDELINES

In general, the review process was conducted in accordance with the procedures described in Standard Review Plan (NUREG-0800) Section 2.5.4 (Reference 1). The geotechnical engineering aspects of the design and as-constructed conditions were reviewed and compared to current criteria, and the safety significance of any differences was evaluated.

The following Regulatory Guides provide information, recommendations, and guidance and, in general, describe a basis acceptable to the NRC staff that may be used to implement the requirements of the above described criteria.

A. Regulatory Guide 1.132, "Site Investigations for Foundations of Nuclear Power Plants." This guide describes programs of site investigations related to geotechnical engineering aspects that would normally meet the needs for evaluating the safety of the site from the standpoint of the performance of foundation and earthwork under anticipated loading conditions including earthquakes in complying with 10 CFR, Part 100, and 10 CFR, Part 100, Appendix A. It provides general guidance and recommendations for developing site-specific investigation programs as well as specific guidance for conducting subsurface investigations, the spacing and depth of borings, and sampling.

B. Regulatory Guide 1.138, "Laboratory Investigation of Soils for Engineering Analysis and Design of Nuclear Power Plants." This guide describes laboratory investigations and testing practices acceptable for determining soil and rock properties and characteristics needed for engineering analysis and design for foundations and earthwork for nuclear power plants in complying with 10 CFR, Part 100, and 10 CFR, Part 100, Appendix A.

V. EVALUATION

A. Site Description

The San Onofre Unit 1 site is located on the Camp Pendleton Marine Reservation on the coast of California in San Diego County about 51 miles northwest of San Diego and about 62 miles southeast of Los Angeles.

The topographic features of the immediate coastal area include a narrow band of beach sand terminating at seacliffs which reach a height of 60 to 80 feet in the vicinity of the site. A gentle coastal plain extends inland to the western foothills of the Santa

Margarita Mountain Range approximately 1-1/2 miles to the east. The plant site is on the shoreline. Prior to construction of the plant, the original plant site elevation at the top of the seacliff bluff ranged from +80 to +115 feet MLLW. The finished plant grade elevation is +20 feet MLLW. Recorded measurements indicate the ground water level to be about elevation +5 feet MLLW (Reference 2).

The subsurface soil structure exposed in grading and in excavation for plant facilities include the Quarternary terrace deposits which overlie a Pliocene age sand material named the San Mateo Formation. The terrace deposits consist of tan, buff, and light brown, silty or clayey, fine to coarse sand with some cobbles. These deposits are crudely stratified with a thickness of up to 55 feet. The San Mateo Formation is a cemented, massive well-graded, yellow-brown, fine to coarse sand with gravel and occasional lenses of thin-bedded gray shale or siltstone and is approximately 1,000 feet thick at the site. At grade, the San Mateo Formation is a poorly cemented but very dense sand.

To accommodate the plant, the seacliff bluff was cut back using a "Bench Design" approach. Cut slope profiles consist of a 15 foot wide bench at the interface of the terrace deposit and the San Mateo Formation. The San Mateo Formation comprises the lower 25 feet of the cut slope. Above and below the bench, the cuts were excavated to a slope of one horizontal to two vertical (Reference 2).

The main plant Seismic Category A facilities include a reactor containment structure and sphere enclosure building, a turbine building and turbine pedestal, an administration and control building, a circulating water system intake structure (pump well), a diesel generator building, a refueling water storage tank, and a seawall. All of the main facilities except portions of the ventilation equipment building, turbine building, refueling water storage tank, and seawall are bearing directly on undisturbed native San Mateo Formation sand. Figure 1 presents a site location plan for the San Onofre Nuclear Generating Station Unit 1 (SONGS 1) plant facilities.

B. Properties of Subsurface Materials

In situ material. The soil investigations at the San Onofre Unit 1 site were performed in September and October 1962. Foundation exploratory drilling was accomplished in May 1963. A total of 14 test holes were drilled at the site. The boring logs depicting the soil conditions encountered in these investigations have been presented in Reference 3. Field investigation efforts included standard penetration tests (SPTs) and soil sampling using a Pitcher rotary core barrel. Surface seismic refraction surveys were also made at the plant site using dynamite blasts as the energy source. Laboratory testing of soil samples was accomplished to determine

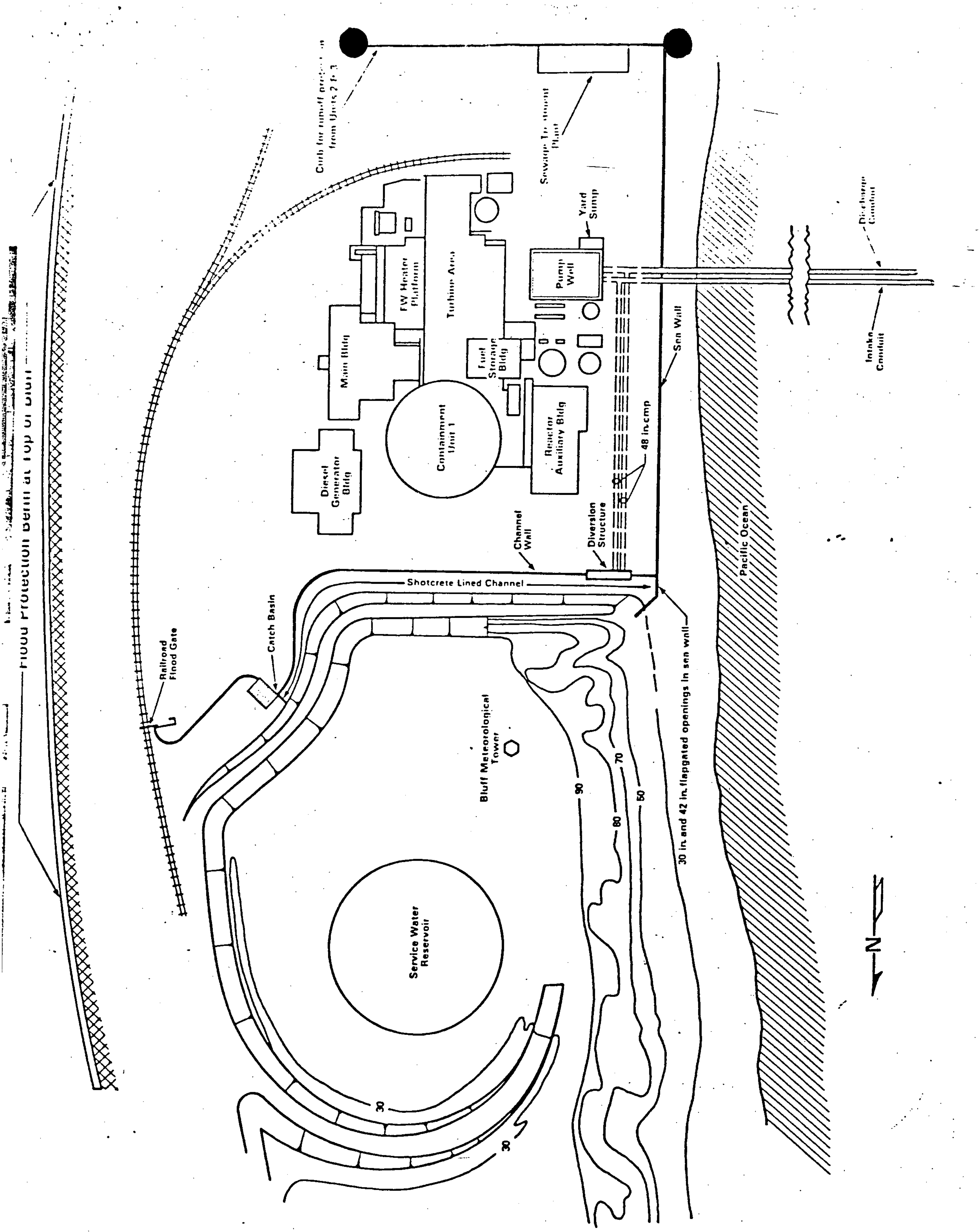


Figure 1 - San Onofre - General Site Plan

significant engineering characteristics and physical properties. Testing included specific gravity determinations, natural moisture content and unit weight determinations, particle size analysis, minimum and maximum relative density determinations, and consolidations and direct shear testing (Reference 3). Considerable additional field exploratory sampling and laboratory testing, including cyclic triaxial testing, was performed in 1972 to 1974 during geotechnical investigations associated with the San Onofre Units 2 and 3 project (Reference 4). Table 1 presents soil strength parameters for undisturbed native San Mateo Formation sand which were developed from the results of the site subsurface and laboratory investigations and used in foundation settlement analyses.

Based on the information presented it is concluded that the scope of field and laboratory testing was adequate to define conservative strength parameters for the undisturbed San Mateo Formation sand.

Backfill material. At the San Onofre Unit 1 site, backfill was originally assessed (References 2, 3, 5) to be reconstituted San Mateo Formation sand compacted to a minimum of 95% Modified Proctor density. The reported results of 55 in place density tests performed in 1964 and 1965 in the reservoir area, around the circulating water screenwell, and in the area of miscellaneous buried utilities indicated that all but 16 areas tested initially met the required 95% Modified Proctor density specification. Field notes presented with the test results indicated that backfill areas where initial tests did not indicate 95% density were reworked and retested.

In April 1982, SCE notified the NRC that backfill soil consisting of San Mateo Formation sand compacted to less than 95% Modified Proctor density was encountered during ongoing modifications for the turbine building. Investigations to evaluate the in situ density of backfill material were undertaken by SCE and a report of field observations and backfill characteristics for the turbine building footing modifications of the north extension and west heater platform was submitted in August 1982 (Reference 6). The report presented the results of 84 in place density tests accomplished in February to May 1982 including 73 tests in the immediate area of the turbine building modifications and 11 tests in miscellaneous utility trench backfill areas at the site. The results indicate that the tested backfill materials were found to be compacted to a density between 80% and 100% Modified Proctor density. Of the 84 tests reported, 47 (56%), 29 (34%) and 9 (10%) tests were below 95%, 90%, and 85% Modified Proctor density, respectively.

Table 1. Summary of soil properties for undisturbed San Mateo Formation Sand - SONGS I Site

Soil Property	Values
o Unified Soil Classification	SW
o Natural Water Content	
Above water table (%)	2
Below water table (%)	11
o Dry Unit Weight (lb/CF)	120
o Shear strength	
Cohesion (K/SF)	0.75
Friction Ø (degrees)	41.5
o Standard Penetration Test (blows/ft)	>100
o In Situ Relative Density (%)	100
o Plasticity Index	Non Plastic
o Shear Modulus, G (lb/SF)	100 Km (σm) 2/3
Km (0.0001% strain)	590
Km (0.001% strain)	315
Km (0.01% strain)	150
Km (0.1% strain)	60
o Poisson's Ratio	0.35
o Seismic Compressional Wave Velocity	
Above water table (FPS)	3000
Below water table (FPS)	7000
o Seismic Shear Wave Velocity	
Above water table (FPS)	1000 - 1200
Below water table (FPS)	1900 - 2750
o Water Table Elevation (feet MLLW)	+5

Furthermore, Reference 6 provided an evaluation of the affected structures, systems and components due to the areal extent and the in situ condition of backfill areas. Consideration was given to the potential dynamic settlement and liquefaction of backfill material due to the DBE event of 0.67g's as well as changes in the dynamic characteristics of the soil-structure interaction parameters. Where it was found that the safety of the structures, systems and components was impaired, modifications are being implemented to restore the design margins consistent with the Systematic Evaluation Program (SEP) criteria.

Recently, the areal extent and the in situ condition of the backfill soils were reassessed based on numerous construction photographs, discussions with construction personnel, excavations and field observations made since the report submittal and photogrammetric analysis of selected construction photographs. This information is reported in Attachment I, which is an update of information presented in Reference 6. In parallel, field surveys were made to update the settlement records and to map possible concrete cracking in selected structures that could be affected by settlement of backfill. The data from these surveys will be presented in a subsequent report.

The updated information on the areal extent and the in situ condition of backfill areas are summarized in the sections below. Based on this information, a reassessment of the safety of the structures, systems and components is being undertaken.

C. Settlement Evaluation

The observed settlement records for several seismic Category A structures covering the period 1964 through 1970 were submitted in (Reference 7). Settlement observations were initiated during construction by establishing settlement markers at strategic locations associated with major structures, and subsequent observations were recorded at varying time intervals. The observations show that settlement of the containment building foundation, and all other structures founded entirely upon undisturbed San Mateo Formation sand was less than 0.4 inches during the period of record. Because of the high in place relative density (100%) and in place strength properties of undisturbed San Mateo Formation sand, liquefaction and subsequent structural settlements are not possible under the dynamic loading conditions associated with an SSE event of 0.67g.

As noted above, additional field surveys have been made recently to update the settlement records and to map concrete cracking in selected structures which could be affected by backfill soils. These structures include the 480 V room slab and the ventilation equipment building. The evaluation of these surveys will be presented in a subsequent report.

Specific aspects of the backfill soil conditions are discussed in the following subsections. Specifically, these sections address the areal extent, potential settlement, shear moduli reduction factors and liquefaction potential of backfill soils.

C.1 Areal Extent of Backfill Soils

At the start of construction for the seismic upgrade modifications, SCE initiated a detailed evaluation of the in situ backfill soil conditions throughout the site to determine the in place engineering properties of the existing backfill materials and the significance of the change in backfill characterization on the seismic reevaluation of affected structures. The initial results of this evaluation were provided in a report entitled, "Report of Soil Backfill Condition - San Onofre Nuclear Generating Station Unit 1," which was submitted to the NRC by SCE in August 1982 (Reference 6).

Recently, the areal extent of the backfill soils and the densities of the backfill materials have been further evaluated based on numerous construction photographs, discussions with construction personnel, excavations and field observations made since the August report submittal, and photogrammetric analysis of selected construction photographs. The current assessment is presented in Attachment I. Specifically, the updated characterization of the areal extent of backfill is shown in Figure 2-22. Cross-sections through the backfills are shown in Figures 2-23 to 2-26.

C.2 Settlement of Backfill

Estimates of settlement for fills above the water table were based on procedures suggested by Silver and Seed (Reference 8). The procedure incorporates relative density of fill, duration of shaking in terms of the number of cycles of cyclic loadings and the estimated level of applied cyclic shear strains, and estimates the resulting volumetric strains in the soil. For fills below the water table, estimates of settlement were obtained using the procedures suggested by Lee and Albaisa (Reference 9). This procedure also estimates the volumetric strain in cohesionless soil subjected to various levels of pore-pressure increments, in terms of the ratio of excess pore pressure to initial effective confining pressure. The procedure incorporates the relative density of the soil and the initial effective confining pressure. For the purpose of this evaluation, the settlement estimates were based on the backfill characterization at the location of the individual foundations shown in Figure 2-22 of Attachment I. In addition, the induced pore pressure, summarized for the various fill categories in Attachment I, Figures 3-1 through 3-4, together with the number of cycles of cyclic loading, given in Attachment I, Appendix C, were utilized in arriving at the estimates for settlement of the various foundations. The specific procedure for estimating settlements is shown by example in Attachment I, Appendix D.

Based on these evaluations, settlement and tilting of affected foundations are postulated when backfill material consisting of reconstituted San Mateo Formation sand compacted to less than 95% Modified Proctor density undergo dynamic loading associated with a 0.67g earthquake. Affected structures, systems and components will be evaluated for the effects of the calculated settlement and tilting to assure that their safety is not impaired. Modifications will be implemented to restore design margins where necessary.

C.3 Shear Moduli Reduction Factors

Shear moduli reduction factors that convert the shear modulus for San Mateo sand, compacted to a minimum of 95 percent relative compaction, to a shear modulus that is appropriate for a lower relative compacted San Mateo sand were developed considering data from Seed and Idriss (Reference 10). This is discussed in detail in Attachment I, Chapter 3 and Appendix F. Variation of damping characteristics with strain is also presented therein.

C.4 Liquefaction Potential of Backfill

The behavior of backfill soils at the site in response to seismic loading depends on the intensity of ground shaking, the density and geometry of the backfill, and the proximity of the water table. Liquefaction at this site is not a flow phenomenon because the surface slope is flat, and all of the fills are contained within limited areas. Liquefaction at the site is therefore defined as the potential for the development of pore water pressure with limited strain potential. A detailed evaluation of the liquefaction potential at San Onofre Unit 1 is presented in Attachment I, Chapter 3 and Appendix C. Backfill soils with a relative compaction of 85 percent and located below the water table have a higher potential for liquefaction. Backfills with higher relative compaction, on the order of 92 percent, have a lower potential for liquefaction but may develop high pore water pressures depending on the density and fill geometry. San Mateo sand material compacted to greater than 95% relative compaction will not liquefy.

VI. CONCLUSIONS

Based on the above information, it is concluded that:

1. There will be no static or dynamic settlement problems associated with a Seismic Category A structure or component founded upon the undisturbed San Mateo Formation sand at the site.

2. The areal extent and engineering properties of backfill materials at the site have been evaluated in detail. Where necessary, modifications will be implemented to restore design margins. Therefore, there will be no static or dynamic settlement problems

associated with any Seismic Category A structure or component which was founded upon or adjacent to reconstituted San Mateo Formation sand at the site.

VII. REFERENCES

1. "Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants - LWR Edition," NUREG-0800, July 1981

2. Final Safety Analysis Report, San Onofre Nuclear Generating Station, Unit 1, Docket No. 50-206

3. Southern California Edison Company letter R. W. Krieger to D. M. Crutchfield, Subject: SEP Topics II-4.D and II-4.F, San Onofre Nuclear Generating Station, Unit 1, dated February 1, 1982

4. Final Safety Analysis Report, San Onofre Nuclear Generating Station, Units 2 and 3, Docket Nos. 50-361/362

5. Southern California Edison Company letter W. C. Moody to D. M. Crutchfield, Subject: "SEP Topics II-4.D and II-4.F, San Onofre Nuclear Generating Station Unit 1," dated November 2, 1981

6. "Report of Soil Backfill Conditions - San Onofre Nuclear Generating Station, Unit 1," August 12, 1982, submitted under cover letter from K. P. Baskin, Southern California Edison to D. M. Crutchfield, NRC, dated August 17, 1982, Subject: "In-situ Soil Conditions, SEP Topic III-6, Seismic Design Considerations, San Onofre Nuclear Generating Station, Unit 1"

7. Southern California Edison Company - Technical Drawing, "San Onofre Nuclear Generating Station Unit 1 Settlement Observation Markers," Drawing Numbers 8202080430 and 8202080435, Revisions 1-4, dated April 17, 1975

8. M. L. Silver and H. B. Seed - "Settlement of Dry Sands During Earthquakes," ASCE Journal of the Soil Mechanics and Foundation Division, Vol. 98, No. SM4, April 1972

9. K. L. Lee and A. Albaisa - "Earthquake Induced Settlement in Saturated Sands," ASCE Journal of the Geotechnical Engineering Division, Vol. 100, No. GT4, Proc. Paper 10496, April 1974, pp. 387-406

10. H. B. Seed and I. M. Idriss - "Soil Moduli and Damping Factors for Dynamic Response Analyses," University of California Earthquake Engineering Research Center, Report No. FERC 70-10, December 1970

JLR:7207

Attachment 1

SOIL BACKFILL CONDITIONS

SAN ONOFRE NUCLEAR GENERATING STATION

UNIT 1

Revision 1

March 1983

1.0 INTRODUCTION

1.1 Background

The overall site soil conditions present at the San Onofre site are reported in Reference 1. The results and the soil parameters described therein are applicable to the native San Mateo formation.

At the beginning of the Systematic Evaluation Program (SEP) Seismic Reevaluation, the backfill at the San Onofre Unit 1 site was assessed to be San Mateo sand having a minimum relative compaction of 95 percent. Therefore, the soil parameters developed for the backfill and used in the SEP analyses were based on this assessment (References 2 and 3).

In a letter to the USNRC dated April 30, 1982, SCE indicated that as a result of soil testing conducted during the construction of the seismic upgrade modifications for the turbine building during the current outage, it was discovered that fill soil with relative compaction less than 95 percent was present. In the local areas where this was encountered, remedies were implemented. In addition, SCE committed to investigate the potential for similar conditions in other areas of the site and to resolve the potential impact of such conditions on the seismic analyses. A preliminary report of the "Soil Backfill Conditions" dated August 12, 1982 was transmitted by the August 17, 1982 SEP Topic III-6 letter from SCE to USNRC.

1.2 Purpose and Organization

The purpose of this report is to provide an updated characterization of the fill soils at San Onofre Unit 1 from the August 12, 1982 report. Additionally, where differences are identified between this characterization and the corresponding basis for the seismic reevaluation of the various structures, systems and components, the effects of the differences are assessed and resolved.

This report is comprised of five sections including this introduction. Specifically, in Section 2, the site backfill conditions are characterized based on a thorough review of site grading drawings, construction photographs, the documentation of San Onofre Unit 1 compaction testing during the original construction, and more recent observations and testing performed in conjuction with various plant modifications.

Section 3 provides a detailed description of the backfill behavior during a 0.67g Housner Design Basis Earthquake (DBE) event. The methodology which is used to determine the effect of the backfill on the SEP seismic reevaluation analysis parameters is also discussed.

Section 4 addresses the specific analysis effects of the soil fills for each of the structures. The significance of the soil fill analysis effects on the previously completed seismic analysis of the structures is evaluated and described.

Similarly, Section 5 addresses the specific backfill soil behavioral effects which are pertinent to the affected safety related systems and components. These effects and results are then evaluated for each individual component. If further resolution is necessary, conceptual modifications are identified which, when implemented, will either preclude the cause of the effects, or adequately mitigate the effects on the component.

2.0 SOIL BACKFILL CHARACTERIZATIONS

This section describes the characterization of the backfill areas at the site. The development of the characterization involved: defining the backfill areas, evaluating the available information on the relative compaction of the fill in the various areas to identify the amount of compaction; characterizing the fills in accordance with the degree of compaction; and assigning an appropriate category to the backfill in each area. The sections that follow describe these steps.

2.1 Areal Extent of Backfill

The first step in defining the backfill areas was the examination of the original site grading plan and the available construction photographs. The plan dimension and locations of structures, as shown on the original excavation plan, were determined to be correct. However, the planned excavation slopes (shown as 1:1 on the original grading plan) were not the same as those shown in the actual construction photographs. Based on an interpretation of numerous construction photographs, discussions with construction personnel, field observations made during subsequent plant excavation (see Section 2.2), and the subsequent photogrammetric analysis of selected construction photographs (Appendix A), it was determined that the actual slopes of the construction cuts were 1/2:1. In addition, a working space of about two to three feet between a structure and the base of the excavation slope was considered to be consistent with the apparent construction procedures used and with photographic evidence. An excavation plan was drawn depicting the tops and bottoms of the excavations using two to three feet of working space around structures and 1/2:1 cut slopes except where photographic evidence indicated otherwise. Areas between the excavation slopes and structural walls were designated as backfill areas. In addition, areas above the anchor blocks, where the finished grade was higher than the elevation of the top of the anchor blocks, were designated as backfill areas. Based on these considerations a site plan showing the areal extent of the backfill was prepared and is presented in Figure 2-1. A water table elevation of +5 feet is used to distinguish between fills which are above and below the water table.

2.2 Characterization of Backfill Compaction

After defining the backfill areas at the site, as discussed in Section 2.1, the compaction of backfill was characterized based on all available information. This information consisted of: results of field tests made during the original construction; observations and tests made in utility trench excavations constructed subsequent to initial construction; observations made during the construction of the sphere enclosure building and the diesel generator building; and observations and tests made during the recent foundation modifications for the seismic upgrade program.

During the initial construction of the plant, field density tests were made by Twining Laboratories. Tests made in the power block area are summarized in Table 2-1. It is noted that the degree of compaction, as reported by Twining Laboratories, was based on a laboratory maximum dry density of 121 pcf. More recent tests made over the past 8 years with the San Mateo sand from San Onofre Units 1, 2, and 3 following the ASTM D 1557-A procedure indicate that a more representative laboratory maximum dry density is 120 pcf. Therefore, the percent relative compactions shown in Table 2-1 are based on a maximum dry density of 120 pcf. The use of a maximum dry density of 120 pcf for San Mateo sand provides a consistent basis for comparisons of relative compaction in the backfills evaluated here and at other locations at the site. Figure 2-2 is a location map for Figures 2-3 through 2-9 which summarize, in plan view, the approximate locations of these tests and their results.

Subsequent to the original construction, backfill observations were made for plant modifications including utility excavations, construction of foundations for the sphere enclosure building and the diesel generator building, the turbine building footing modifications, the auxiliary feedwater piping trench excavation, and the auxiliary feedwater tank excavations. In addition, test pits were excavated in the areas of the refueling water storage tank, ventilation equipment building, and the reactor auxiliary building. These backfill observations consist of field tests and/or probing (with a 3/8-inch diameter, 3-ft long steel probe) as excavations progressed. The location of the tests, results, and observations were carefully documented. The approximate locations and results of these tests are summarized in Figures 2-3 through 2-9. Summaries of the observations made in utility trench excavations are presented in Table 2-2. At the time the observations summarized in Table 2-2 were made, it was concluded that they were the result of placement of uncontrolled utility trench backfills and not representative of the generic condition of areal backfills. Field density tests and observations made on soil exposed in excavations for foundations for the sphere enclosure building are summarized in Table 2-3. The results of field tests and observations made during the turbine building footing modifications and recent excavations and testing for the auxiliary feedwater piping trench, refueling water storage tank, auxiliary feedwater tank, ventilation equipment building and reactor auxiliary building, are included in Appendix B. The observed conditions at the bases of various excavations made for these areas are shown in Figure 2-10. The legend notes in Figure 2-10 describe the observations made during the excavations. The "daylight" lines between backfill and native soil which were observed during the footing excavations were checked against the areal distribution of fill as delineated in Figure 2-1 and were determined to be in good agreement.

Specific observations shown in Figure 2-10 for the turbine building footing modifications have also been documented in ten selected cross sections (located in Figure 2-10) of the excavations. These are shown in Figure 2-11 through 2-17. For example, the first two cross sections are located through the west leg of the northwest turbine building footing as shown in Figure 2-11. At this location, adjacent to the east end of the Fuel Storage Building, the existing soil backfill against the Fuel Storage Building was found to have an average density of 85 percent relative compaction and was overexcavated as shown. During the excavation process the daylight line between fill and native soil shown by the dark line in Figure 2-11 was observed and measured above about elevation 0 and probed below elevation 0. This daylight line shows a slightly steeper than a 1/2:1 slope at the base of the excavation and a slightly flatter than 1/2:1 slope near the top of the excavation. This observation, together with others made, for the turbine building footing modifications are summarized in Table 2-4 and are keyed to the cross sections shown in Figures 2-11 through 2-17. As indicated above, all of these observations are in agreement with the distribution of fill shown in Figure 2-1. The observations made during the excavation for the foundation of the auxiliary feedwater tank and the auxiliary feedwater piping trench are also summarized in Table 2-4. In addition to the plan views showing the locations of the observations described above, the test results were also plotted on cross sections to aid in characterizing the backfill in the various locations. Ten cross sections showing the configuration of the backfill at various locations and the results of the field tests are presented in Figures 2-18 through 2-21.

2.3 Characterization of Backfills

To characterize backfills at various locations, the configuration of the backfill in the excavations was also considered with regard to the amount of working space and the type of compaction equipment observed in construction photographs. Based on this information and the information presented in Sections 2.1 and 2.2, the backfills delineated in Figure 2-1 were characterized into four general categories as described below and as shown in Figure 2-22. Remedial measures undertaken during recent earthwork activities have changed the conditions locally from what is shown in Figure 2-22 as described in Table 2-4 and as discussed in Section 2.4.

Category A - This characterization represents well compacted backfill, with a minimum relative compaction of 95 percent. As shown in Figure 2-22, the area with this type backfill is located mainly over the discharge culvert. Cross-sections presented in Figures 2-19 and 2-20 show that the backfill in this area is wide and placed over a relatively flat base. In addition, construction photographs show compaction equipment being used in this area. Tests made in the area of the turbine building southwest footing modification which are summarized in Figure 2-6 (see Appendix B), indicated high levels of relative compaction.

Category B - This characterization represents moderate to well compacted backfills, with relative compaction of 90 to 95 percent. Backfills of this category are located near the intake structure, between the intake culverts near the south end of the Turbine Building and the shallow fill south of the Reactor Auxiliary Building as shown in Figure 2-22. The characterization and distribution of Category B fill is based on the available information which includes: the Twining test data summarized in Table 2-1 and Figures 2-3, 2-4, 2-6, 2-7, and 2-8; the utility trench observations summarized in Table 2-2; tests and observations made for the turbine footing modifications and other foundation and trench excavations, summarized in Figures 2-3 through 2-6 and 2-8 and cross sections presented in Figures 2-19 through 2-21. In addition, deep narrow fills in these areas, with widths of less than 6 to 10 ft, are assumed to have a lower degree of compaction, which is estimated to be about 85 percent, because of the difficulty of access and maneuvering of compaction equipment.

Category C - This characterization represents moderately compacted backfills, with relative compactions of 85 to 90 percent. In addition, deep narrow fills in these areas (widths of less than 6 to 10 ft), are assumed to have a lower degree of compaction (estimated to be about 85 percent). This characterization includes areas adjacent to the intake structure, in the area of the screen well and tsunami gates, and in the east turbine extension area as shown in Figure 2-22 and on cross sections in Figure 2-24 and 2-26. It is based on data presented in Table 2-2, Table 2-4, and Appendix B and tests shown on cross sections in Figures 2-19 and 2-21.

Category D - This characterization represents backfills with an estimated 85 percent relative compaction. This is based on observations made during construction of a portion of the north extension footing during the recent turbine building footing modifications (Appendix B), as shown in Figures 2-3 and 2-6, and on observations made for miscellaneous pipe support foundations, which are summarized in Table 2-2. These fills have been defined to include narrow, long areas around structures, where it was difficult to maneuver compaction equipment and where a high degree of compaction may not have been considered essential at the time of construction. As shown in Figure 2-22, the fills in this category include the areas around the reactor auxiliary building, the fuel storage building and vent stack foundation, narrow fills around the turbine mat, between the west anchor block and the discharge culvert, and shallow, narrow fills around the control building.

The degree of compaction which is shown to be 85 percent relative compaction for categories B, C, and D fills represents an average value based on the results of field density tests and probings discussed in Section 2.2.

The backfill conditions summarized in Figure 2-22 are considered conservative because in those areas where limited or no data were available, the lowest average conditions were assigned from areas with available data.

2.4 Remedial Measures Implemented During Footing Construction

Some of the new footings for the sphere enclosure building and for the recent turbine building modifications are located within the backfills placed during the original plant construction.

A summary of the specific remedial measures which were implemented to accommodate backfill conditions which were encountered during the construction of the turbine building foundation modifications is presented in Table 2-4. In general, if the fill exhibited a density beneath a new footing of less than 95 percent relative compaction, the soil was overexcavated and the footing base extended to native soil or the structural loads were transferred to other structural elements which are supported by native soil. The overexcavated area was backfilled with lean concrete or the soil was compacted to 95 percent relative compaction. When backfill adjacent to a foundation had a density below 95 percent relative compaction the backfill was generally left in place and the foundation stiffness parameters were modified to reflect this condition as discussed in Section 3. The final footing configurations reflected by these changes are shown on the cross sections shown in Figures 2-11 through 2-17 and Figures 2-23 through 2-26.

Table 2-4 also includes a description of the overexcavation remedial measure which was undertaken during the construction of the sphere enclosure building foundation. As shown in Figure 2-9, only a very minor portion of the foundation was affected. Also, observations made in the area of the auxiliary feedwater pipe trench, the auxiliary feedwater tank foundation, and the refueling water storage tank foundation are presented in Table 2-4.

TABLE 2-1
SUMMARY OF FIELD TEST RESULTS BY TWINING LABORATORIES IN POWER BLOCK AREA

Test No.	Date	Location	Approximate Elevation of Test (ft)	γ_d Field (pcf)	Moisture Content Field (%)	Relative Compaction* (%)	Comments (from Twining Reports)
27	16 Dec 64	Pump Well Area East of Intake Structure	-10	114.8	8.7	95.7	
28	16 Dec 64	Pump Well Area East of Intake Structure	-10	107.4	6.4	89.5	Retested, new designation as test No. 33
29	16 Dec 64	North of Intake Structure	-10	115.1	8.7	95.9	
30	12 Dec 64	North of Intake Structure	-10	111.6	7.5	93.0	Does not meet the required 95% compaction
31	12 Dec 64	South Side of Field Storage Building	+12	108.6	3.6	90.5	To be retested
32	12 Dec 64	South Side Field Storage Building	+12	106.4	3.6	88.7	To be retested
33	18 Dec 64	Pump Well Area East of Intake Structure	-10	115.1	8.1	95.9	
34	18 Dec 64	North Side of Field Storage Building	+13	115.1	5.3	95.9	
35	18 Dec 64	North Side of Field Storage Building	+13	109.2	8.1	91.0	
36	12 Jan 65	South of Turb-Ped Mat.	+14	115.2	4.7	96.0	
37	12 Jan 65	South of Turb-Ped Mat.	+14	115.4	5.8	96.2	
38	12 Feb 65	Between Intake Culverts	2.0	119.9	9.9	99.9	
39	12 Feb 65	Between Intake Culverts	2.0	119.2	8.3	99.3	
40	12 Feb 65	Top of Discharge Culvert - East End	8.0	119.0	7.5	99.2	
41	12 Feb 65	Top of Discharge Culvert - East End	8.0	119.8	8.3	99.8	
42	24 Feb 65	South of Screen Well	-1	103.6	5.3	86.3	
43	24 Feb 65	South of Pump Well	+4	112.1	7.5	93.4	
44	24 Feb 65	South of Pump Well	+4	113.5	8.1	94.6	
45	24 Feb 65	North of Intake Culverts	-6	113.8	6.4	94.8	
46	24 Feb 65	West Side Screen Well	-1	114.5	8.7	95.4	
47	24 Feb 65	North of Screen Well	-1	117.2	9.9	97.7	
48	24 Feb 65	North Side Pump Well	-1	117.1	9.9	97.6	
55	23 Mar 65	South of Screen Well	8.0	119.5	8.1	99.6	
56	23 Mar 65	South of Screen Well	8.0	118.8	12.4	99.0	
57	23 Mar 65	South of Pump Well	13.0	117.8	7.5	98.2	
58	23 Mar 65	Area 12 Over Discharge Culvert	13.0	111.4	5.6	92.8	
59	23 Mar 65	Area 13	13.0	120.5	8.1	100.4	
60	23 Mar 65	Area 11 Over Discharge Culvert	18.0	118.0	7.0	98.3	
61	23 Mar 65	South of Intake Structure	13.0	107.4	3.6	89.5	

* Relative compaction based on a laboratory maximum dry density of 120 pcf.

TABLE 2

SUMMARY OF OBSERVATIONS MADE IN VARIOUS EXCAVATIONS

Date	Excavation Description	Approximate Depth (ft)	Location	Observation*
Mar 76	Manhole Excavation	10-20	Electrical manhole structure 710A and 711A	Backfill with relative compaction of 87 percent to an unknown depth.
Apr 76	Sphere Enclosure Foundation	7	Southwest of column no. C-2 near the Fuel Storage Building	Backfill with estimated relative compaction of less than 95 percent to about el. +7 was removed and replaced with 95 percent compacted backfill.
Oct 76	UPS Trench Backfill	8	West and south side of trench next to manhole nos. 743 & 744	Backfill with relative compaction of about 85 percent to a depth of at least 8 ft (el. +6).
Sep 77	Catch Basin #5	12	South of screen well	Backfill with relative compaction of about 85 percent from surface (el. +14) to at least el. +2.
Oct 77	Utility Trench	6	South of pump well between column lines K & L and west of column line 13	Backfill with relative compaction of about 85 percent from surface (el. +14) to at least el. +8.
Oct-Nov 77	Trenches for Misc. Piping	11	West of pump well near screen well	Backfill with relative compaction of about 85 percent from surface (el. +14) to at least el. +3.
Feb 78	Chlorination Tank Pad and Yard Sump	3	South of intake structure near west wall of pump well	Backfill with relative compaction of about 85 percent from surface (el. +14) to at least el. +11.
Jun 78	Cathodic Protection Boring	9-10	Between pump well and screen well	Backfill with relative compaction of 90 to 95 percent from surface (el. +14) to a depth of 4 to 6 ft and with relative compaction of about 85 percent below el. +15.
May 80	Miscellaneous Footings	5	East of anchor block, north of column line 1	Backfill with relative compaction of about 85 percent from surface (el. +14) to at least el. +9.
Dec 80 - Jan 81	Miscellaneous Pipe Support Footings	4	Against north and west walls of Fuel Storage Building	Backfill with relative compaction of about 85 percent from surface to bottom of excavation. Probing indicated loose soil to additional depth of at least 3 ft (el. +7).
Jan 82	Miscellaneous Footings	5	Against north wall of Fuel Pool near northeast corner of Fuel Storage Building	Backfill with relative compaction of about 85 percent from surface (el. +19) to at least el. +14.

* Observation interpreted from field notes and on discussions with field personnel. Approximate, relative compaction estimated by using a 3/8-inch diameter, 3-ft long steel probe.

TABLE 2-3 - Summary of Field Density Tests in Foundation Excavations, Sphere Enclosure Building

Field Data Sheet

Sheet No. 1

Job Name: SONGS Unit 1 Sphere Enclosure Building

Job Number: B675F

1976 Date	Test Number	Retest by	Retest of	Grid Number	Location of Test	Elev.	Field Dry Density (pcf)	Moist. %	Method	Max. Lab. (pcf)	Rel. Comp. %	Spec. Reg. %	Drawing No., Spec.	Qual Clas
Apr 07	1			S9+57 W5+49	Blow Down Header	15	124	6	SC	120	103	95	[illegible]	SR
Apr 08	2			S9+58 W5+50	" " "	17	118	9	"	"	99	"	"	"
Apr 13	3			S9+43 W5+17	Column C-7	13	123	10	"	"	102	"	"	"
"	4			S9+31 W4+87	Column C-8	13	121	7	"	"	100	"	"	"
Apr 14	5			S9+30 W4+64	Column C-9	13	121	8	"	"	101	"	"	"
"	6			S9+57 W5+50	Blow Down Header	19	120	8	"	"	100	"	"	"
Apr 19	7			S9+40 W5+10	Column C-7	15	119	6	"	"	100	"	"	"
"	8			S9+32 W4+90	Column C-8	15	120	6	"	"	100	"	"	"
"	9			S9+31 W4+59	Column C-9	15	121	6	"	"	100	"	"	"
"	10			S9+40 W4+38	Column C-10	13	121	5	"	"	101	"	"	"
Apr 20	11			S9+43 W4+32	Column C-10	15	122	8	"	"	102	"	"	"
Apr 22	12			S9+62 W5+33	Column C-6	13	118	9	"	"	98	"	"	"
Apr 26	13			S9+59 W5+30	Column C-6	15	114	5	"	"	95	"	"	"
Apr 29	14			S9+68 W4+17	Column C-11	13	118	7	"	"	98	"	"	"
"	15			S9+90 W4+06	Column C-12	13	121	7	"	"	101	"	"	"
Apr 30	16			S9+60 W4+15	Column C-11	15	118	7	"	"	98	"	"	"
"	17			S9+95 W4+05	Column C-12	15	122	8	"	"	102	"	"	"
May 10	18			S10+55 W5+18	Column C-2	9	118	10	"	"	98	"	"	"
May 11	19			S10+59 W5+20	Column C-2	11	121	9	"	"	100	"	"	"
May 12	20			S10+40 W5+27	Stack footing	9	121	8	"	"	101	"	"	"
"	21			S10+58 W5+30	" "	11	122	9	"	"	101	"	"	"

Remarks: SC = Sand Cone Density Test (ASTM D1556-64)

* = Test requested by Bechtel

SR = Safety Related

Class 1 & 2 Reviewed By:

TABLE 2[illegible] (continued)

Field Data Sheet

Sheet No: 2

Job Name: SONGS Unit 1 Sphere Enclosure Building

Job Number: B675F

1976 Date	Test Number	Retest by	Retest of	Grid Number	Location of Test	Elev.	Field Dry Density (pcf)	Moist. %	Method	Max. Lab. (pcf)	Rel. Comp. %	Spec. Req. %	Drawing No., Spec.	Quality Class
May 12	22			S10+40 W5+41	Stack footing	9	116	9	SC	120	96	95	[illegible]	SR
May 13	23			S10+38 W5+39	" "	11	121	9	"	"	101	"	"	"
"	24			S10+18 W4+05	Column C-13	12	118	8	"	"	98	"	"	"
May 14	25			S10+44 W5+30	Stack footing	10	123	7	"	"	102	"	"	"
Sep 24	41			S9+60 W5+55	Seal Water Filter	15	123	8	"	"	103	"	"	"
"	42			S9+61 W5+52	Enclosure Structure	17	122	7	"	"	102	"	"	"

Remarks: SC = Sand Cone Density Test (ASTM D1556-64)

* = Test requested by Bechtel

SR = Safety Related

Class 1 & 2 Reviewed By: [signature]

TABLE 2-4

SUMMARY OF REMEDIAL MEASURES
FOR SOIL BACKFILL CONDITIONS ENCOUNTERED IN RECENT EXCAVATIONS
(sheet 1 of 4)

Foundation	Soil Condition Encountered	Remedial Measures Implemented
A. Turbine Building		
1. North Turbine Footing (see Sections 1-1 and 2-2, Fig. 2-11)	Most of the footing is founded on native soil or 95 percent compacted backfill. During the excavation of the western portion of the footing 80 to 93 percent compacted backfill was encountered.	Backfill on western end of excavation was removed and replaced with concrete to el. +1 ft. Backfill below el. +1 ft was compacted in place by vibration.
2. Northwest Turbine Footings		
° Footing E-11 (see Section 4-4, Fig. 2-13)	Most of the footing is founded on native soil except for a small portion along the east wall. Backfill against the side of the footing is dense except for small portions of the east and south walls where it is about 85 percent relative compaction.	The stiffness parameters were modified.
° Footing C-9 (see Section 3-3, Fig. 2-12)	Most of the footing except a small width near the north side is founded on native soils. Backfill against the side of the footing is 85 to 87 percent relative compaction.	The stiffness parameters were modified.

TABLE 2-4 (CONTINUED)

SUMMARY OF REMEDIAL MEASURES
FOR SOIL BACKFILL CONDITIONS ENCOUNTERED IN RECENT EXCAVATIONS
(sheet 2 of 4)

Foundation	Soil Condition Encountered	Remedial Measures Implemented
3. West Turbine Building	The footing is founded on native soil or backfill with 90 to 95 percent relative compaction. Backfill against the side of the footing has 90 percent relative compaction.	The stiffness parameters were modified.
4. Southwest Turbine Footing	The northern and western portions of the footing are founded on backfill with relative compaction of 95 to 100 percent. The remaining footing had backfill with 83 to 85 percent relative compaction at elevation +7 ft. Backfill against the side of the footing varied between 90 and 95 percent relative compaction.	The excavation was deepened to about elevation +3 ft., and the overexcavated area was filled with concrete.
5. Outrigger Turbine Footing	Tests in the excavation showed back-fill at a relative compaction of 87 to 93 percent to elevation +3 ft.	The footing was modified to be supported by the intake culverts, and the overexcavation below the footing base at elevation +5 ft. was backfilled with concrete.
6. Northeast Footing		
° Footing E-3	Most of the footing is founded on native soil. During the excavation of the western end of the footing 80 to 85 percent compacted backfill was encountered.	Backfill on the western end of the excavation was removed and replaced with concrete.

TABLE 2-4 (CONTINUED)

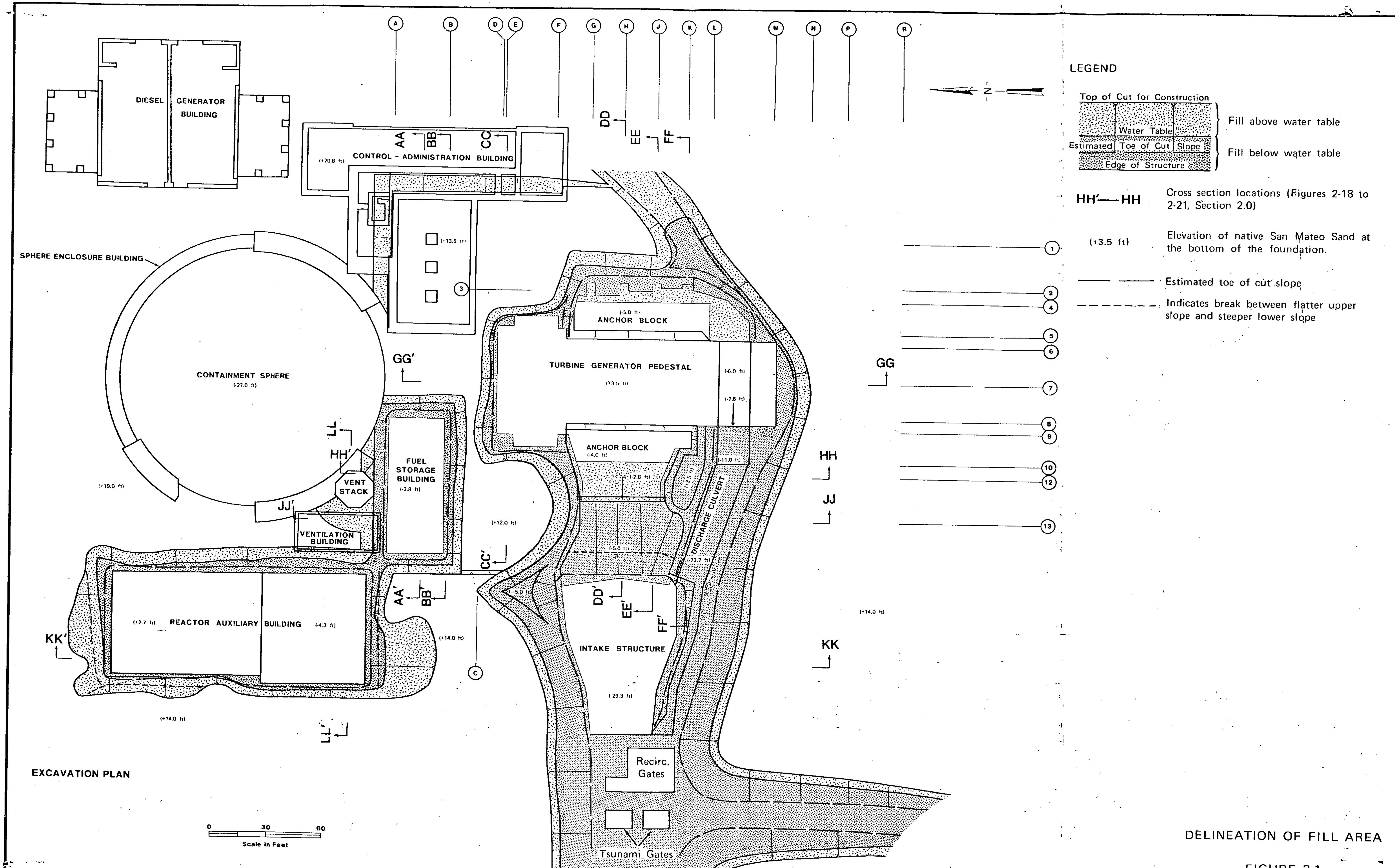
SUMMARY OF REMEDIAL MEASURES
FOR SOIL BACKFILL CONDITIONS ENCOUNTERED IN RECENT EXCAVATIONS
(sheet 3 of 4)

Foundation	Soil Condition Encountered	Remedial Measures Implemented
7. East Footings		
° Footing A (see Sections 5-5 and 6-6, Fig. 2-14)	The footing is founded on native soil.	
° Footing B (see Sections 7-7 and 8-8, Fig. 2-15)	Most of the footing is founded on native soil except in a 10 ft wide area at the west end. In that area the backfill has a relative compaction of about 89 percent or lower, down to elevation about -5.0 feet.	The excavation at the west end was deepened to elevation -1.0 ft, and the overexcavated area was filled with concrete.
8. Southeast Footing		
° Footing C (see Section 9-9, Fig. 2-16)	Approximately the northern two thirds of the footing is founded on the existing anchor block at elevation +8.5 ft. In the south end, the footing is founded on backfill with a relative compaction of about 88 percent or lower down to elevation +5.0 ft.	Northern two-thirds of the footing is founded on the existing anchor block at elevation +8.5 feet. The remaining southern one-third of the footing is founded on backfill with a relative compaction of 88 percent or lower. This condition was considered in the reevaluation analysis.
° Footing E (see Section 10-10, Fig. 2-17)	Approximately the southern half of the footing is founded on native soil at elevation +14.5 ft. In the northern portion, the backfill has a relative compaction of 80 to 91 percent down to elevation +12 ft.	1. The excavation at the north end was deepened to elevation +8.5 ft at the top of turbine mat and the overexcavated area was filled with concrete. 2. The stiffness parameters were modified.

TABLE 2-4 (CONTINUED)

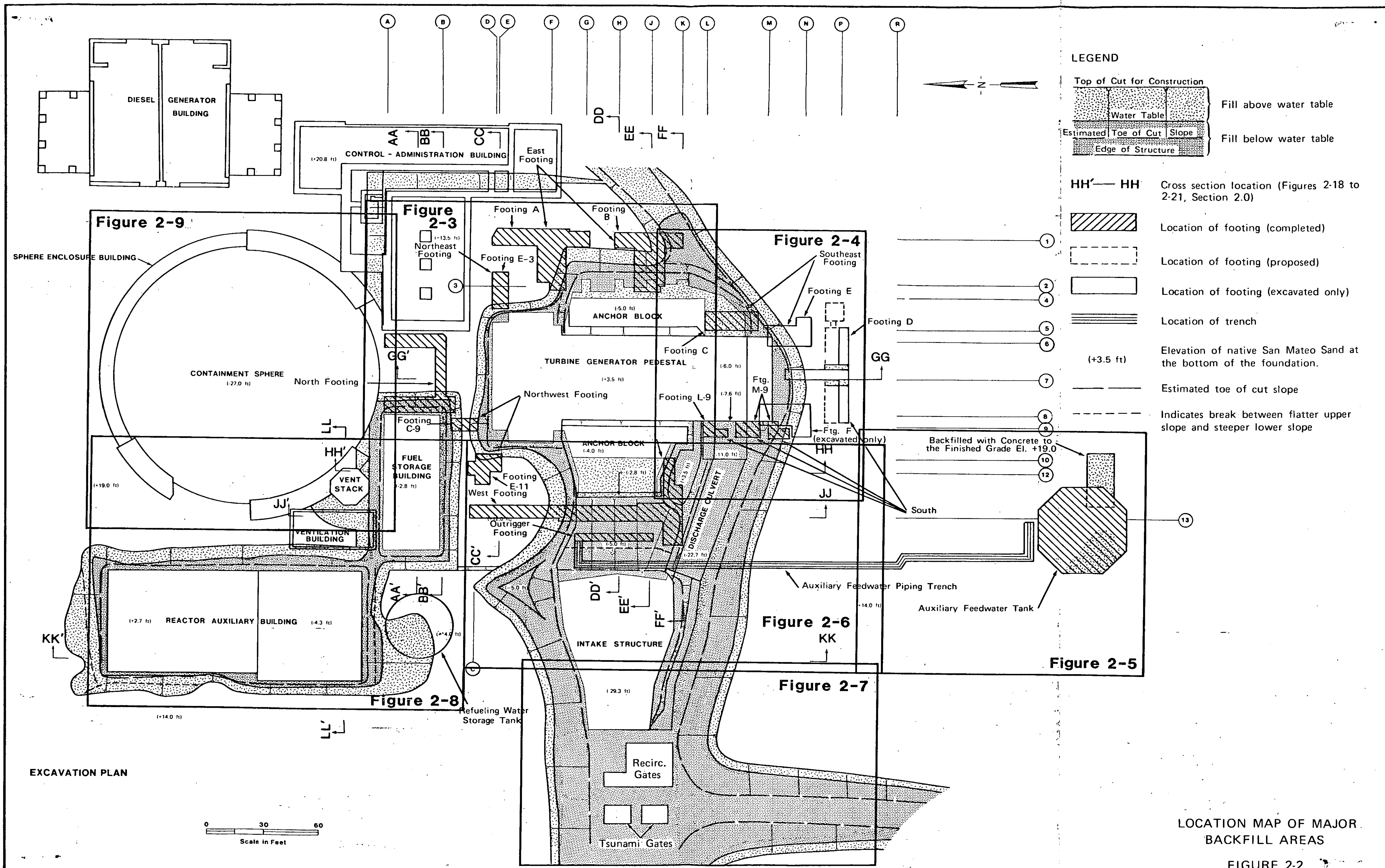
SUMMARY OF REMEDIAL MEASURES
FOR SOIL BACKFILL CONDITIONS ENCOUNTERED IN RECENT EXCAVATIONS
(sheet 4 of 4)

Foundation	Soil Condition Encountered	Remedial Measures Implemented
B. Sphere Enclosure Building	During the construction of the sphere enclosure building, the footing excavation at the southwest end of the building indicated some backfill at a relative compaction of less that 95 percent.	The excavation for the sphere enclosure building's foundation at this location was extended to elevation +7 ft. and replaced with fill compacted to 95 percent relative compaction.
C. Foundation of New Auxiliary Feedwater Tank	The foundation is founded on native soil except for a small portion at the west end. In that area the backfill has a relative compaction of about 97 percent or higher. Also at the east end of the excavation, an existing septic tank was removed that extended below the base of the foundation.	The area of the septic tank at the east end of the excavation was overexcavated to elevation +7.0 ft and backfilled with concrete.
D. Auxiliary Feedwater Piping Trench	Most of the trench is founded on native soil except for the portion at the north end in the intake culverts area. In that area the backfill has a relative compaction of about 88 percent or lower.	The excavation in the intake culverts area was made to elevation +4. A concrete u-shaped trench was constructed to transfer the load of the piping trench to the intake culvert thereby eliminating the need for the backfill to support the trench.
E. Refueling Water Storage Tank	Approximately 60 percent of the footing is founded on native soil with the remaining 40 percent on recompacted soil in the northwest section as shown in Figure 2-2. In this area the backfill has a relative compaction of about 92 percent or higher.	The stiffness parameters were modified.



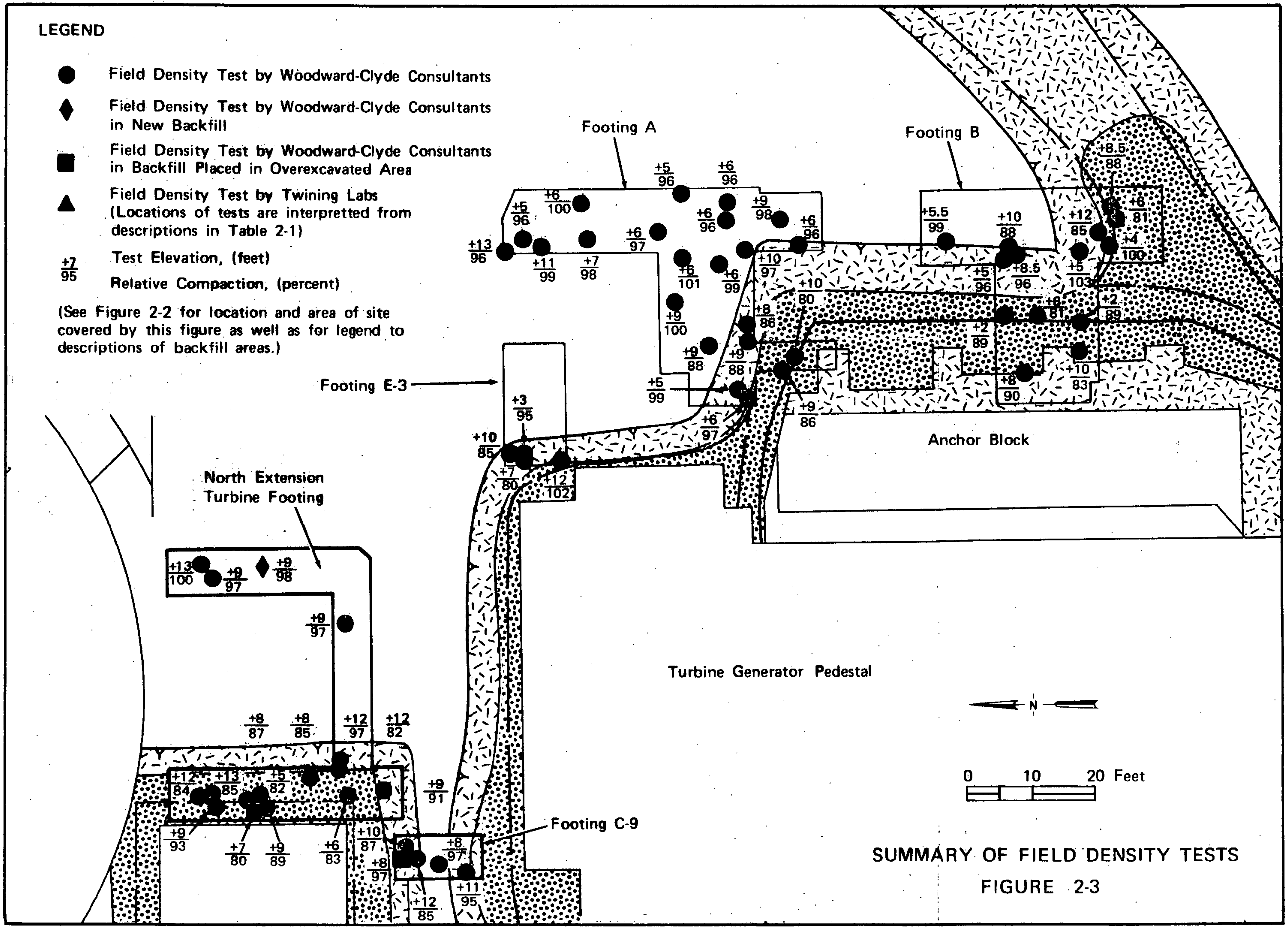
DELINEATION OF FILL AREA

FIGURE 2-1



LOCATION MAP OF MAJOR BACKFILL AREAS

FIGURE 2-2



SUMMARY OF FIELD DENSITY TESTS

FIGURE 2-3

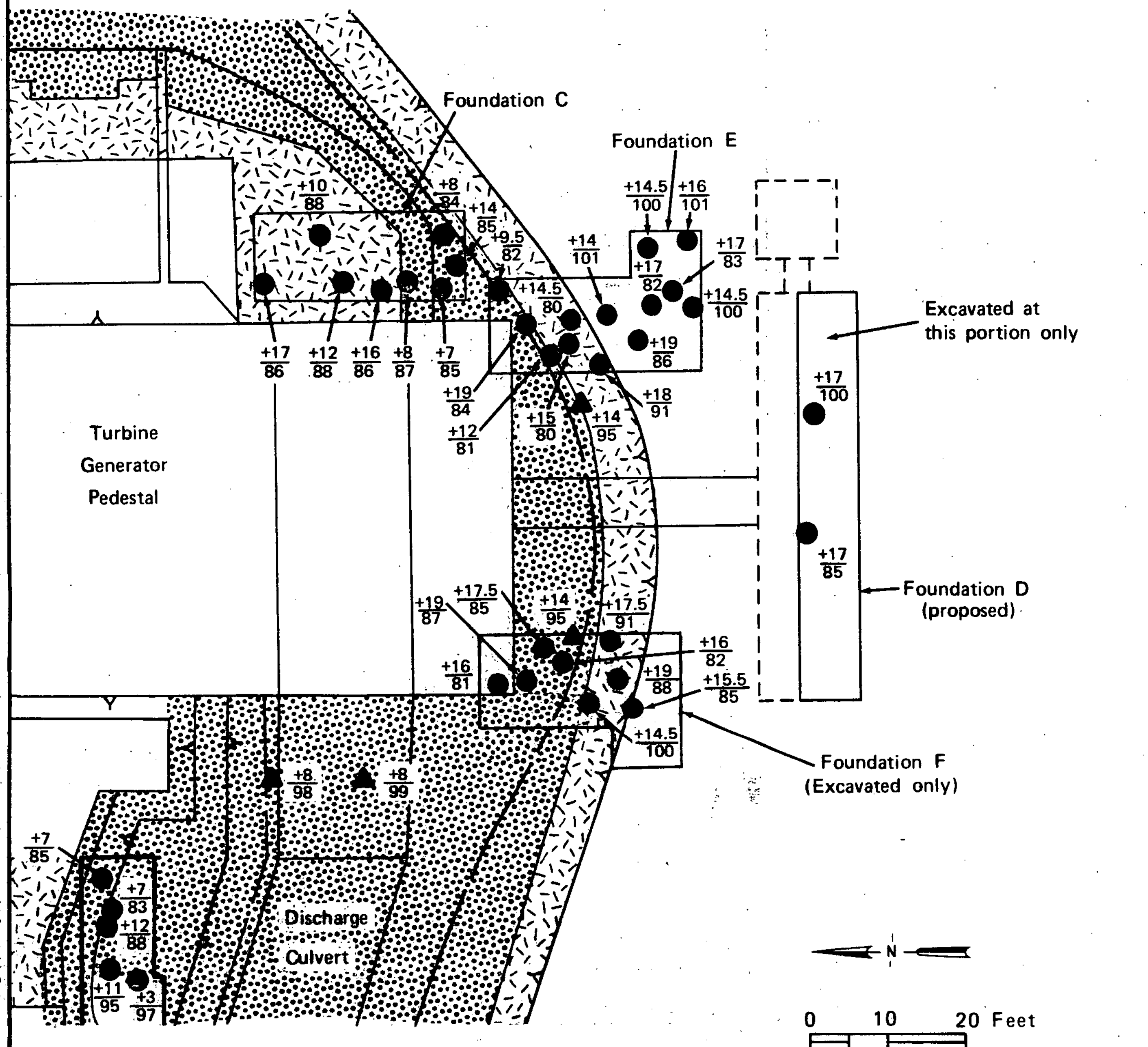
LEGEND

● Field Density Test by Woodward-Clyde Consultants

▲ Field Density Test by Twining Labs (Locations of tests are interpretted from descriptions in Table 2-1)

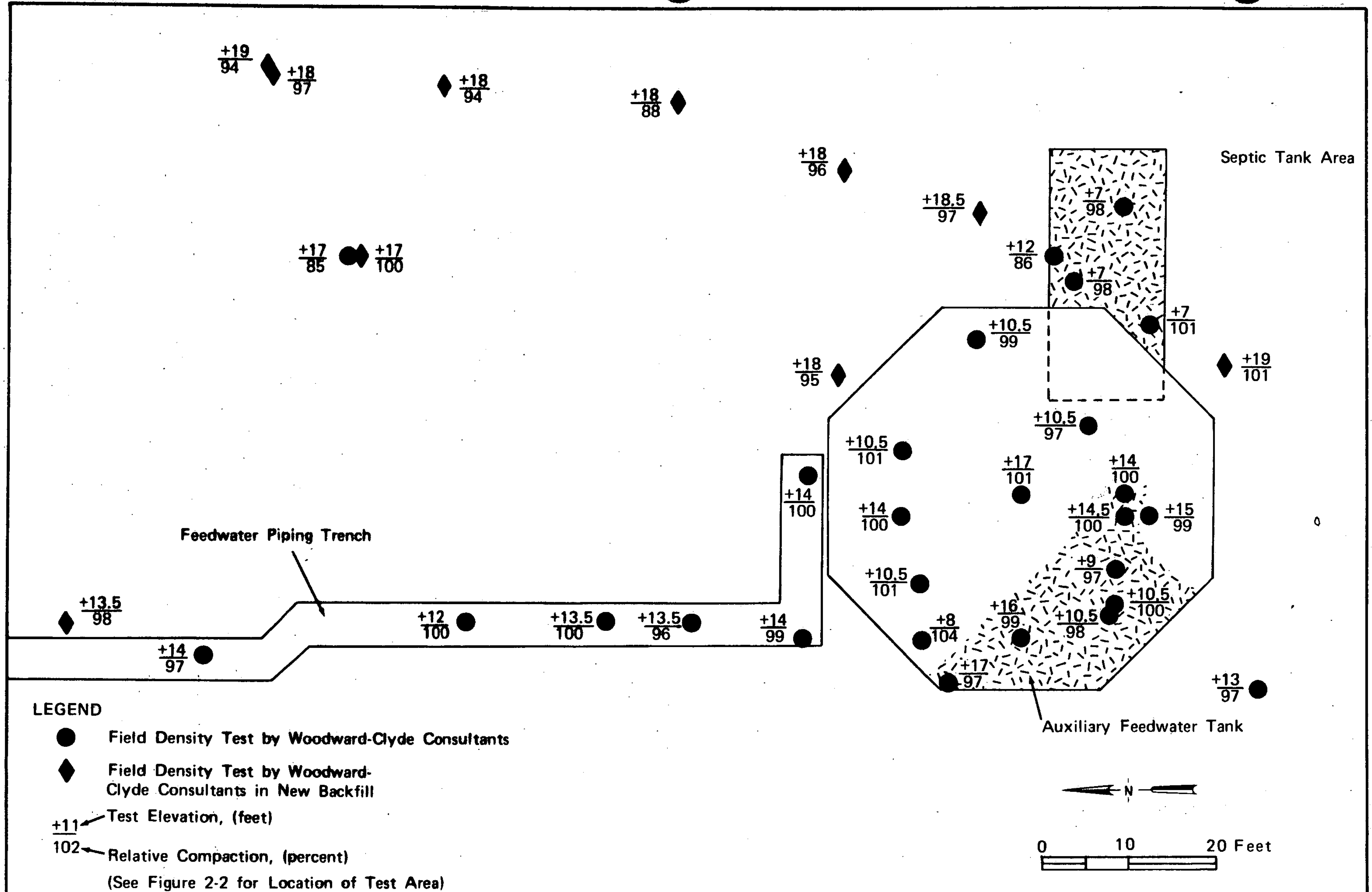
+7 — Test Elevation, (feet)
95 — Relative Compaction, (percent)

(See Figure 2-2 for Location of Test Area)



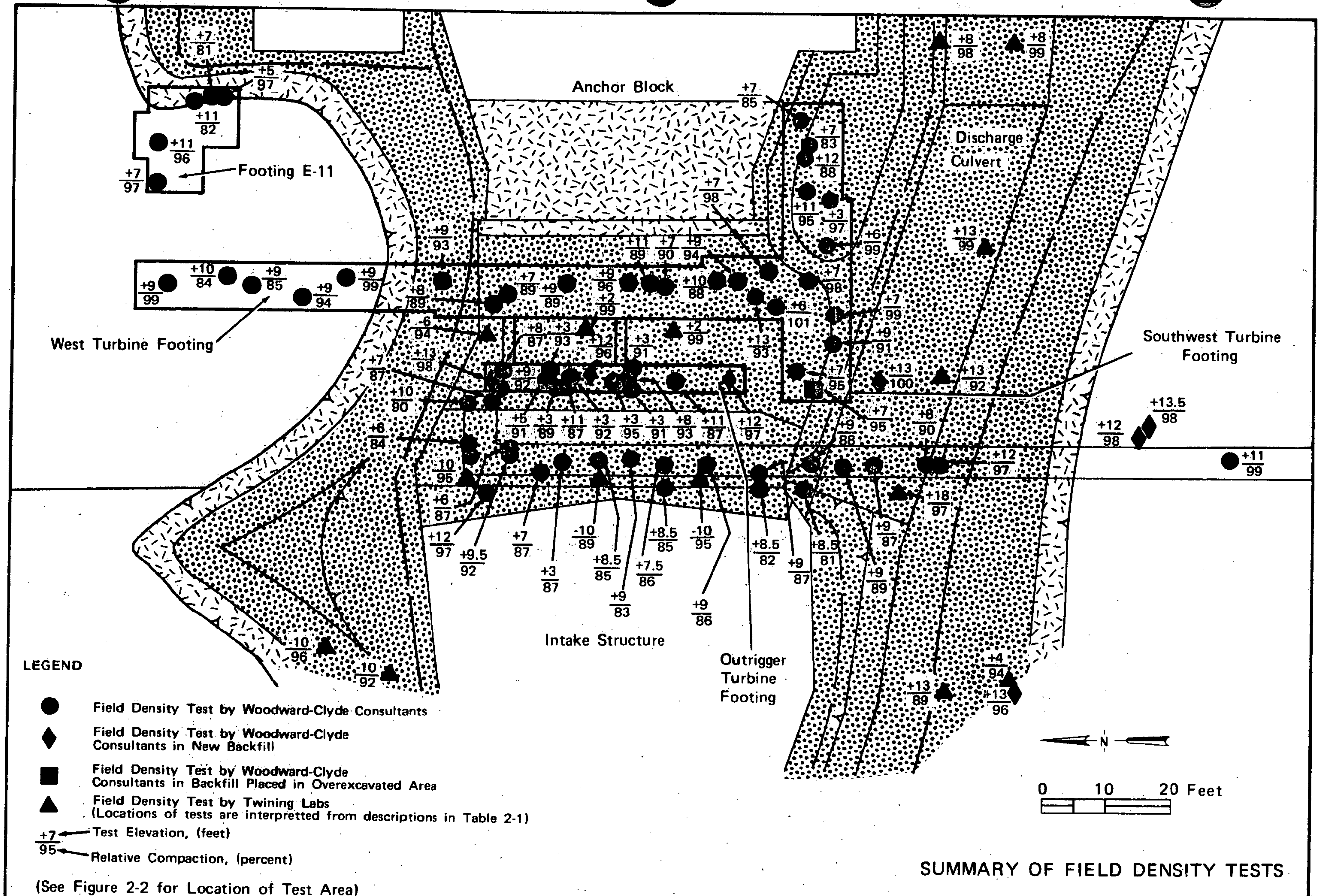
SUMMARY OF FIELD DENSITY TESTS

FIGURE 2-4



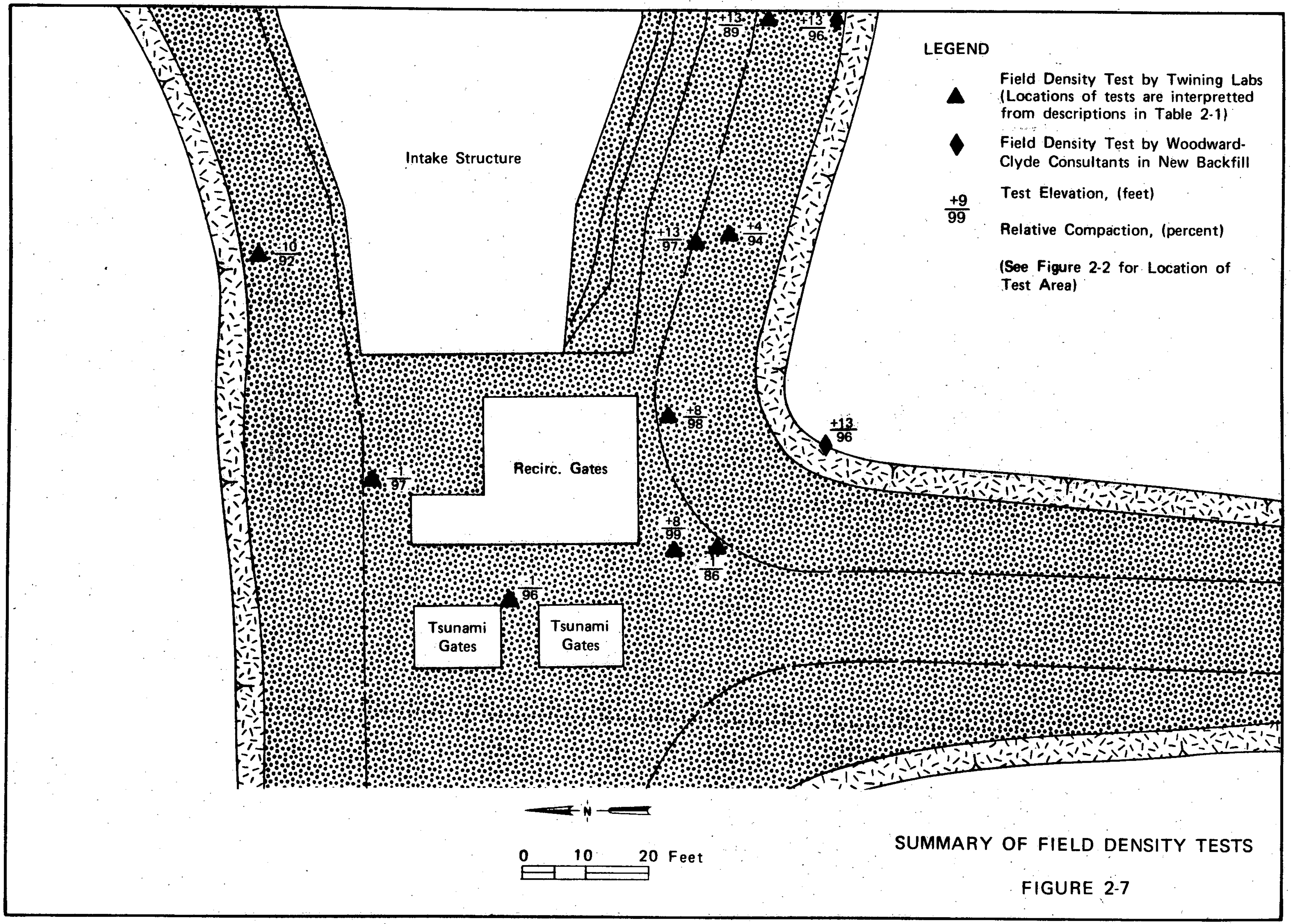
SUMMARY OF FIELD DENSITY TESTS

FIGURE 2-5



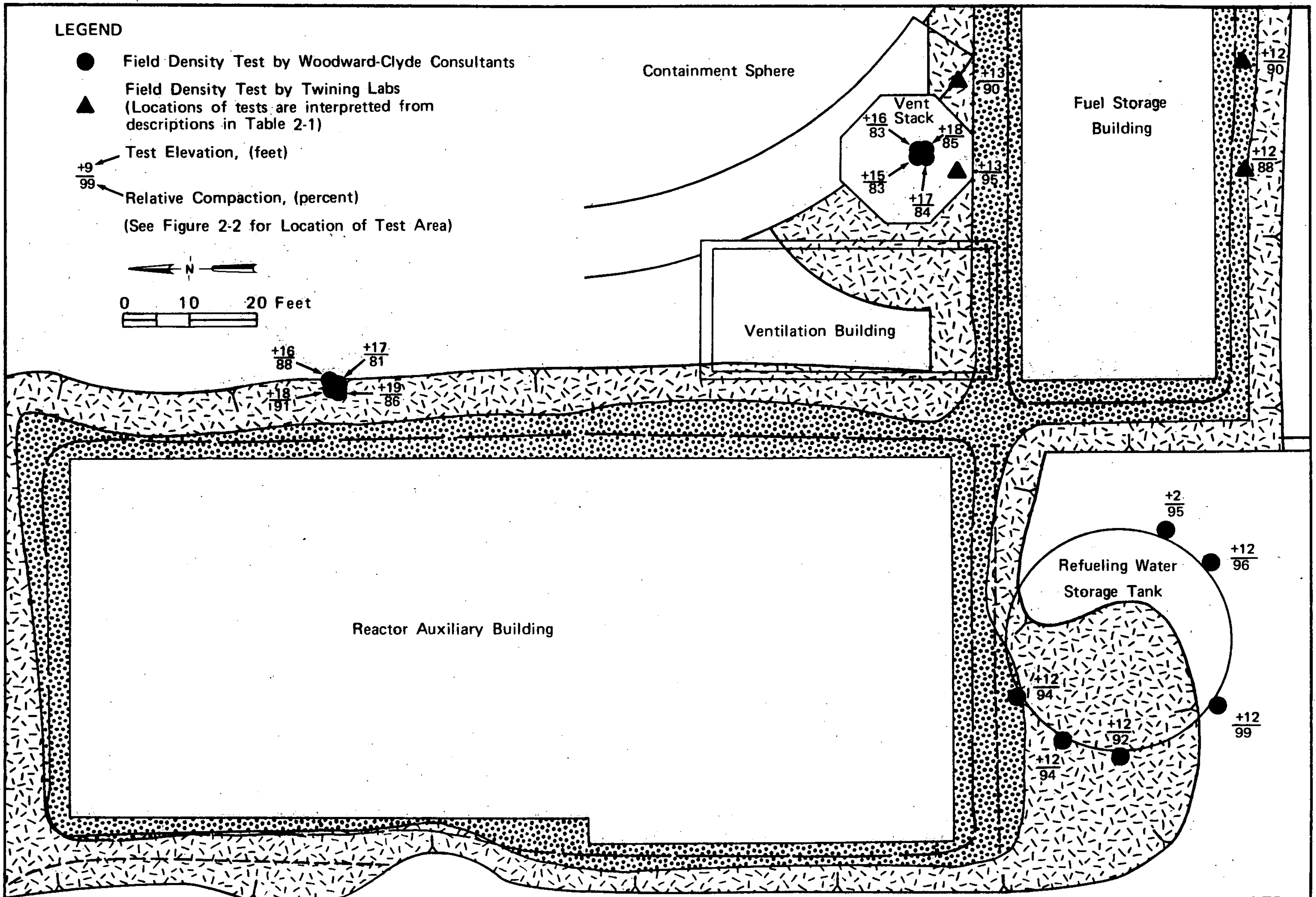
SUMMARY OF FIELD DENSITY TESTS

FIGURE 2-6



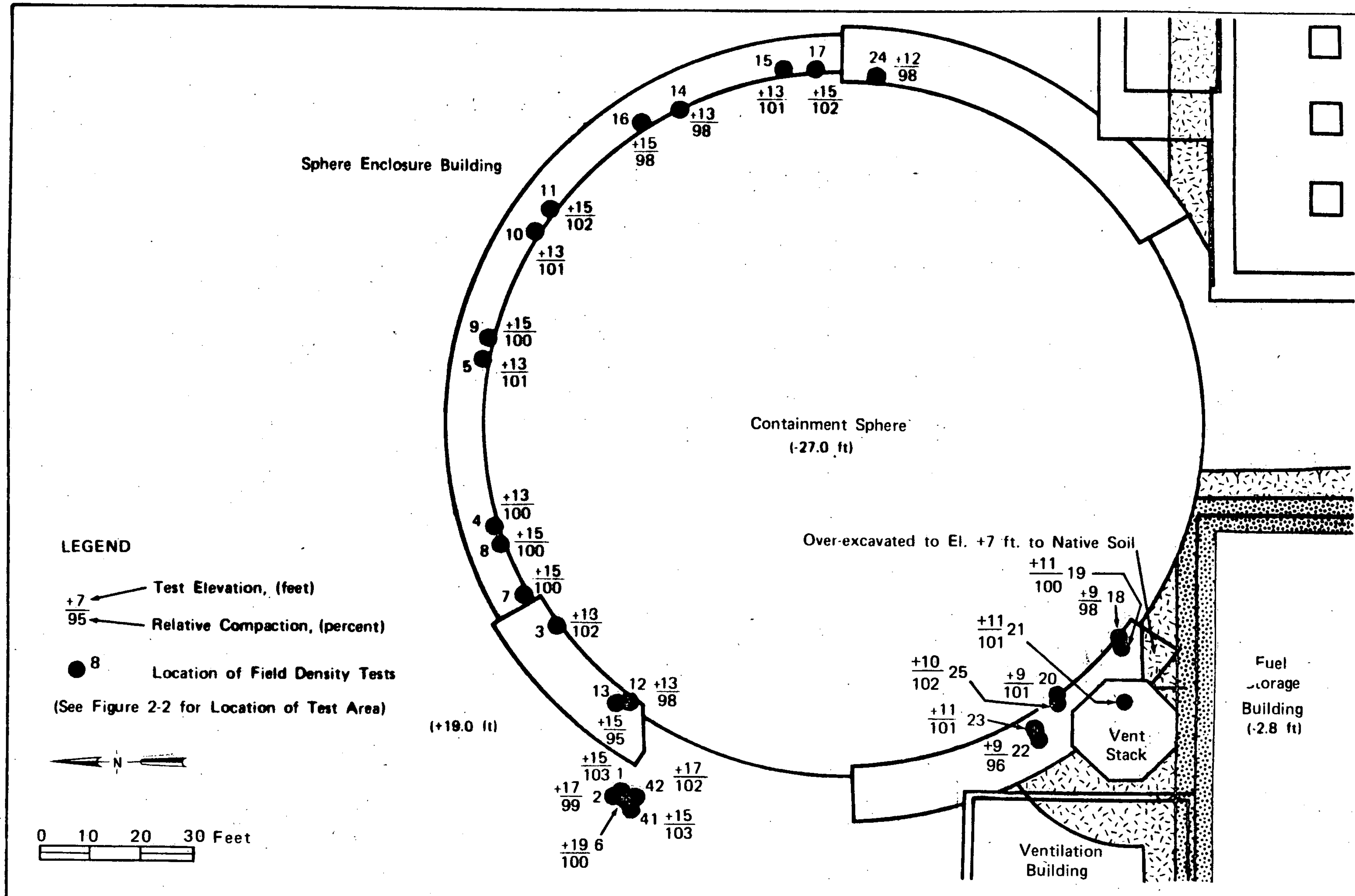
SUMMARY OF FIELD DENSITY TESTS

FIGURE 2-7

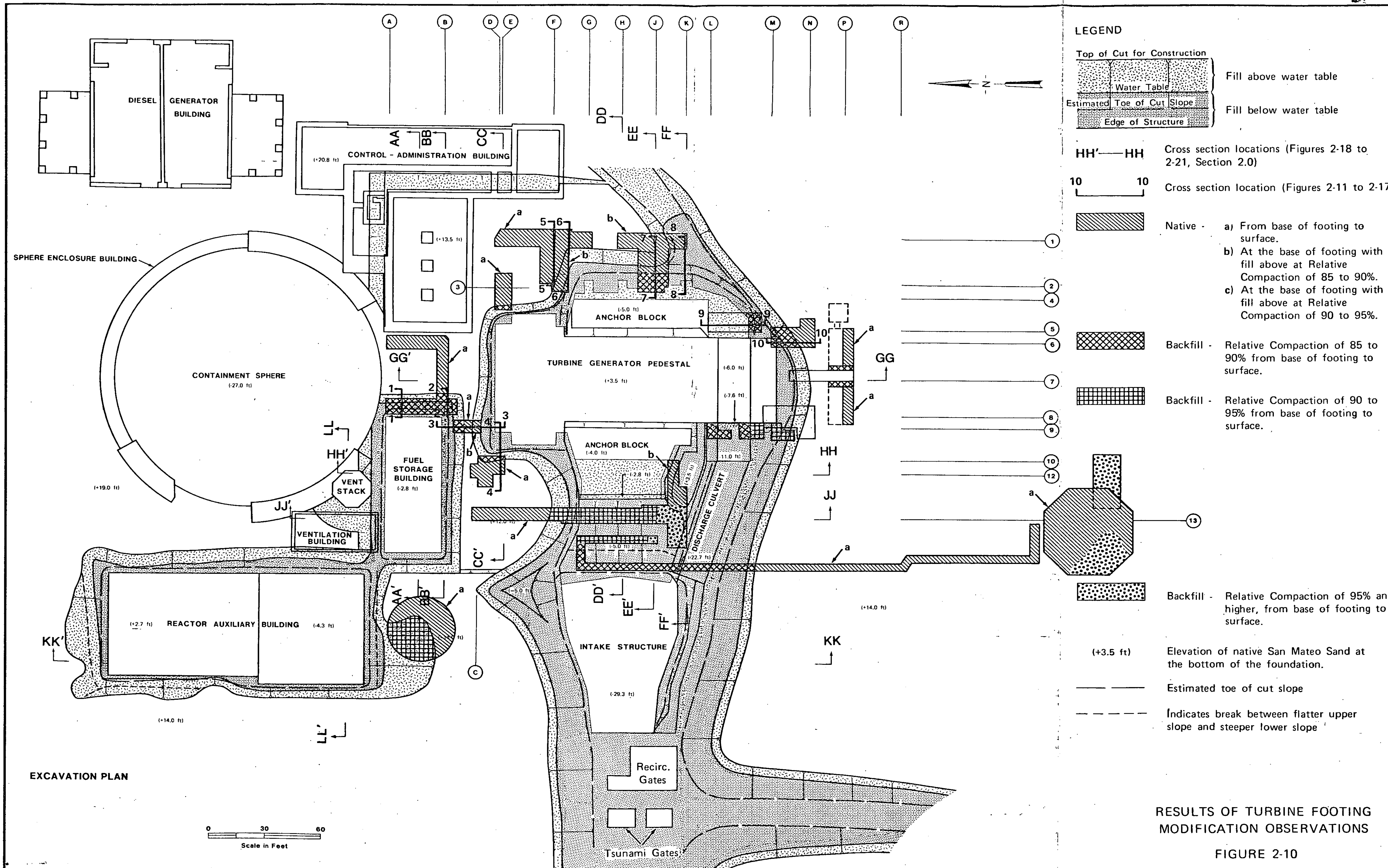


SUMMARY OF FIELD DENSITY TESTS

FIGURE 2-8

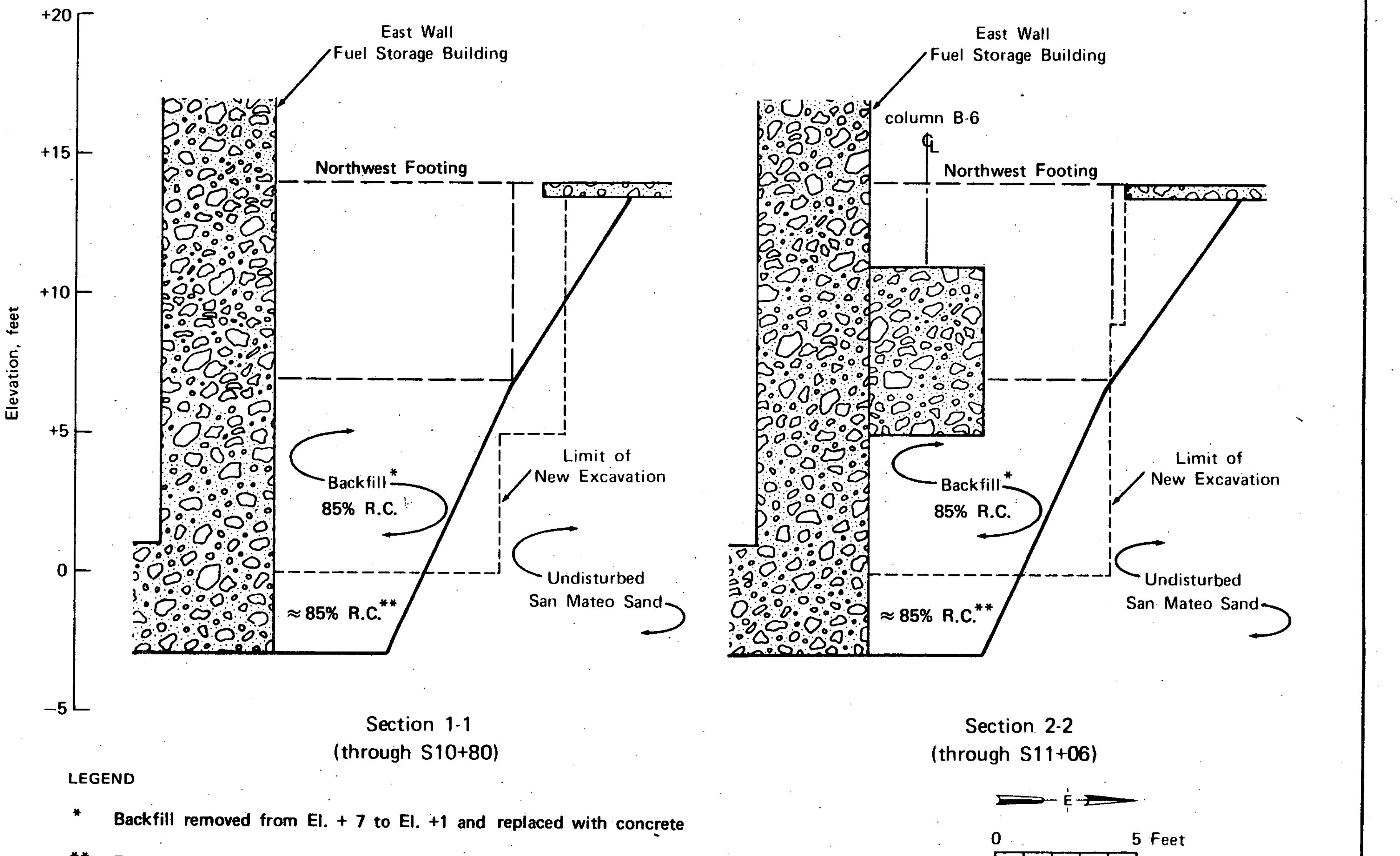


SUMMARY OF FIELD DENSITY TESTS
FIGURE 2-9



RESULTS OF TURBINE FOOTING MODIFICATION OBSERVATIONS

FIGURE 2-10



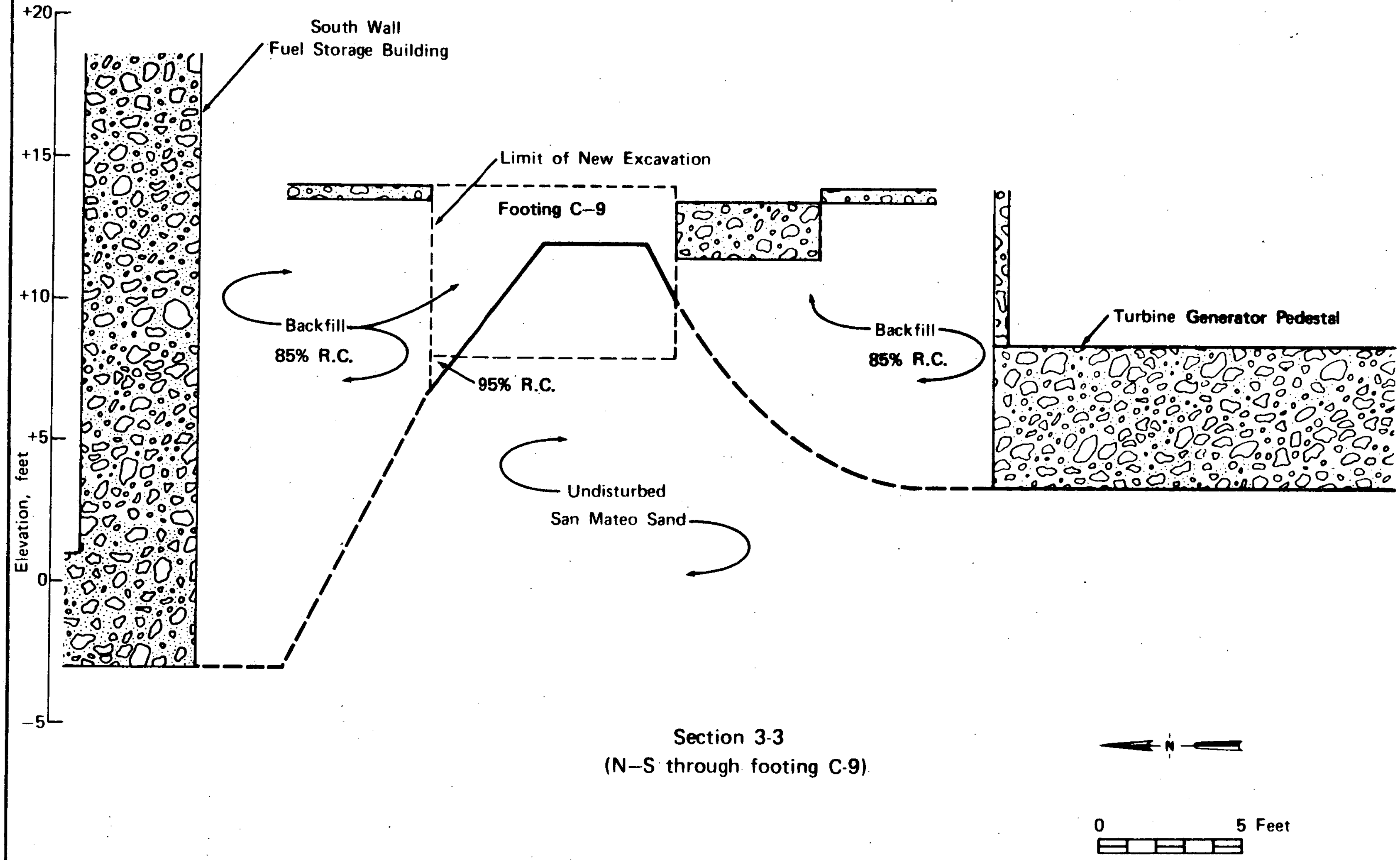
LEGEND

* Backfill removed from El. + 7 to El. +1 and replaced with concrete

** Relative compaction estimated from probing in-situ

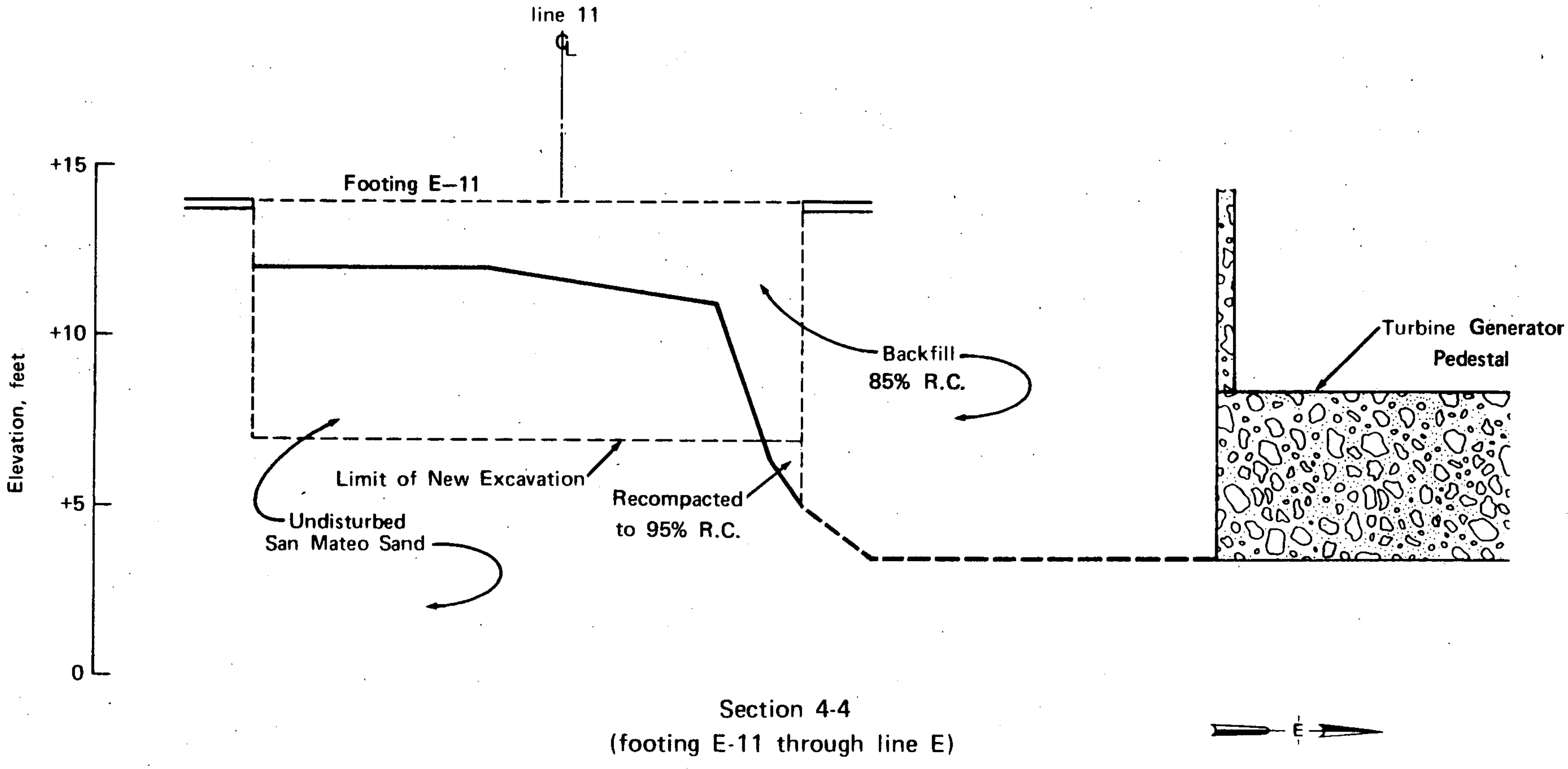
See Figure 2-10 for location of the sections on this figure.

LOCAL SOIL CONDITIONS — NORTHWEST FOOTING
FIGURE 2–11



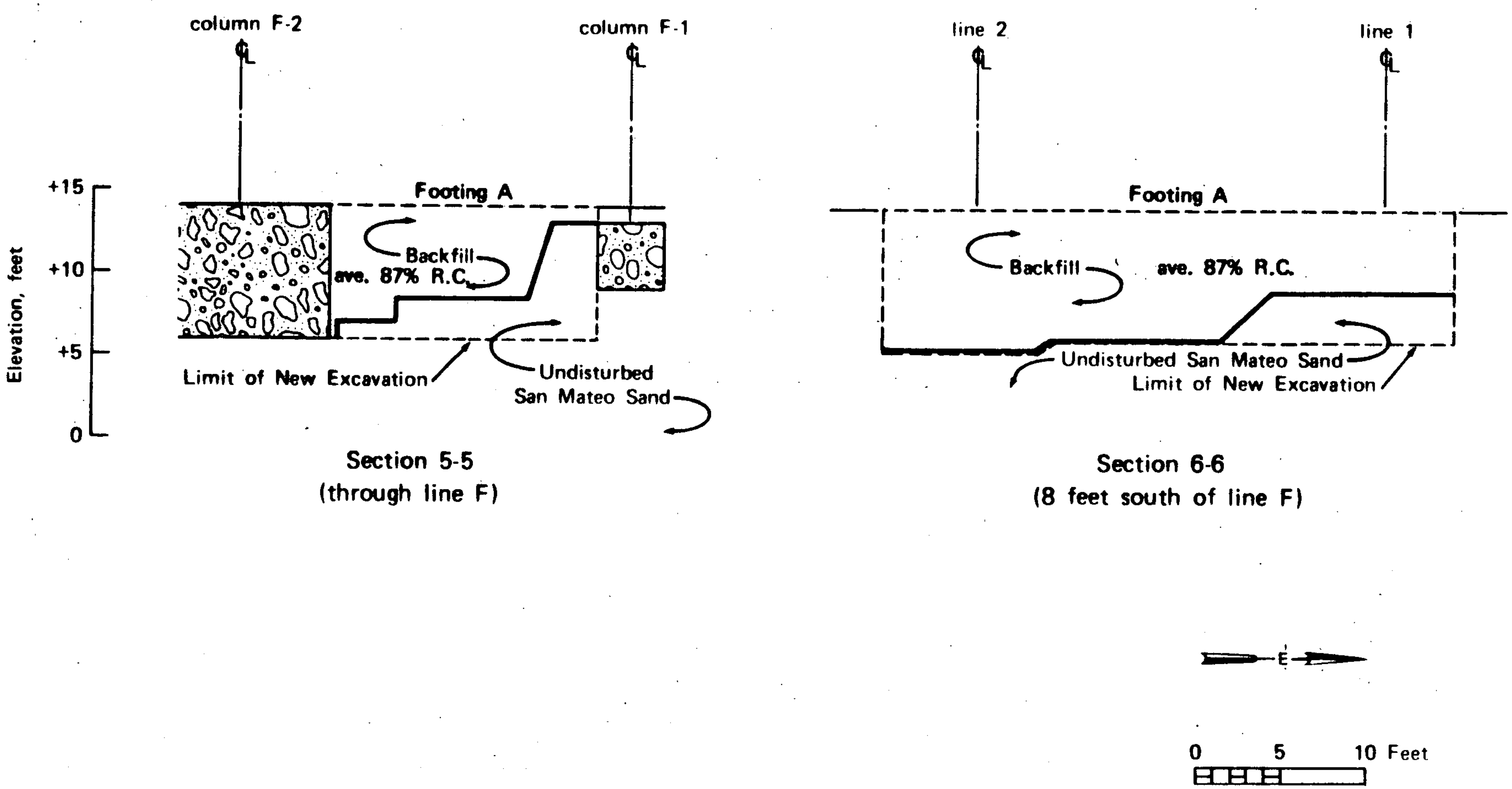
LOCAL SOIL CONDITIONS –
FOOTING C–9

FIGURE 2–12



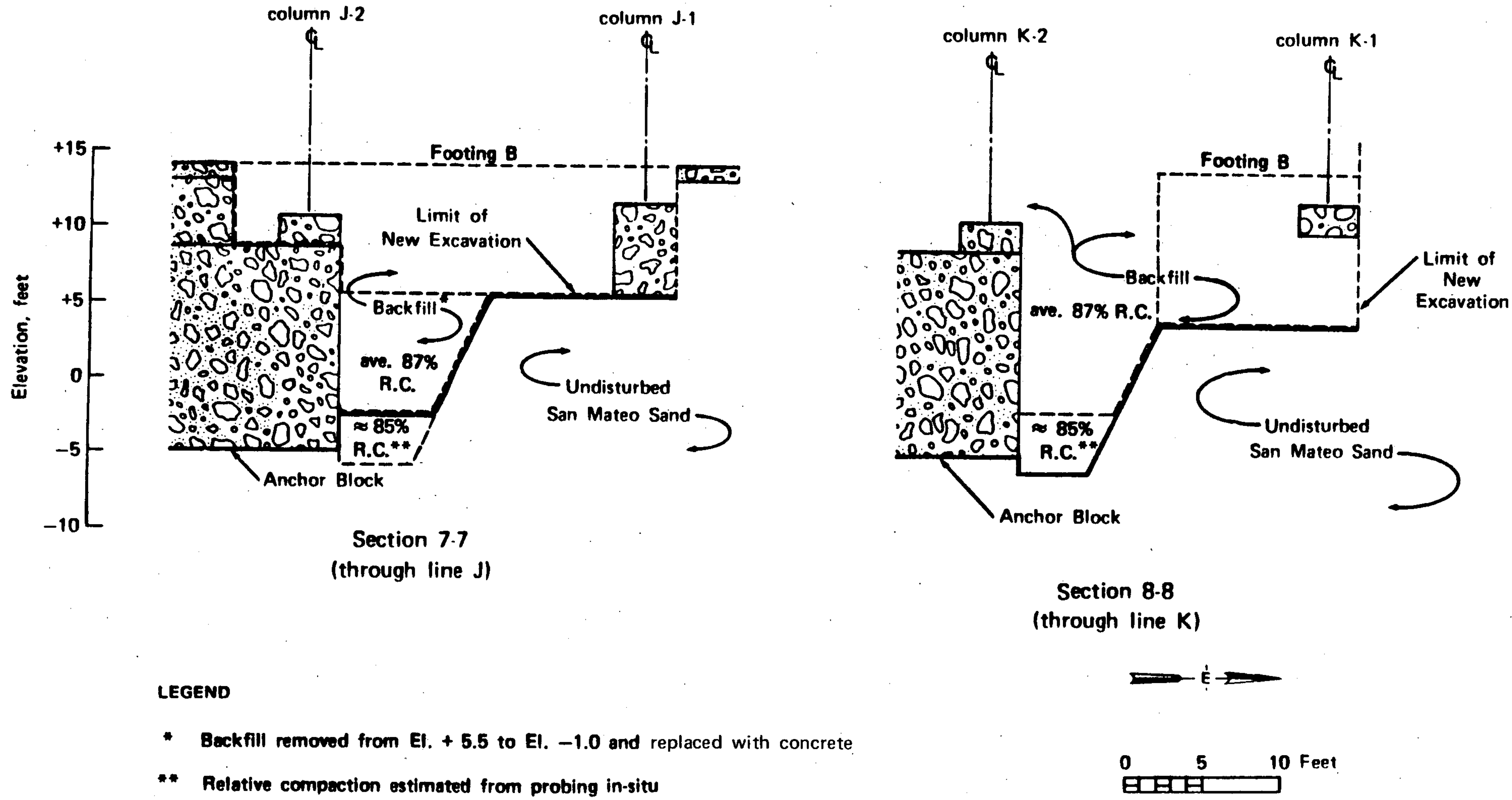
LOCAL SOIL CONDITIONS –
FOOTING E–11

FIGURE 2–13



LOCAL SOIL CONDITIONS – FOOTING A

FIGURE 2–14



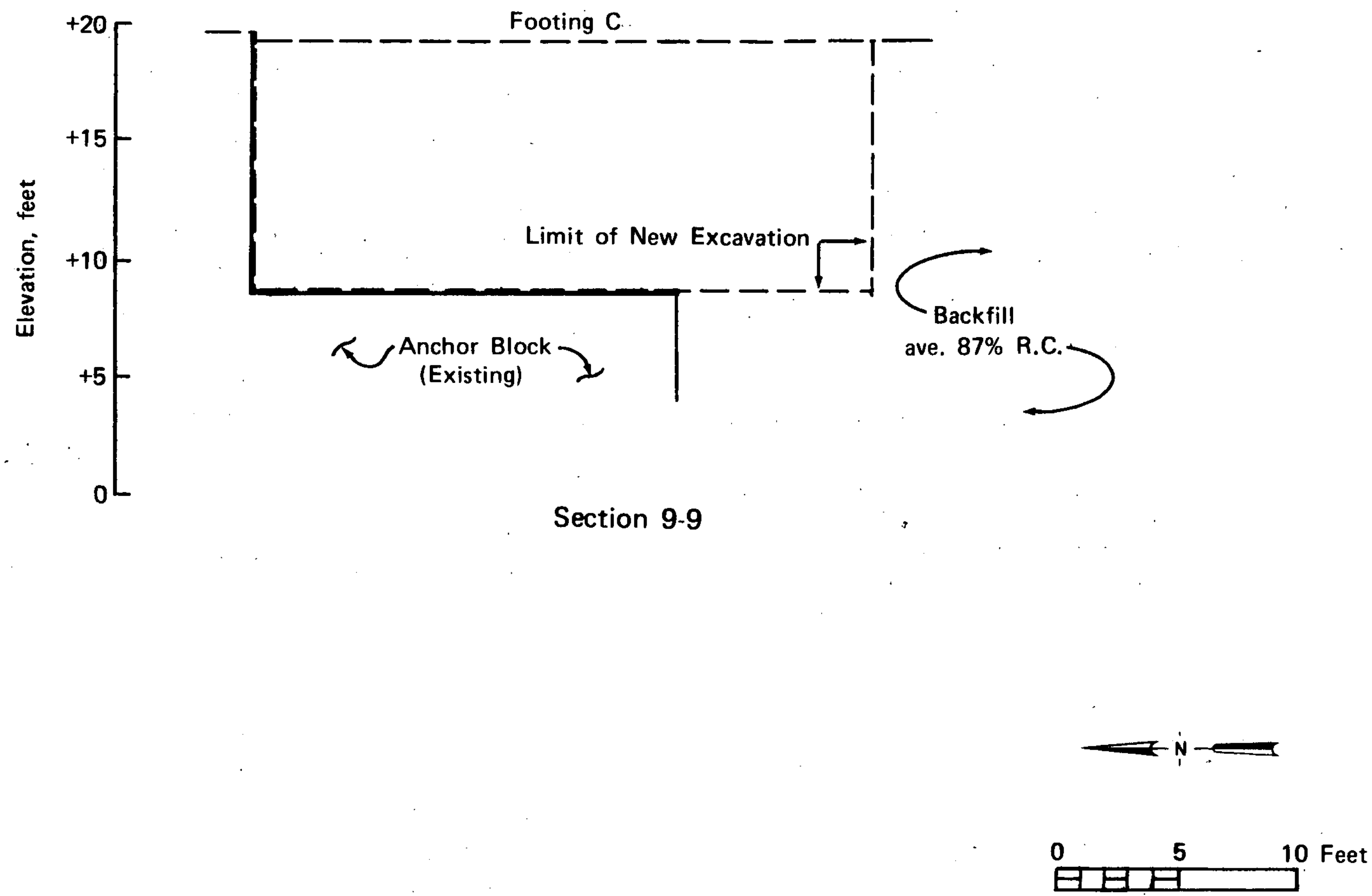
LEGEND

* Backfill removed from El. + 5.5 to El. −1.0 and replaced with concrete

** Relative compaction estimated from probing in-situ

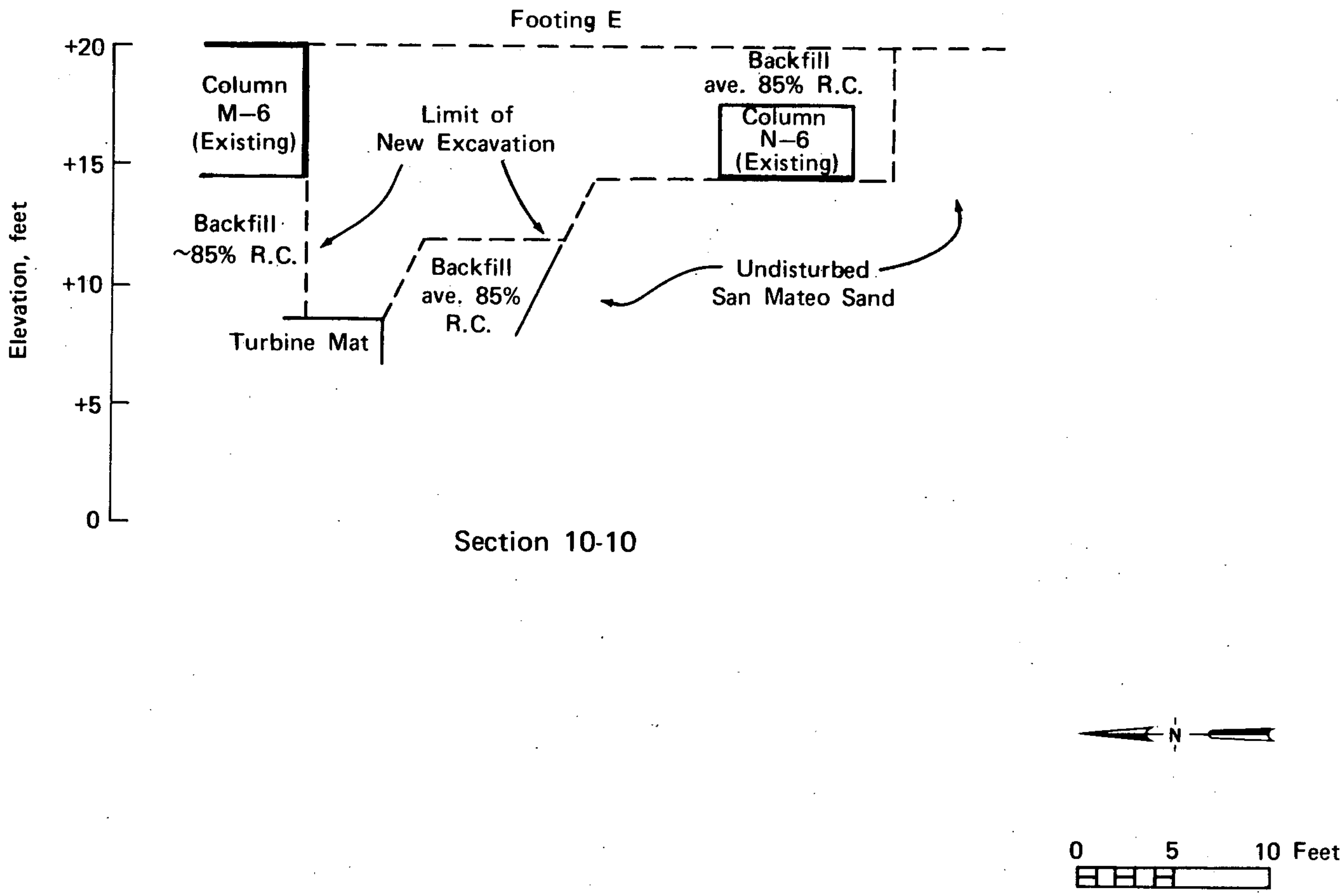
LOCAL SOIL CONDITIONS – FOOTING B

FIGURE 2–15

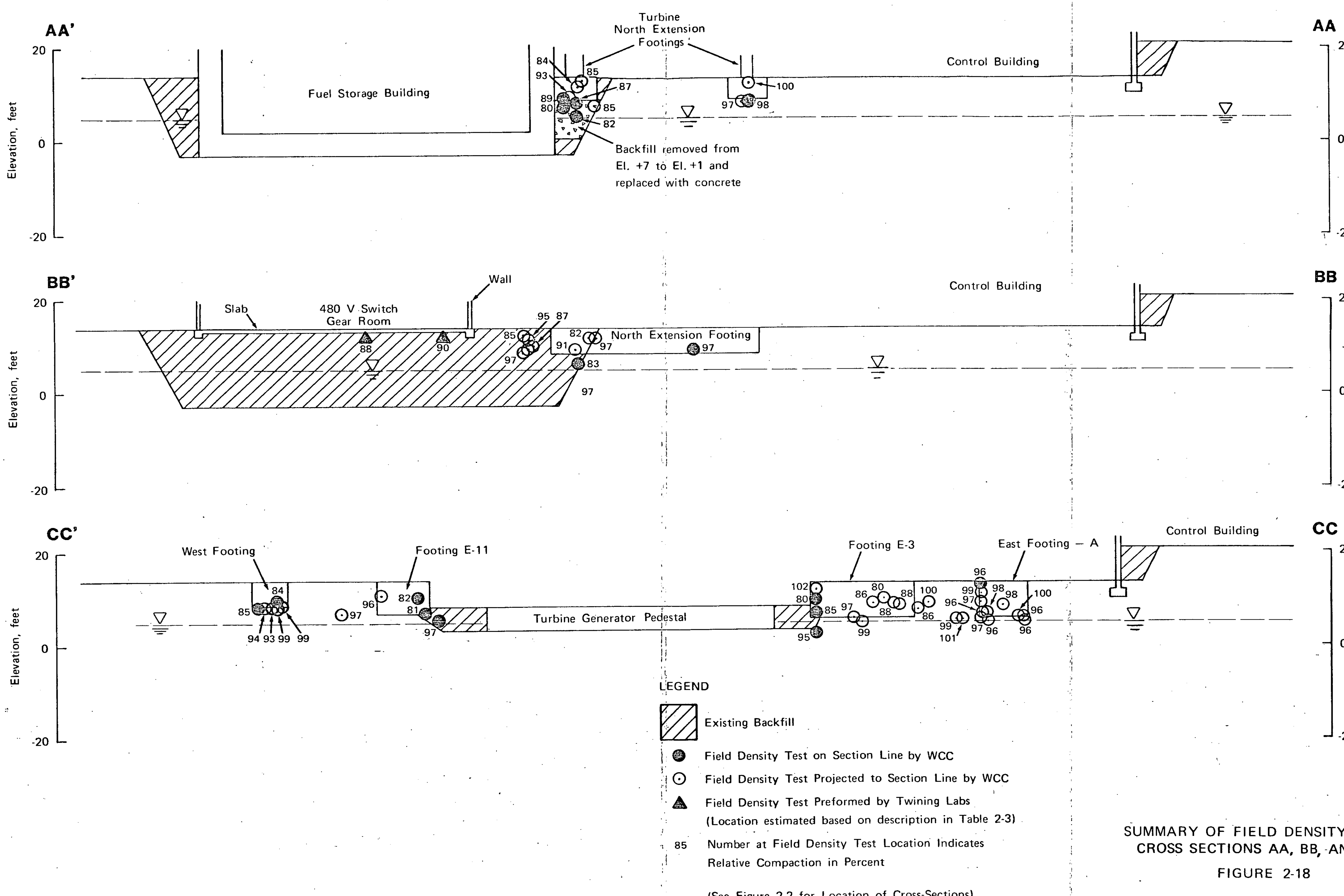


LOCAL SOIL CONDITIONS –
FOOTING C

FIGURE 2–16

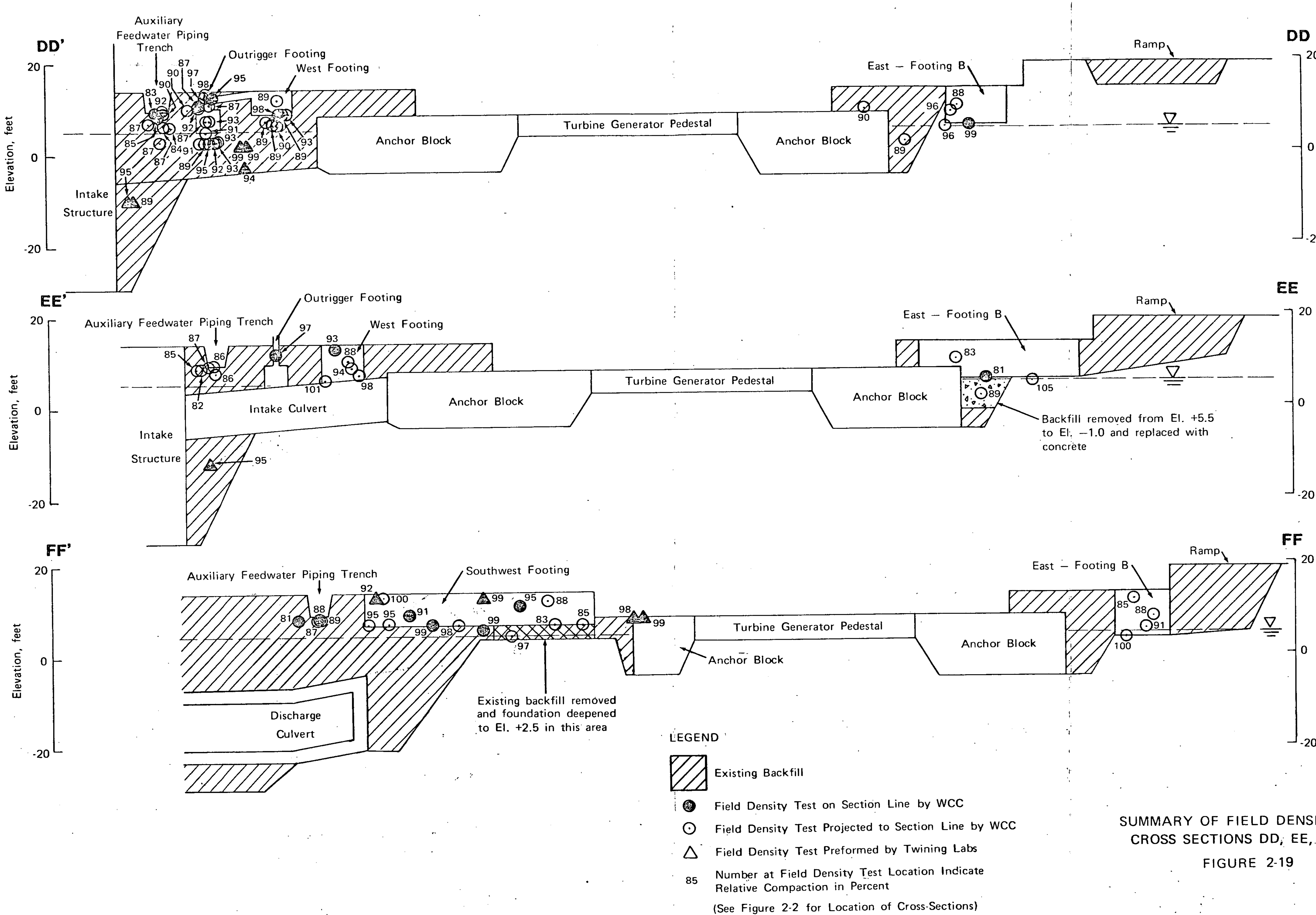


LOCAL SOIL CONDITIONS –
FOOTING E
FIGURE 2–17



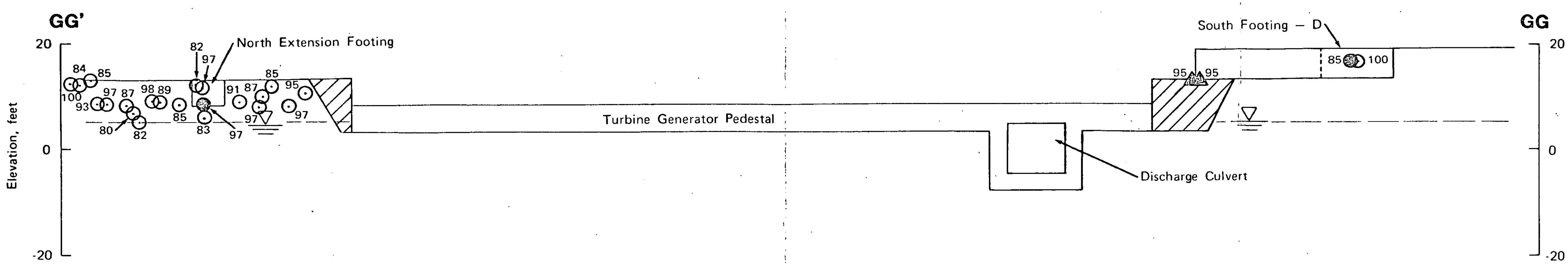
SUMMARY OF FIELD DENSITY TESTS
CROSS SECTIONS AA, BB, AND CC

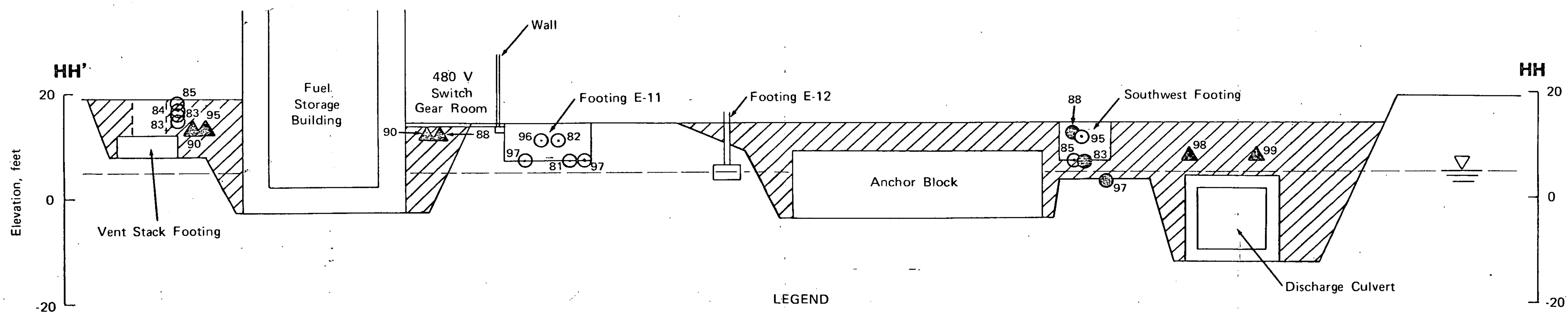
FIGURE 2-18



SUMMARY OF FIELD DENSITY TESTS
CROSS SECTIONS DD, EE, AND FF

FIGURE 2-19





LEGEND

Existing Backfill

Field Density Test on Section Line by WCC

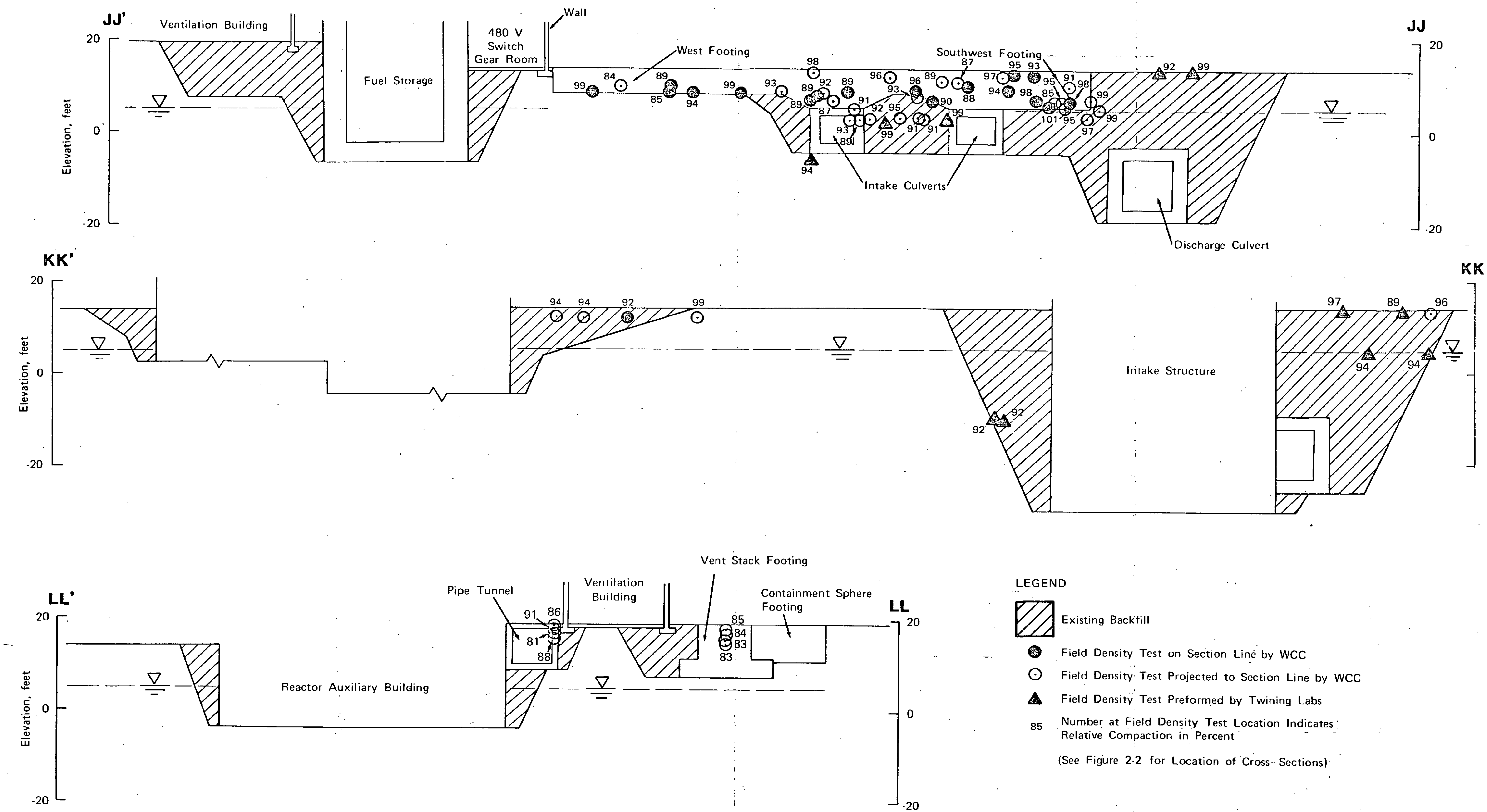
Field Density Test Projected to Section Line by WCC

Field Density Test Preformed by Twining Labs

85 Number at Field Density Test Location Indicates Relative Compaction in Percent

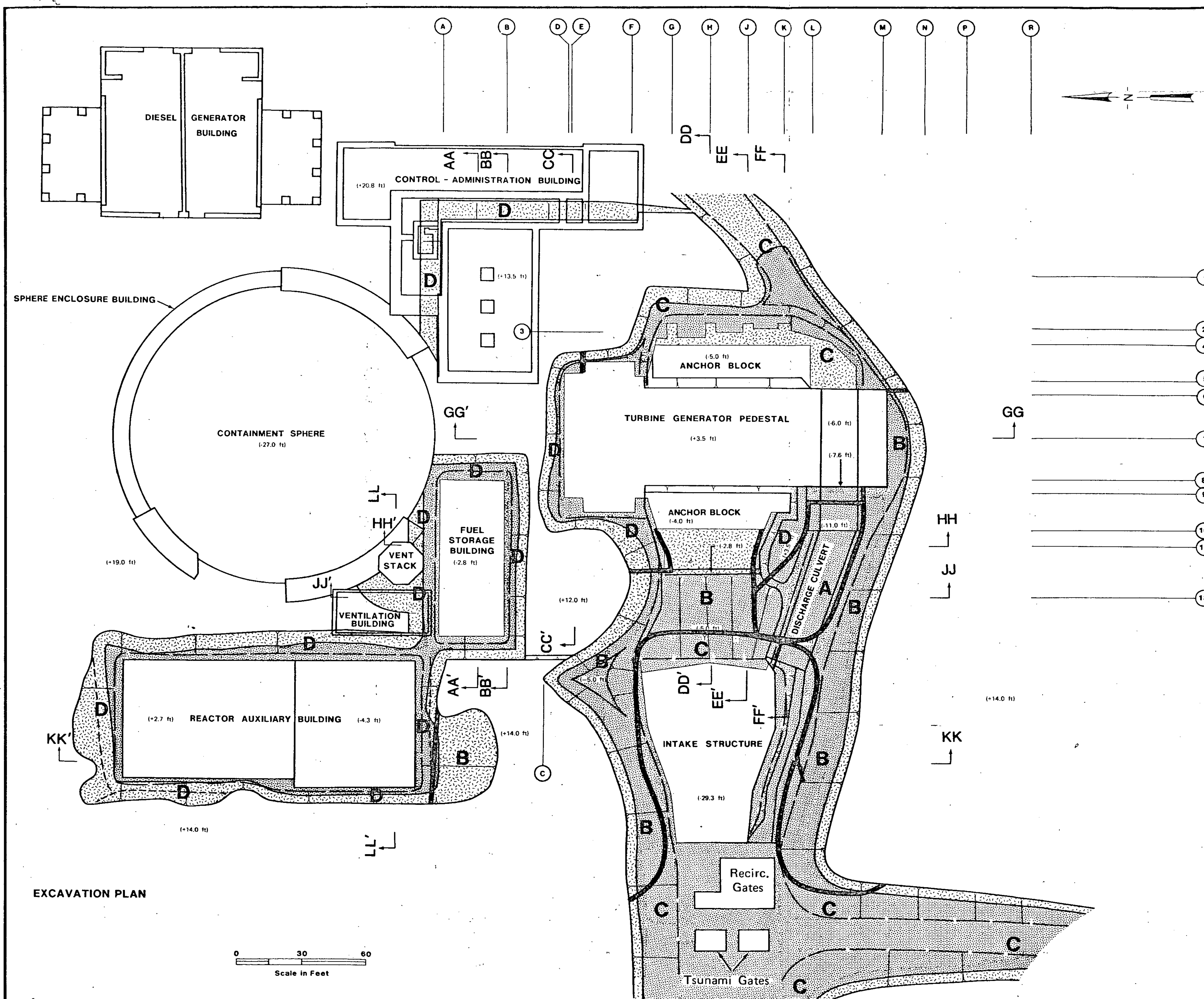
(See Figure 2-2 for Location of Cross-Sections)

SUMMARY OF FIELD DENSITY TESTS
CROSS SECTIONS GG AND HH
FIGURE 2-20



SUMMARY OF FIELD DENSITY TESTS
CROSS SECTIONS JJ, KK, AND LL

FIGURE 2-21



LEGEND

Top of Cut for Construction
Water Table
Estimated Toe of Cut Slope
Edge of Structure

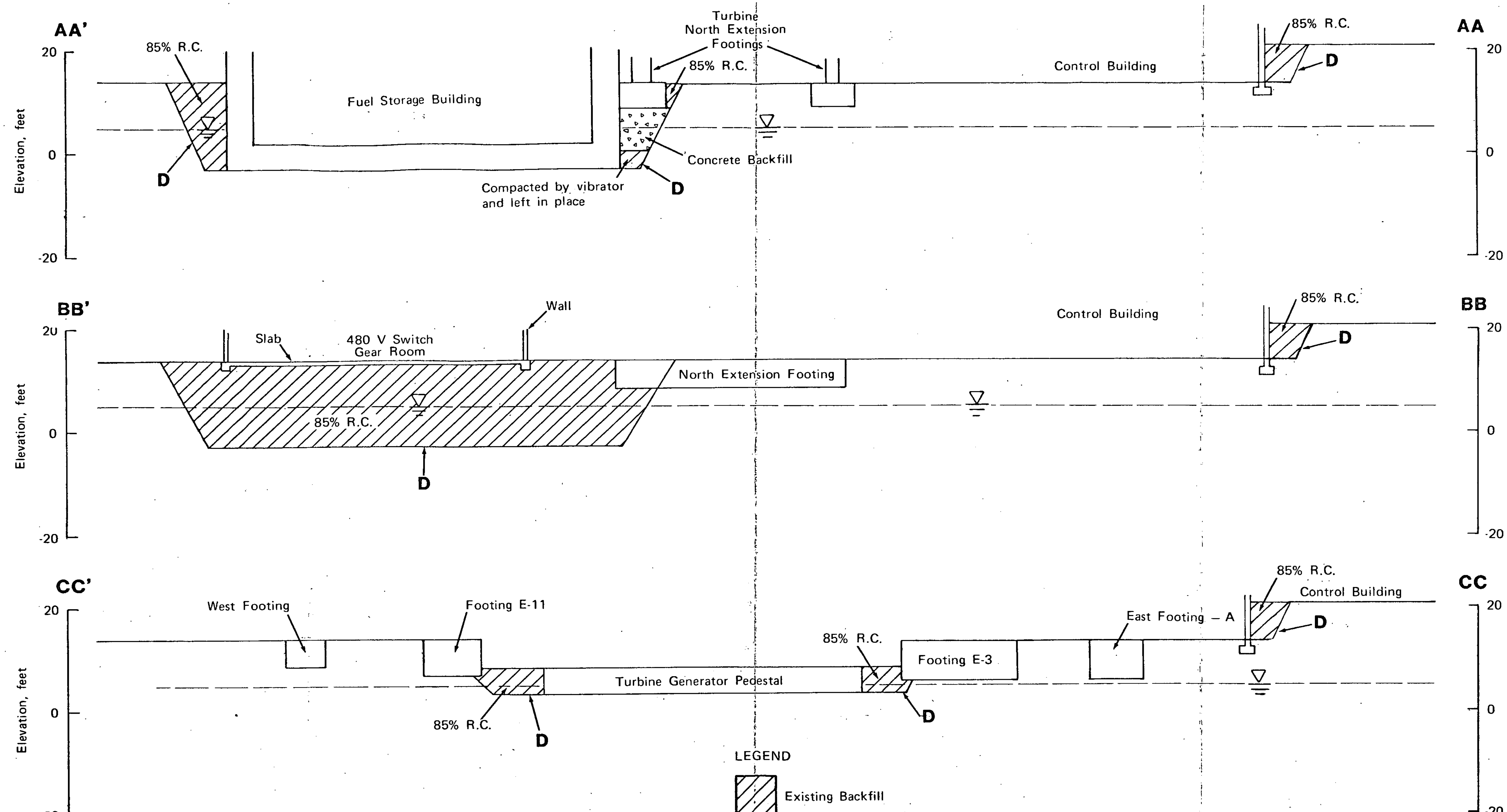
Fill above water table

Fill below water table

HH——HH'	Cross section locations (Figures 2-18 to 2-21, Section 2.0).
A	Well compacted, ~95%.
B	Moderately to well compacted, ~90 to 95%. Deep narrow fills with width <6 to 10 feet, ~85%.
C	Moderately compacted, ~85 to 90%. Deep narrow fills with width <6 to 10 feet, ~85%.
D	85% relative compaction.
(+3.5 ft)	Elevation of native San Mateo Sand at the bottom of the foundation.
—— ——	Estimated toe of cut slope
- - - - - - -	Indicates break between flatter upper slope and steeper lower slope

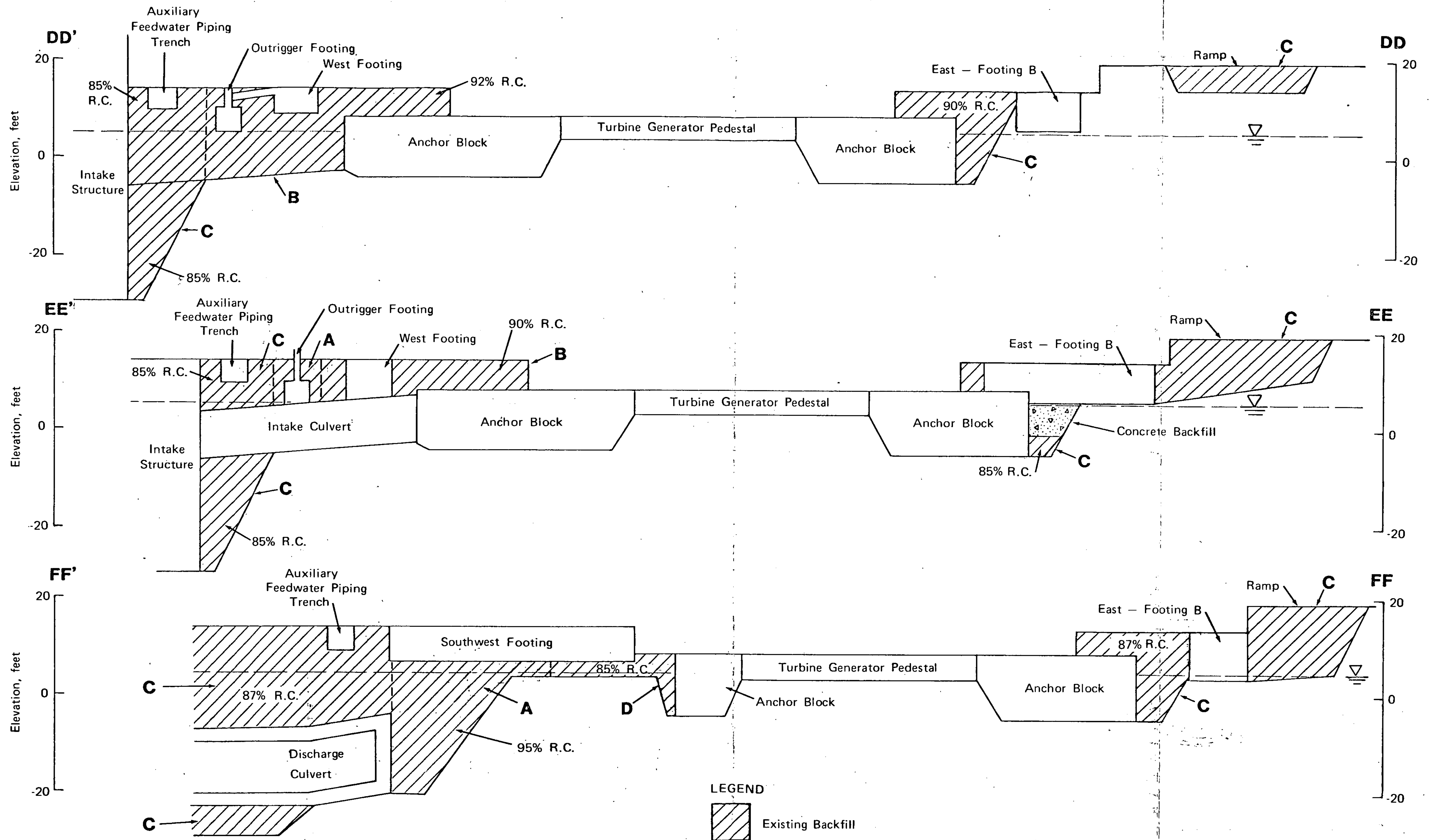
SOIL BACKFILL CHARACTERIZATION AND IDENTIFICATION

FIGURE 2-22



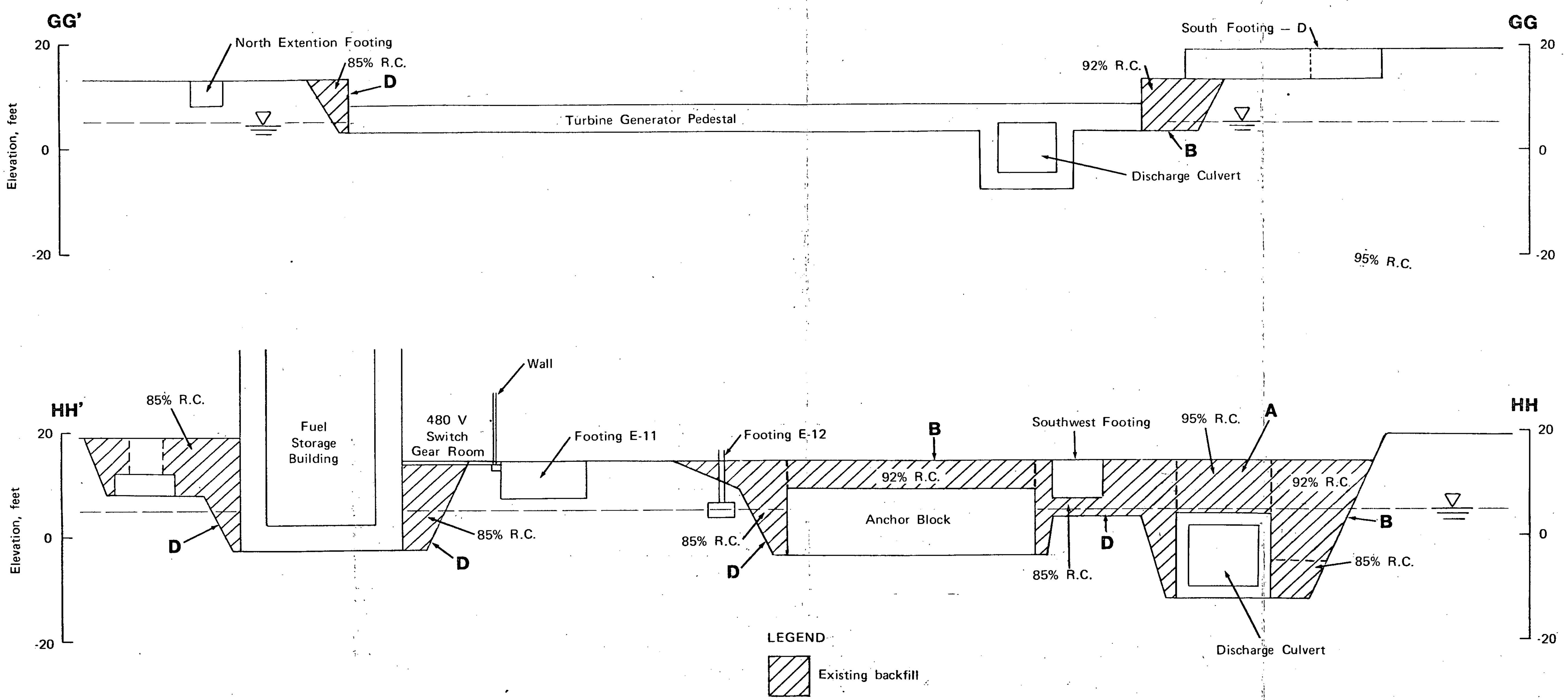
CROSS SECTIONS AA, BB, AND CC
BACKFILL CHARACTERIZATION

FIGURE 2-23



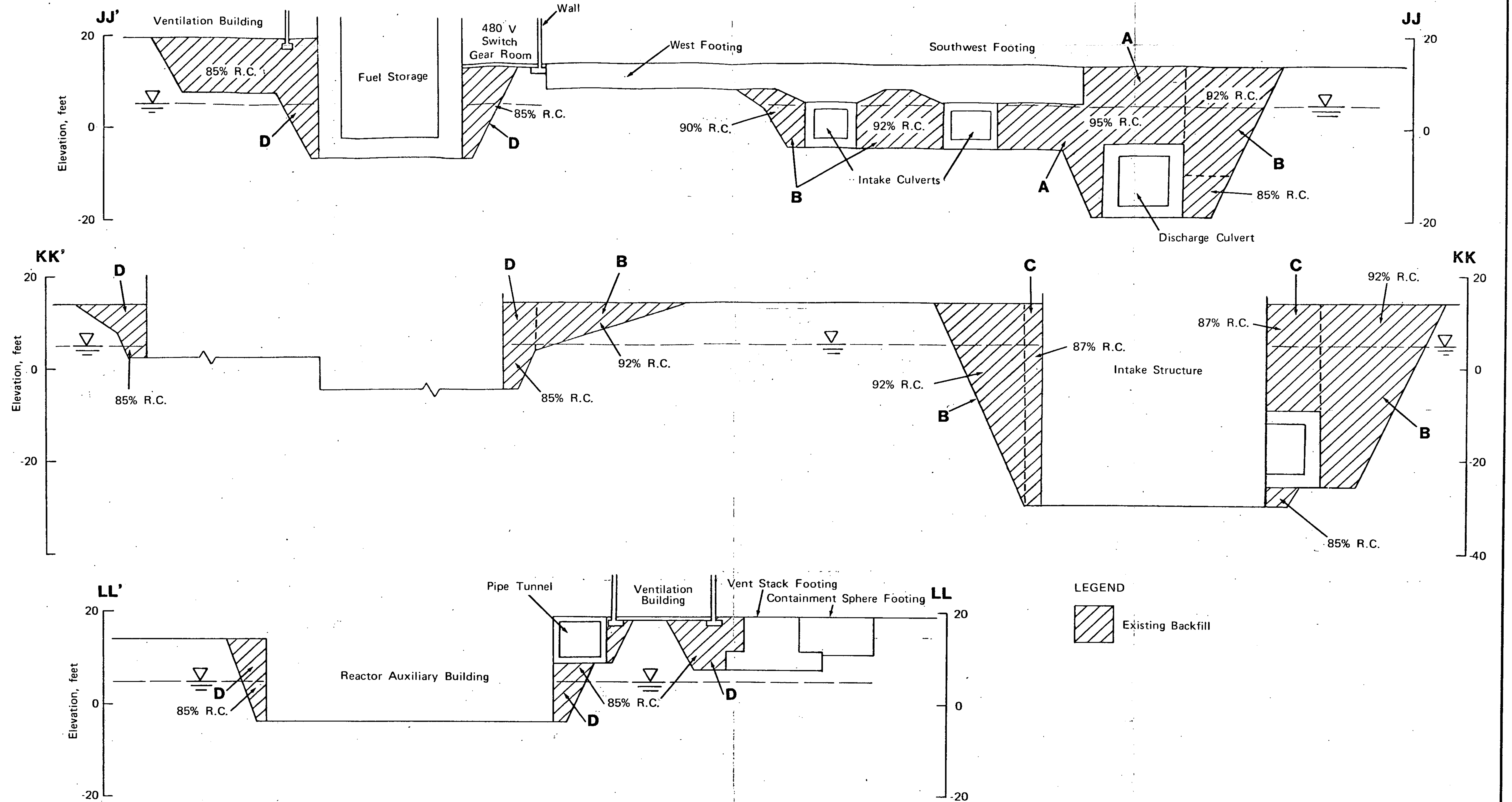
CROSS SECTIONS DD, EE, AND FF
BACKFILL CHARACTERIZATION

FIGURE 2-24



CROSS SECTIONS GG AND HH
BACKFILL CHARACTERIZATION

FIGURE 2-25



CROSS SECTIONS JJ, KK AND LL
BACKFILL CHARACTERIZATION

FIGURE 2-26

3.0 RESPONSE OF BACKFILL SOILS TO SEISMIC LOADING

3.1 General

The behavior of backfill soils at the site in response to seismic loading depends on the intensity of ground shaking, the density and geometry of the backfill, and the proximity of the water table. The intensity of seismic shaking has been defined as that which is associated with a 0.67g Housner response spectrum, stipulated for design to be caused by an M7 earthquake at a closest distance of 8 km on the hypothesized Offshore Zone of Deformation (OZD). The density and geometry of the backfills and the water table locations at the site have been established in Section 2 with the final characterization shown in Figure 2-22.

Liquefaction at this site is not a flow phenomenon because the surface slope is flat, and all of the fills are contained within limited areas as defined in Figure 2-22. Liquefaction at the site is therefore defined as the potential for the development of pore water pressure with limited strain potential. A detailed evaluation of the liquefaction potential at San Onofre Unit 1 is presented in Appendix C. Backfills with higher relative compaction, on the order of 92 percent, have a lower potential for liquefaction but may develop high pore water pressures depending on the density and fill geometry. Backfill soils with a relative compaction of 85 percent and located below the water table have a higher potential for liquefaction. Ranges in values associated with the specific results developed in Appendix C are summarized in terms of the factor of safety and pore pressure ratio (ratio of induced pore water pressure to effective confining pressure) versus depth below the water table. These ranges of values for category A, B, C, and D fill soils are given in Figures 3-1 through 3-4, respectively.

Figure 3-1 shows that the factor of safety against liquefaction for category A fill soils is on the order of 1.5 or greater with the associated induced pore water pressure ratio (r_u) being less than 0.2. This factor of safety is similar to that found for the San Onofre Units 2 and 3 plant area, as documented in Section 2.5.4.8 of the SONGS 2 and 3 FSAR (and summarized in Figure 3-5 for ease in reference). Therefore, backfill soils at the site which are compacted to a minimum of 95 percent relative compaction will exhibit the same response to seismic loadings as the native soil. The category B fill soils, delineated in Figure 2-22, will respond similar to the category A fill soils from a liquefaction standpoint in the wider portions of the fills. These soils, however, may develop high pore water pressure in the deeper, narrower portions of a fill and therefore may have low factor of safety (less than 1.0) against potential liquefaction as shown in Figure 3-2. The upper portions of category C fill soils below the water table, delineated in Figure 2-22, should have low to moderate factors of safety against liquefaction, and moderate to high potential for high pore water pressures. However, the deeper portions of category C fills located below the water table may have low factors of safety against liquefaction (less than 1.0) and may have a high potential for developing high pore water pressure as shown in Figure 3-3. The category D fill soils, delineated in Figure 2-22, may exhibit low factors of safety against liquefaction (less than 1.0) at all depths below the water table, and exhibit a high potential to develop high pore water pressures as shown in Figure 3-4.

The results of the liquefaction potential analysis discussed above and summarized in Figures 3-1 through 3-4 for the various categories of fill soils have been used to determine the effect of the response of the site backfill soils to seismic shaking. This is discussed below in Sections 3.2 and 3.3.

3.2 Settlement Response of Backfills

As shown in Figure 2-22 and discussed in Section 2.0, all major structures are supported on native soil except for small portions of the ventilation equipment building and the turbine building (discussed in Section 4). Therefore, the ventilation equipment and turbine buildings are the only structures where seismic induced soil settlements are considered. In addition to the structures, there are equipment foundations and structural components (discussed in Section 5) which are considered to be susceptible to seismic-induced settlements. The general procedures used to estimate settlements are discussed below with specific estimates given for the ventilation equipment building and turbine building in Section 4 and the equipment foundations and components in Section 5.

Estimates of settlement for fills above the water table were based on procedures suggested by Silver and Seed (Reference 4). The procedure incorporates relative density of fill, duration of shaking in terms of the number of cycles of cyclic loadings and the estimated level of applied cyclic shear strains, and estimates the resulting volumetric strains in the soil. For fills below the water table, estimates of settlement were obtained using the procedures suggested by Lee and Albaisa (Reference 5). This procedure also estimates the volumetric strain in cohesionless soil subjected to various levels of pore-pressure increments, in terms of the ratio of excess pore pressure to initial effective confining pressure. The procedure incorporates the relative density of the soil and the initial effective confining pressure. For the purpose of the present evaluation, the settlement estimates were based on the backfill characterization at the location of the individual foundations shown in Figure 2-22. In addition, the induced pore pressure, summarized for the various fill categories in Figures 3-1 through 3-4, together with the number of cycles of cyclic loading, given in Appendix C, were utilized in arriving at the estimates for settlement of the various foundations. The specific procedure for estimating settlements is shown by example in Appendix D.

As discussed in Appendix D, the estimates of settlement of fills, both above and below the water table, were adjusted to account for factors such as variations in the depth of fill below the foundations and the proximity of adjacent boundaries which constrain the development of shear strains in the fill. These adjustments were made based on engineering judgement and the experience of Woodward-Clyde Consultants (WCC). The settlements estimated were reviewed in detail, structure by structure, and agreed upon by the project consulting review board (Drs. I. M. Idriss, H. B. Seed, and R. L. McNeill). The settlement values are summarized in Sections 4 and 5 for each building or equipment foundation and conservatively reflect the estimates arrived at in this manner. The potential response of the various foundations to the settlements were postulated on the basis of considerations which included: size of foundation, configuration of underlying fill, proximity of the water table to the foundation, and interfaces with the walls of adjacent structures. Based on actual observations of settlements made in the field and on the results of mechanistic analyses of pore pressure induced settlements by Seed, Martin, and Lysmer (Reference 6), all liquefaction induced settlements for fills below the water table would occur after the shaking had ceased and therefore are characterized as "post-seismic settlements." The settlements for fills above the water table could occur during seismic shaking. However, because the largest settlements are dominated by the effects of high pore water pressures or liquefaction, most of these settlements would occur after seismic shaking has ceased. For example, of the estimated settlement of 3 to 5 inches documented for the Turbine plant cooler foundation in Appendix D, about one inch would be expected to occur during seismic shaking with the remainder being classified as post seismic settlement.

The seawall in the vicinity of the intake conduits is founded on up to 21 ft of fill. This fill on the plant side of the seawall is characterized in Section 2 of this report as being moderately well compacted to between 85 to 90 percent relative compaction. The soil on the ocean side of the seawall would not experience high pore water pressure because special vertical drains have been provided as discussed in Appendix E. The soil on the plant side of the seawall, however, could experience high pore water pressures during DBE level seismic shaking. These high pore water pressures could lead to settlements of the soil beneath and adjacent to the seawall of the order of 4 to 6 inches assuming the gravel drains were not in place. It is judged that this settlement would be on the order of 3 inches considering the stabilizing influence of the existing gravel drains. The seawall is a z-section sheetpile wall with a gunite covering. This type of wall is very flexible and can accommodate distortion on the order of several inches without failure. In fact this type of wall was originally developed to avoid failures that had been observed to result from the settlement of walls where straight section sheet piling was used. The Unit 1 seawall will easily accommodate the estimated about 3-inch post seismic settlement as well as the estimated extreme settlement of 4 to 6-inches.

The offshore conduits were constructed essentially at the base of the excavation with an estimated maximum 2 ft of fill beneath them locally. For this reason, maximum post seismic settlements of the conduits would be less than one inch. This level of settlement would easily be accommodated by the pipe joints which can withstand at least 6 inches of differential settlement.

3.3 Assessment of Effect of Backfill on SEP Parameters

The procedures used to develop SEP design parameters are summarized in the "Balance of Plant Structures Seismic Reevaluation Criteria" (Reference 3). These procedures are appropriate for foundations bearing on native or category A fill soils or embedded into these soils. For those foundations bearing on category B, C, or D fill soils or embedded into these soils, some modifications to the SEP design parameters have been made. Category B, C, and D fills extending below the water table may experience high pore water pressures (see Figures 3-2, 3-3, and 3-4) during the seismic shaking associated with the 0.67g Housner design spectrum event. This will cause lower soil stiffnesses and higher damping. The embedment effects on the soil stiffness parameters (spring constants) could be reduced, and the lateral pressures could increase from the initial parameters which were developed assuming a minimum of 95 percent relative compaction existing in the backfill material. For soil fills above the water table and compacted to densities which are less than 95 percent relative compaction only slight changes in stiffness parameters are expected as discussed below.

To assess the influence of backfill conditions on the stiffness parameters for the various foundations, the total stiffness of an embedded foundation, K', was considered to be comprised of two components: the unembedded stiffness value, K; and an increment due to embedment, ΔK, where;

$$K' = K + \Delta K$$

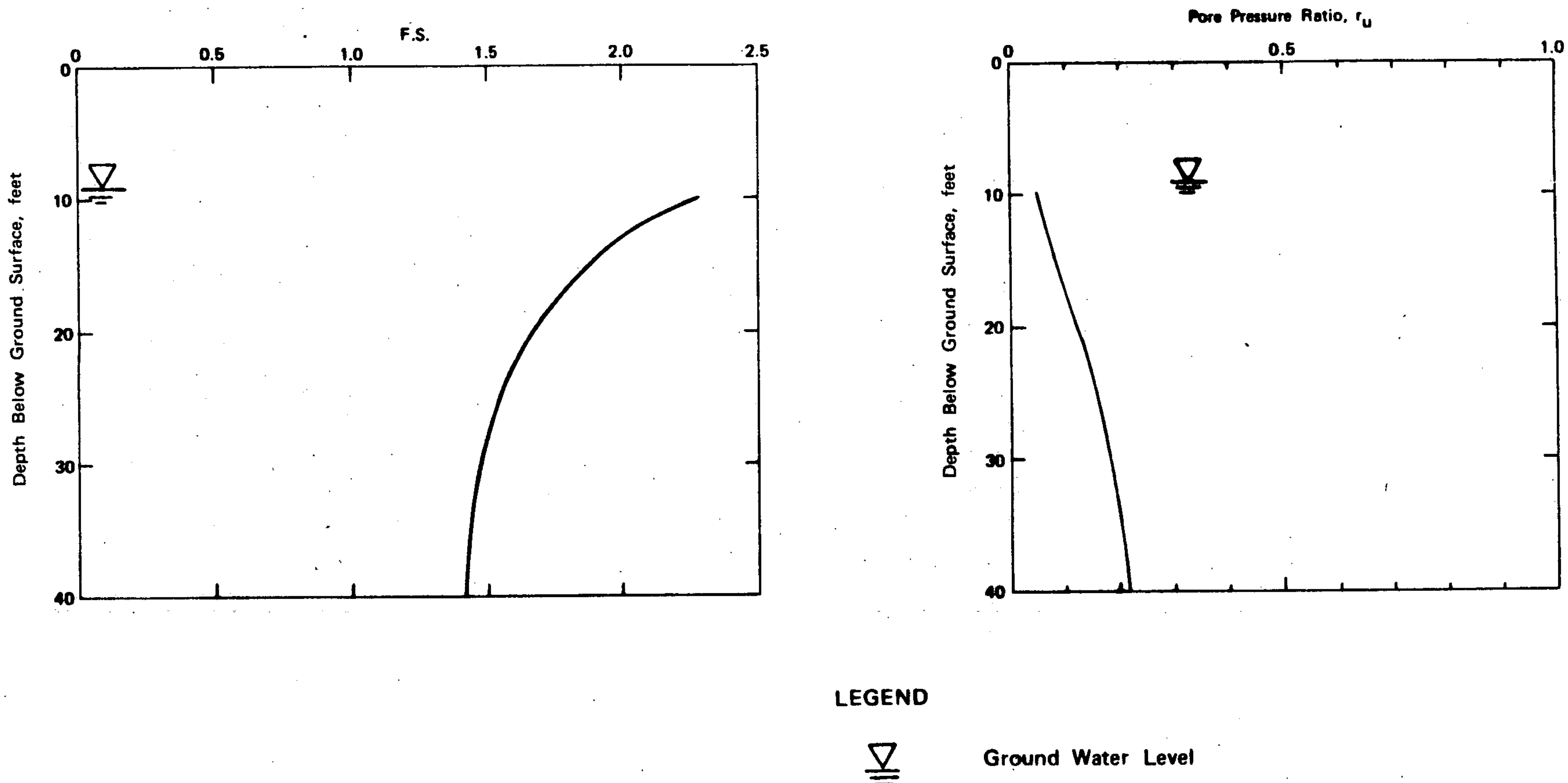
For foundations bearing and embedded in material of the same density, ΔK is proportional to K. The constant of proportionality is defined as C_2 in Reference 3 and is equal to the ratio of K'/K and is dependent upon the ratio of the embedment depth to the equivalent radius of the foundation. The value of K is primarily dependent on the geometry of the foundation, the shear modulus and Poisson's ratio for the supporting soil. Thus, for a given foundation, the influence of change in the supporting soil conditions is evaluated by considering the change in the shear modulus value of the soil. Because the increase in stiffness, ΔK, due to embedment can be expressed in terms of C_2 and K, ($\Delta K = K [C_2-1]$) the influence of the backfill can also be assessed in terms of the change in the shear modulus of the soil.

The reduction factor, R_f, that converts the shear modulus for San Mateo sand, compacted to a minimum of 95 percent relative compaction, to a shear modulus that is appropriate for a lower relative compacted San Mateo sand was developed considering data from Seed and Idriss (1970) (Reference 7) for cohesionless soils as discussed in detail in Appendix F. As developed in Appendix F for the high strain developed during DBE level ground shaking it was determined that R_f = 0.74 for 80 percent relative compaction and R_f = 0.84 for 90 percent relative compaction are appropriate. When the soil underlying a foundation undergoes initial liquefaction, r_u = 1.0, the embedded stiffness value is considered to be a small fraction of the value for nonliquefied soil. For the present evaluation, the unembedded stiffness for liquefied soil was judged to be one-tenth of the value for soil not experiencing initial liquefaction.

Based on WCC experience with static and dynamic properties of compacted San Mateo sand, it is concluded that San Mateo sand backfill compacted to a minimum relative compaction of 95 percent has as good a contact with the foundation sides as those cases with foundations constructed directly against the native soil without forms. Thus, a contact efficiency factor of 1.0 was used for backfills with a minimum relative compaction of 95 percent. For backfills with relative compactions of 85 or 90 percent, contact efficiency factors of about 1/2 and 2/3, respectively, were judged appropriate. In addition to the above considerations, a further reduction in ΔK, by a factor of 0.5 was judged appropriate where the supporting soil beneath soil fill providing embedment is likely to experience initial liquefaction.

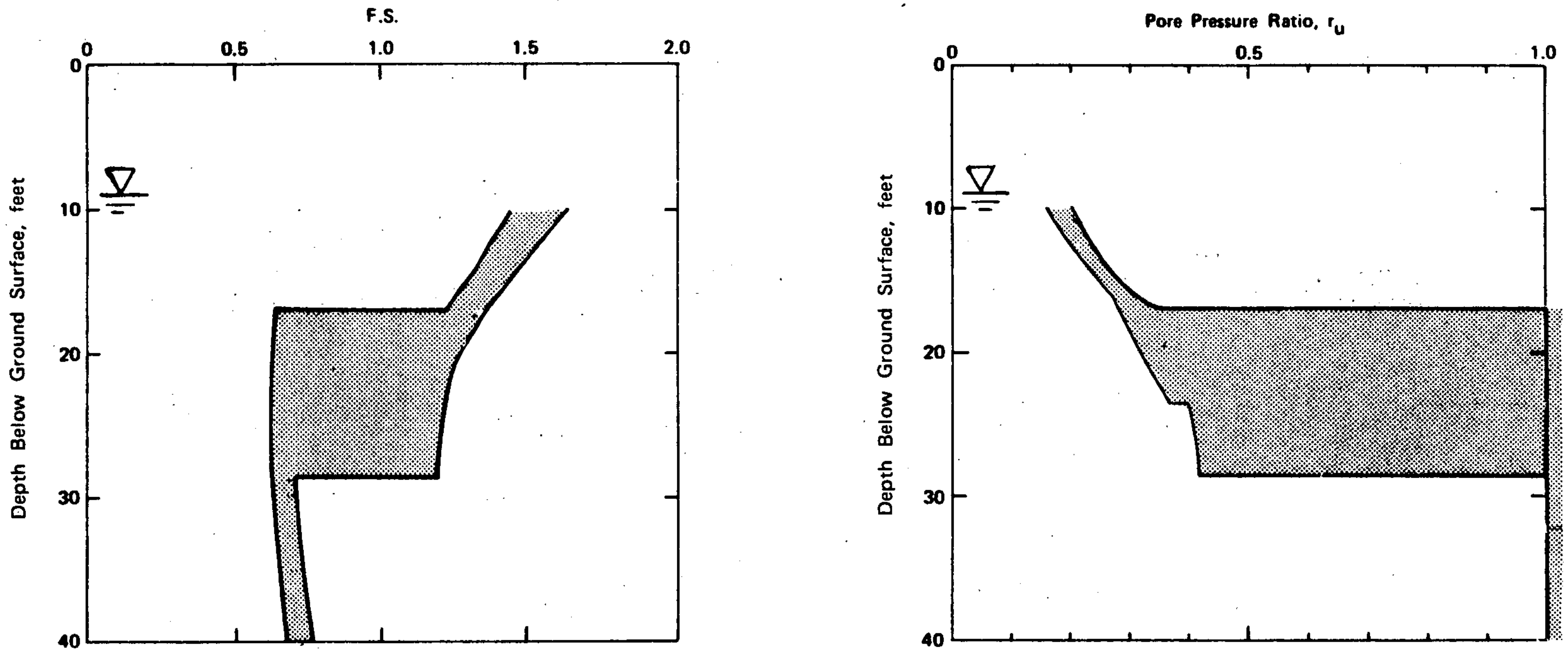
The above described procedures were utilized to obtain the best estimate of the stiffness parameters based on the average embedment and bearing conditions associated with a given foundation. For the case where the embedment and/or bearing soils are subject to liquefaction a range of values for the nonliquefied to the liquefied condition were developed for the evaluation.

The effect of soils compacted to densities less than 95 percent relative compaction is not considered significant except for the case where initial liquefaction occurs. This effect is significant for soils below the water table exhibiting pore water pressure ratios, r_u, of 1.0, as developed in Figures 3-2, 3-3, and 3-4. For these cases, an applied lateral pressure equal to the total overburden pressure ($r_u = 1$) characterizing the liquefied fill, is given as shown in Figure 3-6.



F.S. & r_u VS. DEPTH FOR CATEGORY A FILL SOILS

FIGURE 3-1



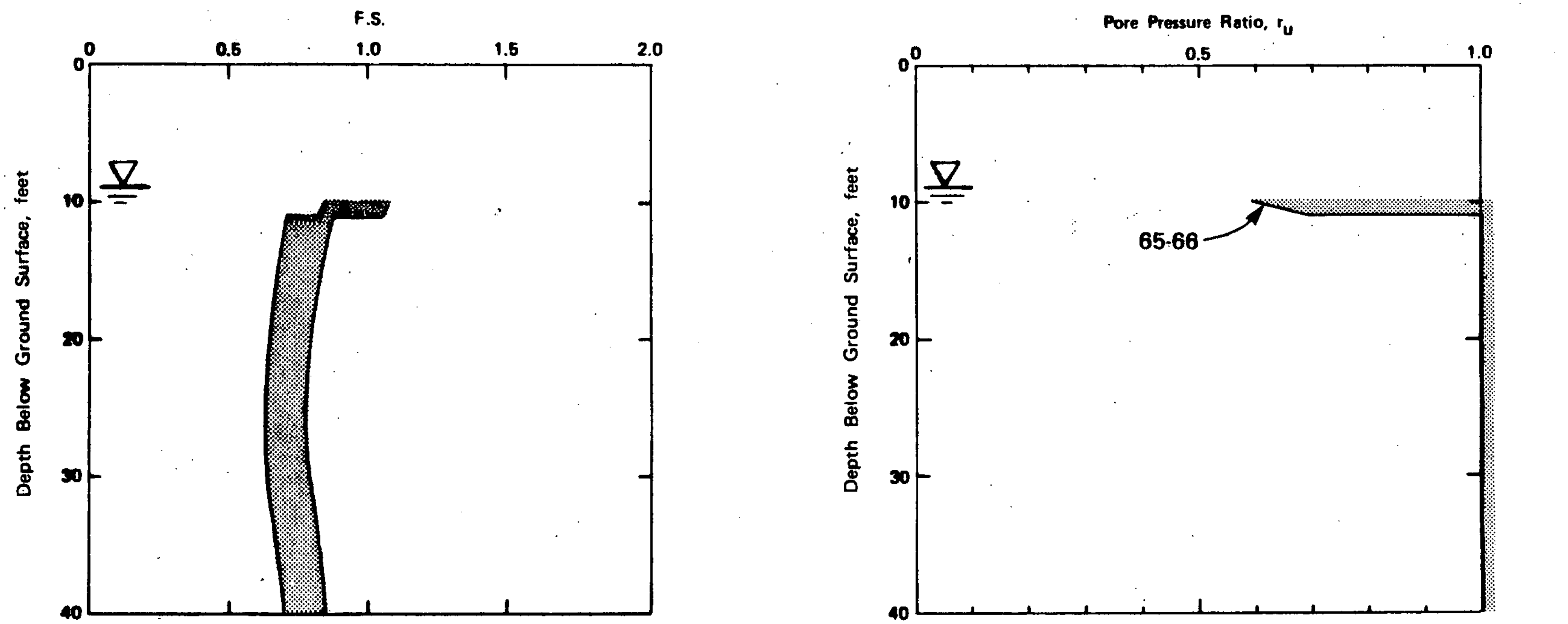
LEGEND

▽ Ground Water Level

NOTE: Range of values are based on results presented in Appendix C.

F.S. & r_u VS. DEPTH FOR CATEGORY B FILL SOILS

FIGURE 3-2

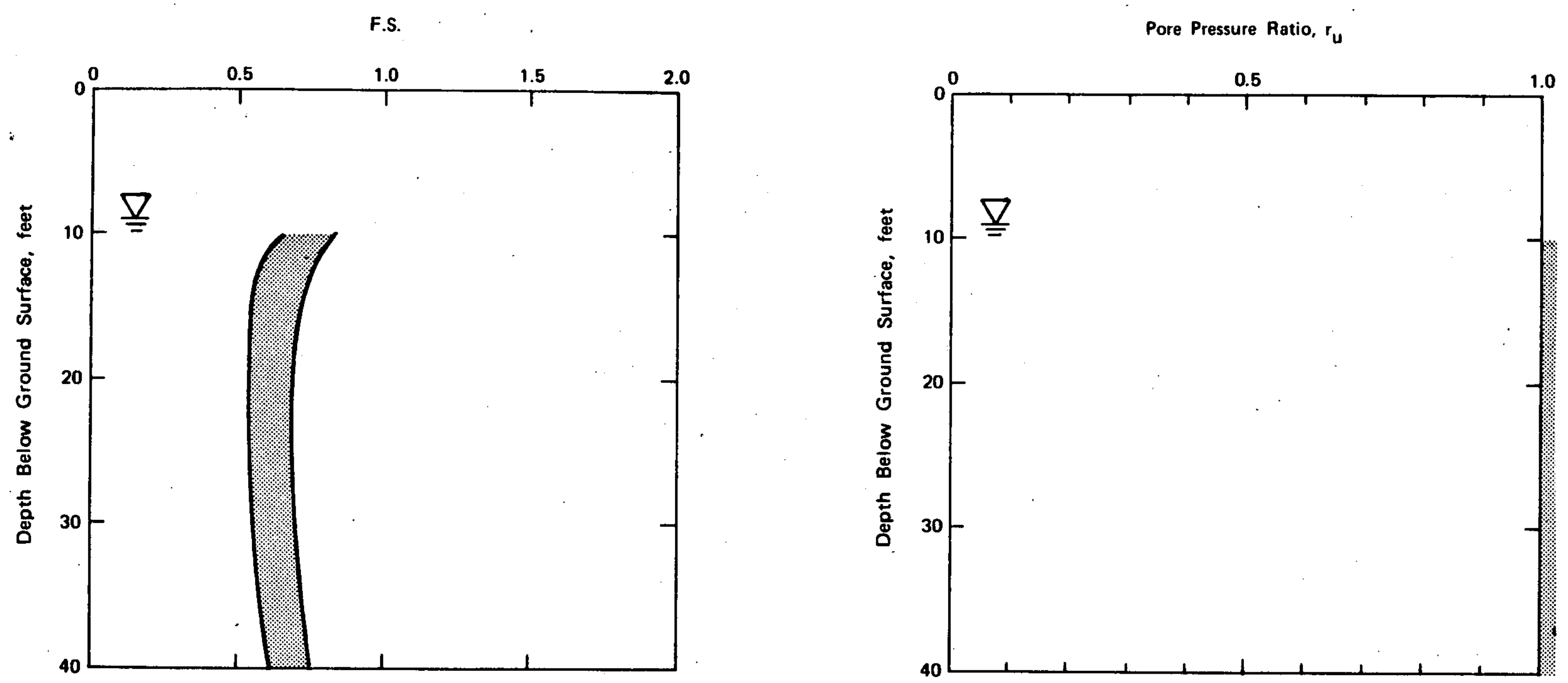


LEGEND

▽ Ground Water Level

NOTE: Range of values are based on results presented in Appendix C.

F.S. & r_u VS. DEPTH FOR
CATEGORY–C FILL SOILS
FIGURE 3-3



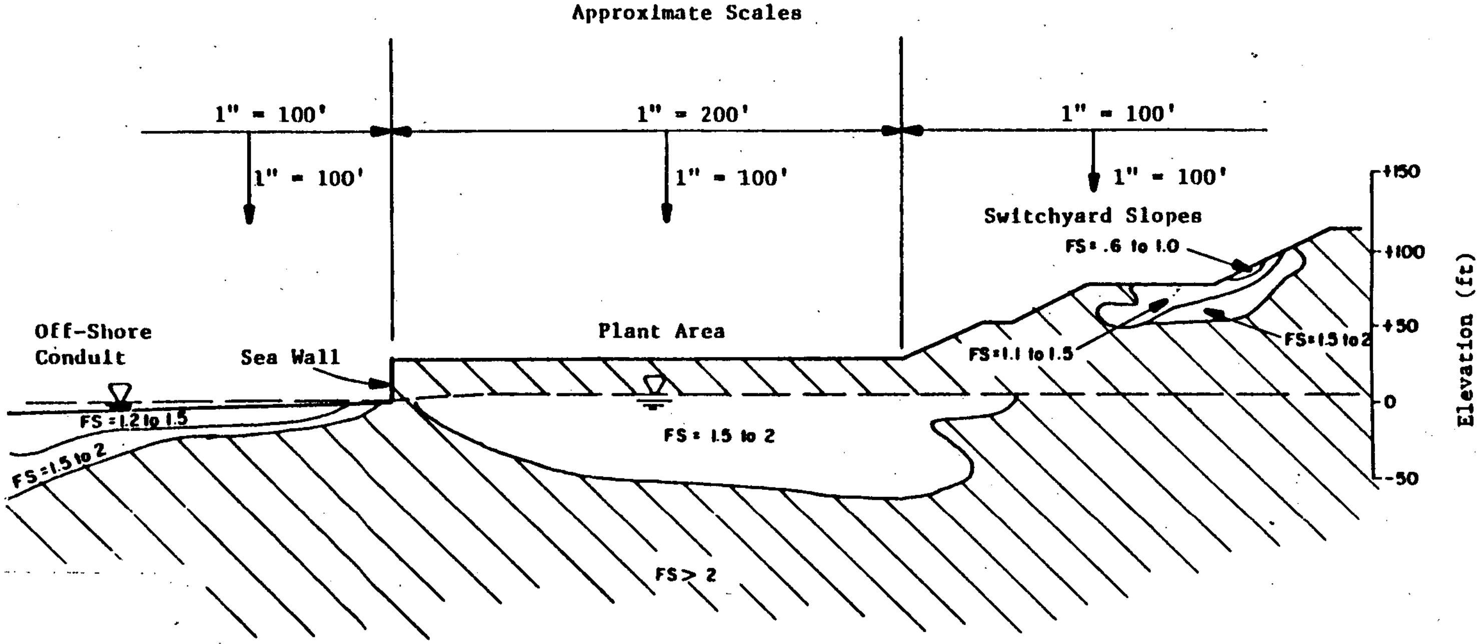
LEGEND

▽ Ground Water Level

NOTE: Range of values are based on results presented in Appendix C.

F.S. & r_u VS. DEPTH FOR
CATEGORY D FILL SOILS

FIGURE 3-4

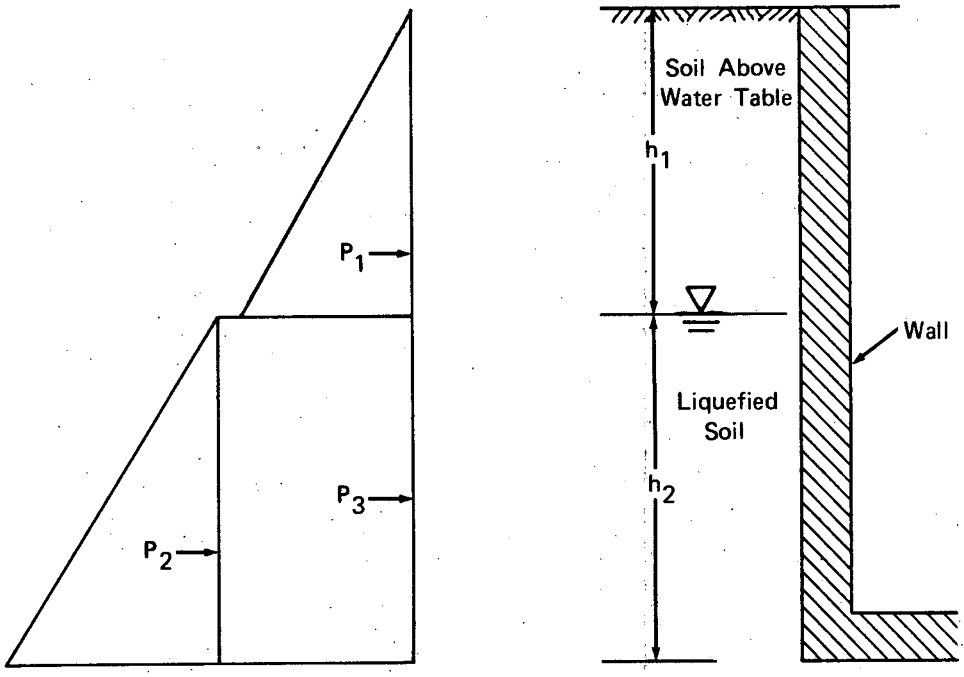


Notes:

1. F.S. defined as the ratio of the stress necessary to cause ± 5% strain to the induced stress.
2. ∇ = water table
3. Based on choice of worst conditions encountered for all analysis values.

SUMMARY OF FS AGAINST LIQUEFACTION FOR NATIVE SOILS AT THE SONGS UNITS 2 AND 3 SITES

FIGURE 3-5



Pressure Distribution

$P_1 = k_A \frac{\gamma_T h_1^2}{2}$ $\qquad k_A$ - as recommended in Table 4 of Reference 2

$P_2 = 130 \times \frac{h_2^2}{2}$ $\qquad$ - Pressure due to liquefied soil

$P_3 = (\gamma_T h_1)\ h_2$ $\qquad$ - Pressure due to surcharge (or non-liquefied soil) above liquefied soil

SCHEMATIC SHOWING PROCEDURE TO EVALUATE LATERAL PRESSURE ON WALLS

FIGURE 3-6

REFERENCES

1. "Material Property Studies, San Onofre Nuclear Generating Station", San Onofre Nuclear Generating Station Units 2 and 3 PSAR, Amendment 11, Attachment A3 to Appendix 2E, March 13, 1972.

2. Balance of Plant SONGS Unit 1. "Soil-Structure Interaction Methodology Report", Revision 1, Woodward-Clyde Consultants, Orange, California, July 1978.

3. "Balance of Plant Structures Seismic Reevaluation Criteria" San Onofre Nuclear Generating Station, Unit 1, February 17, 1981.

4. Silver, M. L., and Seed, H. B., Settlement of Dry Sands During Earthquakes, Journal of the Soil Mechanics and Foundation Division, ASCE, v. 98, no. SM4, April 1972.

5. Lee, K. L., and Albaisa, A., Earthquake induced settlements in saturated sands, Journal of the Geotechnical Engineering Division, ASCE, Vol. 100, No. GT4, Proc. Paper 10496, April 1974, pp. 387-406.

6. Seed, H. B., Martin, P. P., and Lysmer, J., The generation and dissipation of pore water pressures during soil liquefaction, Report No. EERC 75-26, Earthquake Engineering Research Center, University of California, Berkeley, California, August 1975.

7. Seed, H. B., and Idriss, I. M., Soil moduli and damping factors for dynamic response analyses, Report No. EERC 70-10, Earthquake Engineering Center, University of California, Berkeley, California, 1970.

APPENDIX A

ANALYSIS OF 1964-65 "ORIGINAL PLANT" CONSTRUCTION PHOTOGRAPHS SONGS UNIT 1 FACILITIES

A-1.0 INTRODUCTION

A detailed assessment has been made of original plant construction photographs to evaluate the areal extent and slope inclinations of the excavations for the Turbine Generator Pedestal-Intake Structure, the Fuel Storage Building, and the Reactor Auxiliary Building at the SONGS Unit 1 site. This assessment is based upon 35 mm photographs acquired in 1964 and 1965 and upon a site plan showing the locations, dimensions, and elevations of various Unit 1 facilities at the time of construction. In general, the analysis of the construction photographs indicates that the depth, slopes and areal extent of the excavations for the major structures are essentially in agreement with initial interpretations reported in the 17 August 1982 "Soil Backfill Conditions" report and shown in Figure A-1. Specifically, Figure A-1 shows the distribution of major fills at the site overlain by a heavy line representing the currently interpreted surface contact between backfill and native San Mateo Sand. The following sections describe the key elements of the analysis and the results of the assessment.

A-2.0 METHODOLOGY

The assessment of the areal extent, configuration, and slope inclination of the excavations was based primarily on the examination and analysis of the 35 mm oblique photographs. All the available construction photographs were examined and reviewed, and 17 high-quality photographs were selected for detailed analysis. The analysis of the photographs consisted of applying basic photogrammetry principles (Williams 1969) to locate and map the configuration of the top of the excavation slopes and the slope inclination of the excavations. These basic photogrammetry principles are based on the fact that: 1) on any photograph, the image of any vertical or horizontal parallel lines converge to vanishing points (located off the photograph) or are parallel depending on the angle at which the photograph was taken with respect to the horizon; 2) features or objects that have a rectangular shape in plan view and within an area photographed will have a definable trapezoidal shape in the photograph; and 3) given the location and dimensions of a known feature in an area photographed, the location of other features within the area photographed can be located with reasonable accuracy (Williams, 1969).

A-2.1 Areal Extent of the Tops of Excavations

The areal extent and configuration of the top of the excavation for the Turbine Generator Pedestal-Intake Structure, Reactor Auxiliary Building, and Fuel Storage Building were delineated by using the photogrammetry principles discussed in Section A-2.0. The photogrammetric technique, which is described in detail by Williams (1969), provides a method to construct a plan map of features visible on small, oblique photographs taken from the air or on the ground with a hand-held 35mm camera. The construction photographs used in the analysis are listed in Table A-1. The discussion that follows summarizes the photogrammetric mapping technique used to delineate the extent of the top of the excavations shown by the heavy line in Figure A-1.

The initial step in the assessment consists of the construction of two diagrams (Figures A-2 and A-3) that provide a medium for transferring features visible on the oblique photographs to a plan map. The basic principle is that a square or rectangular feature appears as a trapezoid on an oblique photograph. The first step is to construct a trapezoidal grid array on the photograph (Figure A-2) based on a feature (structure) in the photograph that has a known rectangular or square dimension and can be identified on an existing plan map. A rectangular grid array is then constructed on the plan map (Figure A-3) based on the same feature identified on the photograph. Because the grid arrays on the photograph and on the plan map are based on the same feature, the location of other features on the same horizontal plane can be transferred from the photograph to the plan map. Utilizing the map and photo grid arrays to locate features or points that lie at elevations different than the original horizontal plane, a principal vertical plane section is constructed as shown in Figure A-4. The function of the principal vertical plan section is to adjust the grid array on the photograph to correspond with the elevation of the feature of interest relative to the elevation of the plane of the original grid array. Using the adjusted grid array, features or points located at various elevations can be transferred from the photograph to the plan map.

The principal plane section is also used to identify the location of the Isocenter (ISO) point along the trace of the principal vertical plane on the photograph and on the plan map. The ISO point is the one point where angles measured off the photo can be directly transferred to the plan map (Williams 1969). The ISO point is also used to triangulate the location of features or points that are visible on two or more photographs.

A-2.2 Slope Inclination Analysis

To assess the slope inclination of the excavations, lines were plotted along the image of vertical features such as edges of buildings or columns, construction forms, and well casings, that were visible on the photographs analyzed (Table A-2). For each photograph analyzed, a number of vertical lines were drawn and were projected off the photograph to their convergent point (i.e., point PP in Figure A-2) to insure that they represented true vertical lines on the photograph. A line was then drawn along the apparent slope inclination of the excavation. The angle measured between the apparent slope inclination visible on the photograph and the vertical line represents the apparent inclination of the excavation slope. The true angle of any inclined slope or surface is always measured perpendicular to the trend of the slope. Only in those photos where the plane of the photo is nearly perpendicular to the trend of the slope do measured slope angles represent true slope angles. In most cases the slope angles measured from the construction photographs represent an apparent slope angle which is less than the true slope angle. Thus the use of the measured apparent slope angles is a conservative slope condition (i.e., yields a flatter slope than actually exists).

A-3 ASSESSMENT

Based on the analysis of the construction photographs and field data acquired since the construction of the Unit 1 facilities in 1964-1965, the location and configuration of the top of the excavations were found to be as shown in Figure A-1. The analysis of the construction photographs also indicated that the average slope inclination of the excavations was about 1/2 to 1 (horizontal to vertical).

The following subsections discuss the specific assessments made for the excavations for the Turbine Generator Pedestal-Intake Structure, Fuel Storage Building, and Reactor Auxiliary Building.

A-3.1 Turbine Generator Pedestal - Intake Structure

The construction plans indicated: 1) that the excavations for the Turbine Generator Pedestal-Intake Structure had an elongated configuration in the east-west direction with irregular north and south sides to accommodate the shape of the Turbine Generator Pedestal; and 2) access roads into the excavation were located in the southeast and southwest corners of the excavation (Figure A-1). Based on the analysis of the construction photographs P13 through P16 for the Turbine Generator Pedestal-Intake Structure (Figures A-5 through A-8), the top of the excavation in area C-3 (reference grid on Figure A-1) has a semicircular shape. There is a V-shaped reentrant that opened to the south and was located near the western edge of area C-3 in Figure A-1. In area C-3 Figure A-1), analyses of the construction photographs and the field data (discussed in Section 2.0 along Cross-Section JJ to JJ' (Figure 2-21) show minor differences in the elevation at which the native San Mateo Sand was encountered in the trench excavation relative to the estimated top of the excavation. This is partly due to the fact that the slope inclination for the upper portion of the excavation in this area is about 1-1/2 to 1. The best and most conservative estimates of the top of the excavation are shown in Figure A-1.

The study of the remainder of the top of the excavation for the Turbine Generator Pedestal-Intake Structure yield similar conclusions as to the location of the top of slope as were made in previous studies. The one exception involves the access road located in area D-1 (Figure A-1). The access road trends toward the northeast as shown in Figure A-1 rather than to the southeast.

Examination of construction photographs P14 and P6, Figures A-6 and A-9 respectively, indicates that a gently westward sloping bench of Native San Mateo Sand is located between the west Anchor Block and the eastern end of the Intake Structure. (Figure A-1 and Section EE-EE', Figure 2-19). The gentle inclination of the bench and the steeper slope inclination at the western end of the bench toward the Intake Structure excavation are clearly shown in Figure A-9. In Figure A-9, the dashed line and Y symbols indicate the approximate edge of the bench and top of the slope face respectively. The location of the edge of the bench and therefore the top of the cut slope around the bench is best seen in Figure A-10. A line has been drawn on this photo along the west side of the Fuel Storage Building and extended southward toward the Turbine Generator Pedestal-Intake Structure excavation. Although a rigorous reconstruction of this photograph has not been made, a visual examination indicates that the top of the cut slope on the west end of the bench is located just east of the line extending from the west wall of the Fuel Storage Building. When difference in elevation between the top of the form work for the Fuel Storage Building and the bench is accounted for, the top of the cut slope on the west end of the bench is found to lie even closer to the west edge of the Fuel Storage Building. The location of the top of cut slope along the west end of the bench in native San Mateo sand is in good agreement with the location shown in Figure A-1.

Based upon the analysis of photographs P6, P4, P7, P8, P3, and P5 of the excavation as shown in Figures A-9 and A-11 through A-15, respectively, and upon a visual examination of the photographs used to delineate the top of the excavation, the slope inclination for the sides of the Turbine Generator Pedestal-Intake Structure excavation is estimated to be 1/2 to 1. Locally the upper one-third of a part of the excavation in area C-3 to C-5 (Figure A-1) was found to exhibit a flatter slope. Examination of photographs indicates that the upper one-third of the slope in these areas of the excavation had an inclination of the order of 1 to 1 or 1-1/2 to 1.

A-3.2 Fuel Storage Building

Based on the analysis of construction photograph P13 shown in Figure A-5, the location and configuration of the top of the Fuel Storage Building excavation is in agreement with previous interpretations as shown in Figure A-1. No slope inclination measurements were made for the Fuel Storage Building excavation due to the lack of useable construction photographs for this purpose. However, the steepness of the north and west sides of the excavation and the relatively small distance between the top of the slope and the wall visible in Figure A-5, would suggest that the slope is very steep, on the order of 1/2 to 1 or steeper.

A-3.3 Reactor Auxiliary Building

The excavation plans for the Reactor Auxiliary Building show the excavation being rectangular in shape with the southern half of the excavation being wider than the northern half. Analysis of construction photographs P10, P17, P11, P12, and P9 (Figures A-16 through A-20 respectively), indicates that the configuration and location of the top of the excavation are correctly shown on Figure

A-1. In addition, a semi-circular excavation for the Vent Stack Foundation was identified east of the main Reactor Auxiliary Building excavation adjacent to the Containment Sphere and Fuel Storage Building in photograph P10 (Figure A-16). The areal extent of this excavation is delineated in Figure A-1 (area B-3).

The configuration of the excavation along the north side of the Reactor Auxiliary Building is somewhat irregular as shown in Figure A-1 (Area A-3 to A-4). The irregularity appears to be due to a relatively flatter slope inclination for the upper one-fourth to one third of the excavation as opposed to the lower portion of the slope. As shown in photograph P17 (Figure A-17) the shape and width of the reentrant along the top of the excavation at the northwest corner suggests that this location may have been an access road.

The lower portion of the south side of the excavation for the Reactor Auxiliary Building is a linear 1/2 to 1 cut. However, the top of the excavation is irregular especially along the western two-thirds of the excavation. The top of the excavation along this portion of the south side has a semi-circular configuration as shown by the diagonally lined area in Figure A-1 (Area C-4) and curved dashed lines on Figure A-20. The configuration and extent of the top of excavation in this area is due to a relatively flatter slope inclination for the upper 5 to 8 feet of the excavation compared to the lower portion which has an average slope inclination of 1/2 to 1. This area was probably used for access during the excavation for the building foundation.

Based on the slope inclination analysis of photos P9, P1, and P2 (Figures A-20 through A-22, respectively) the inclination of the Reactor Auxiliary Building excavation is 1/2 to 1 or steeper except for the upper 5 to 10 ft of the excavation along the north and south sides of the excavation which has an estimated slope of 5 to 1.

TABLE A-1

CONSTRUCTION PHOTOGRAPHS
TOP OF EXCAVATION DELINEATION

Facility	Photo Number	Photo Date	Photo View	Figure Number
Reactor Auxiliary Building	P10	8/23/65	South	A-16
Reactor Auxiliary Building	P11	9/3/65	South	A-18
Reactor Auxiliary Building	P12	7/23/65	South	A-19
Reactor Auxiliary Building	P9	7/30/65	East	A-20
Reactor Auxiliary Building	P17	7/30/65	North	A-17
Turbine Generator Pedestal Excavation	P13	10/27/64	West	A-5
Turbine Generator Pedestal and Intake Structure Excavation	P14	10/27/64	Northeast	A-6
Turbine Generator Pedestal and Intake Structure Excavation	P15	8/26/64	Northwest	A-7
Turbine Generator Pedestal Excavation	P16	8/26/64	Northeast	A-8
Turbine Generator Pedestal and Intake Structure Excavation	P18	11/30/64	North	A-10

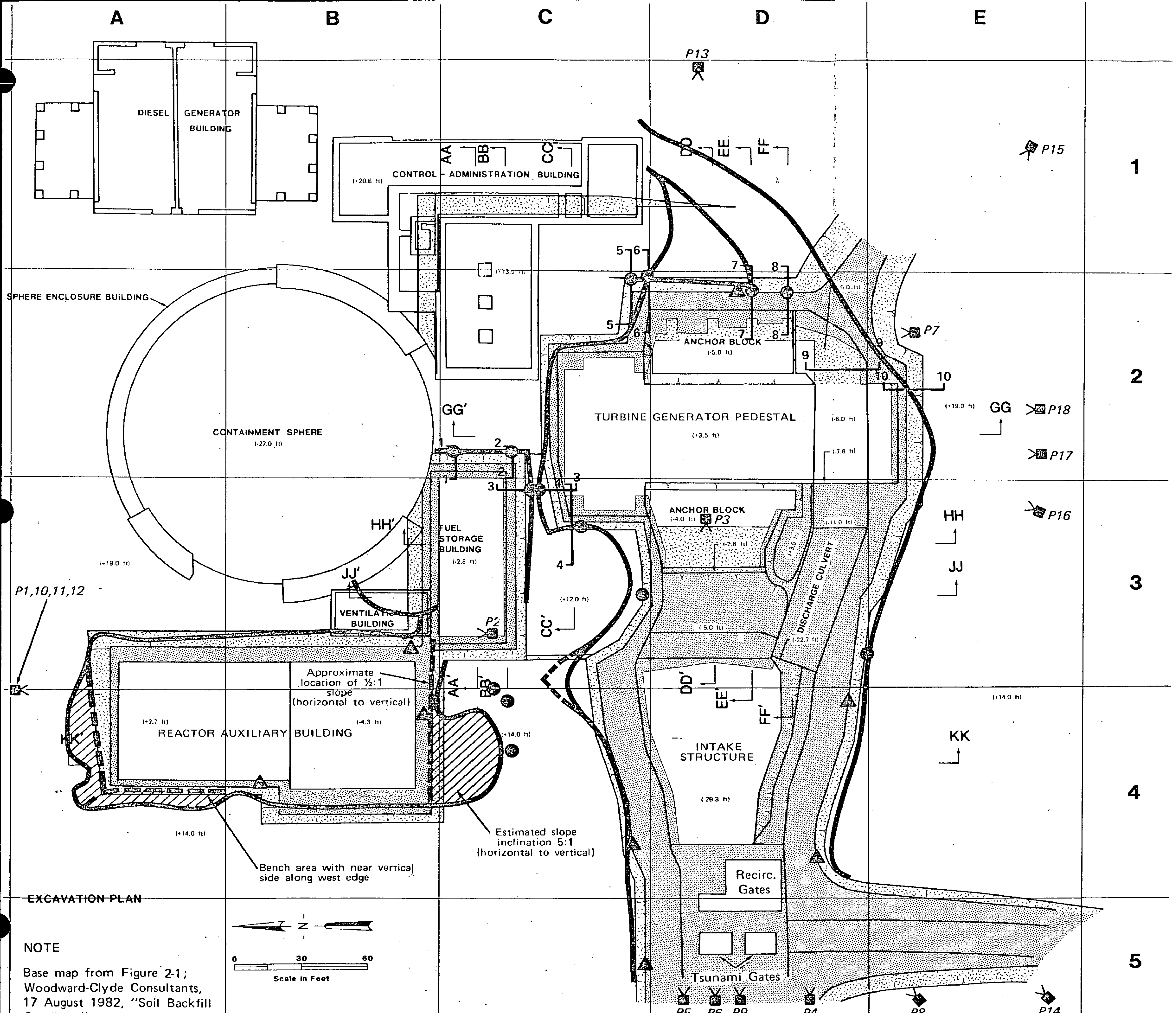
TABLE A-2

CONSTRUCTION PHOTOGRAPHS
SLOPE ANGLE ANALYSIS

Facility	Photo Number	Photo Date	Photo View	Estimated Slope Angle[1]	Figure Number
Reactor Auxiliary Building	P1	6/30/65	South	30°	A-21
Reactor Auxiliary Building	P2	8/28/65	North	24-28°	A-22
Reactor Auxiliary Building	P9	7/30/65	East	25°	A-20
Intake Structure Excavation	P3	10/29/64	West	42°[2] 26°[3]	A-14
Turbine Generator Pedestal and Intake Structure Excavation	P4	8/21/64	East	27°	A-11
Turbine Generator Pedestal and Intake Structure Excavation	P5	11/14/65	East	26°	A-15
Turbine Generator Pedestal Excavation	P6	12/14/64	East	25°	A-9
Turbine Generator Pedestal Excavation	P7	10/14/64	North	24°	A-12
Intake Structure Excavation	P8	10/27/64	Northeast	28°	A-13

Notes:

1. Estimated slope angles are measured from the vertical
2. Upper one-third of slope
3. Lower two-thirds of slope



NOTE

Base map from Figure 2-1; Woodward-Clyde Consultants, 17 August 1982, "Soil Backfill Condition" report.

LEGEND

- Fill above water table
- Fill below water table
- HH'—HH Cross section locations (Figures 2-18 to 2-21, Section 2.0)
- 3 — 3 Location of local soil conditions (Figures 2-11 to 2-17, Section 2.0)
- P15 Photograph number and approximate look direction. Open end of "V" looks in direction of photograph.
- Approximate location of top of cut slopes based on interpretation of construction photographs.
- Approximate location of areas with gentle slope angles.
- Approximate location where apparent slope angles were measured from construction photographs.
- Location of undisturbed San Mateo Sand found in trenches.
- B C D E / 1 2 Reference grid for areas discussed in Appendix A.

PHOTOGRAPH REFERENCE CHART

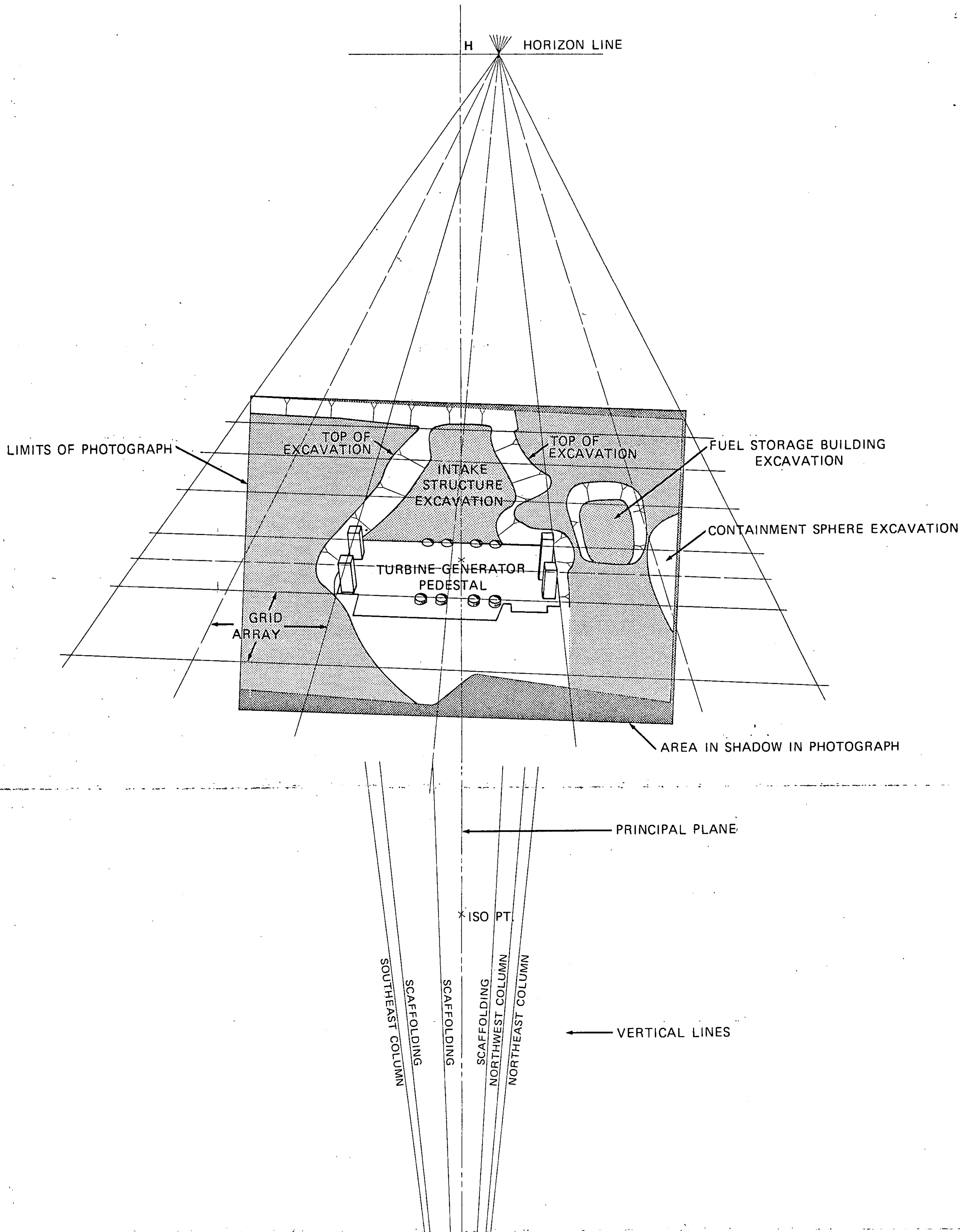
Photograph Number	Figure Number	Photograph Number	Figure Number
1	A-21	10	A-16
2	A-22	11	A-18
3	A-14	12	A-19
4	A-11	13	A-5
5	A-15	14	A-6
6	A-9	15	A-7
7	A-12	16	A-8
8	A-13	17	A-17
9	A-20	18	A-10

Woodward-Clyde Consultants

LIMITS OF THE FOUNDATION / CONSTRUCTION EXCAVATIONS

Project No. 413521
SONGS UNIT 1
SEISMIC RE-EVALUATION

Fig. A-1



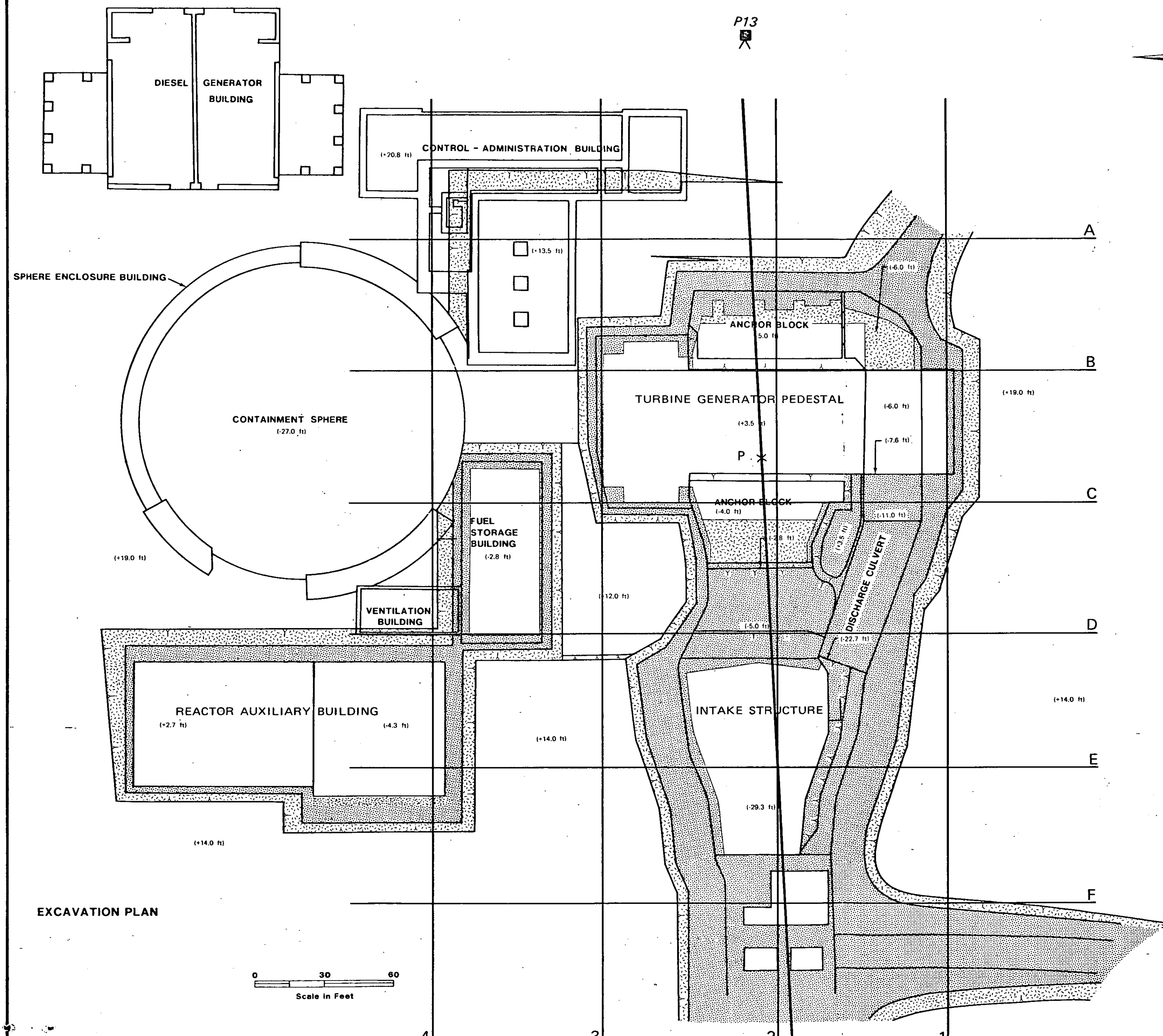
REFERENCE – PHOTOGRAPH P13 (FIGURE A-5)

Woodward-Clyde Consultants

LINE DRAWING OF FIGURE A-5 WITH GRID ARRAY AND VERTICAL LINES PLOTTED

Project No. 413521
SONGS UNIT 1
SEISMIC RE-EVALUATION

Fig. A-2



LEGEND

Fill above water table

Fill below water table

P — Location of principal plane section, x indicates the location of the center of the photograph.

E, F, 2, 1 — Grid array lines used for mapping features from the photograph onto the base map.

P13 — Photograph number and approximate look direction. Open end of "V" looks in direction of photograph.

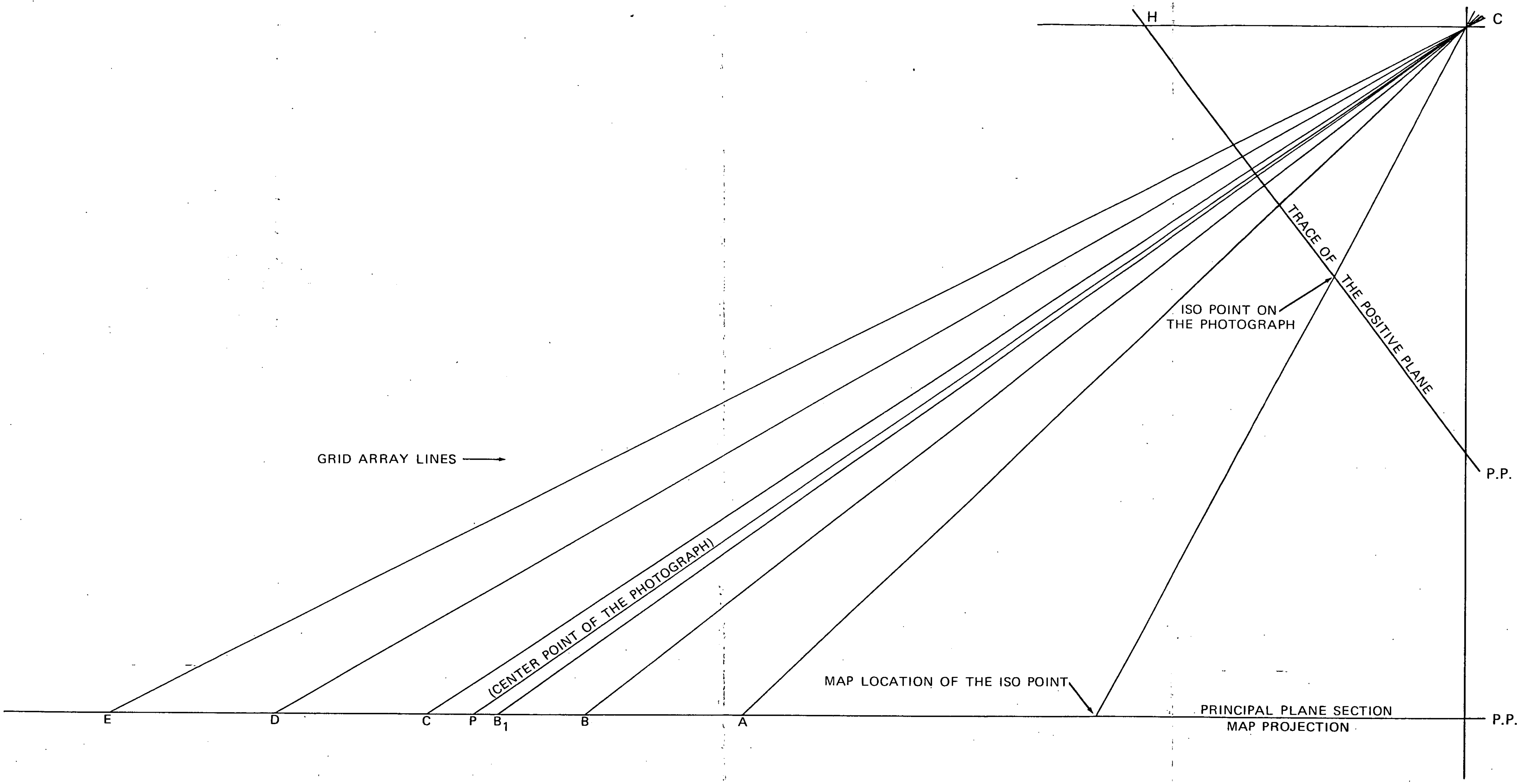
NOTE

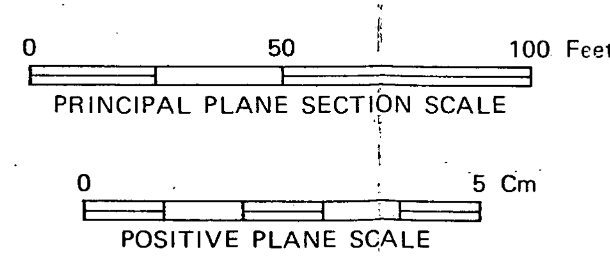
Base map from Figure 2-1; Woodward-Clyde Consultants, 17 August 1982, "Soil Backfill Condition" report.

Woodward-Clyde Consultants

SITE PLAN WITH THE SUPERIMPOSED GRID ARRAY FOR FIGURE A-2

Project No. 413521 SONGS UNIT 1 SEISMIC RE-EVALUATION	Fig. A-3



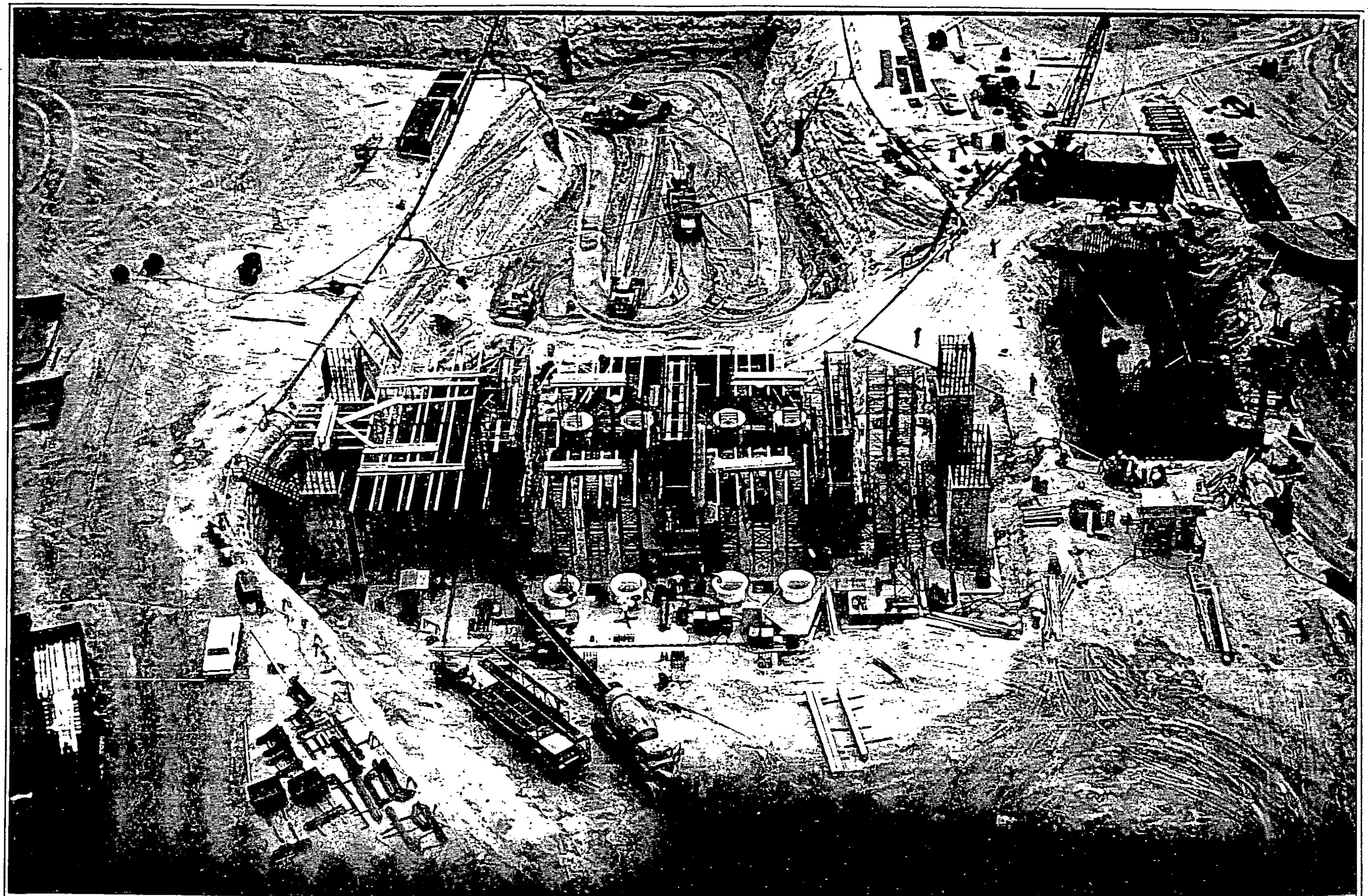


Woodward-Clyde Consultants

PRINCIPAL PLANE SECTION
FOR PHOTOGRAPH P13
(FIGURE A-2)

Project No. 413521
SONGS UNIT 1
SEISMIC RE-EVALUATION

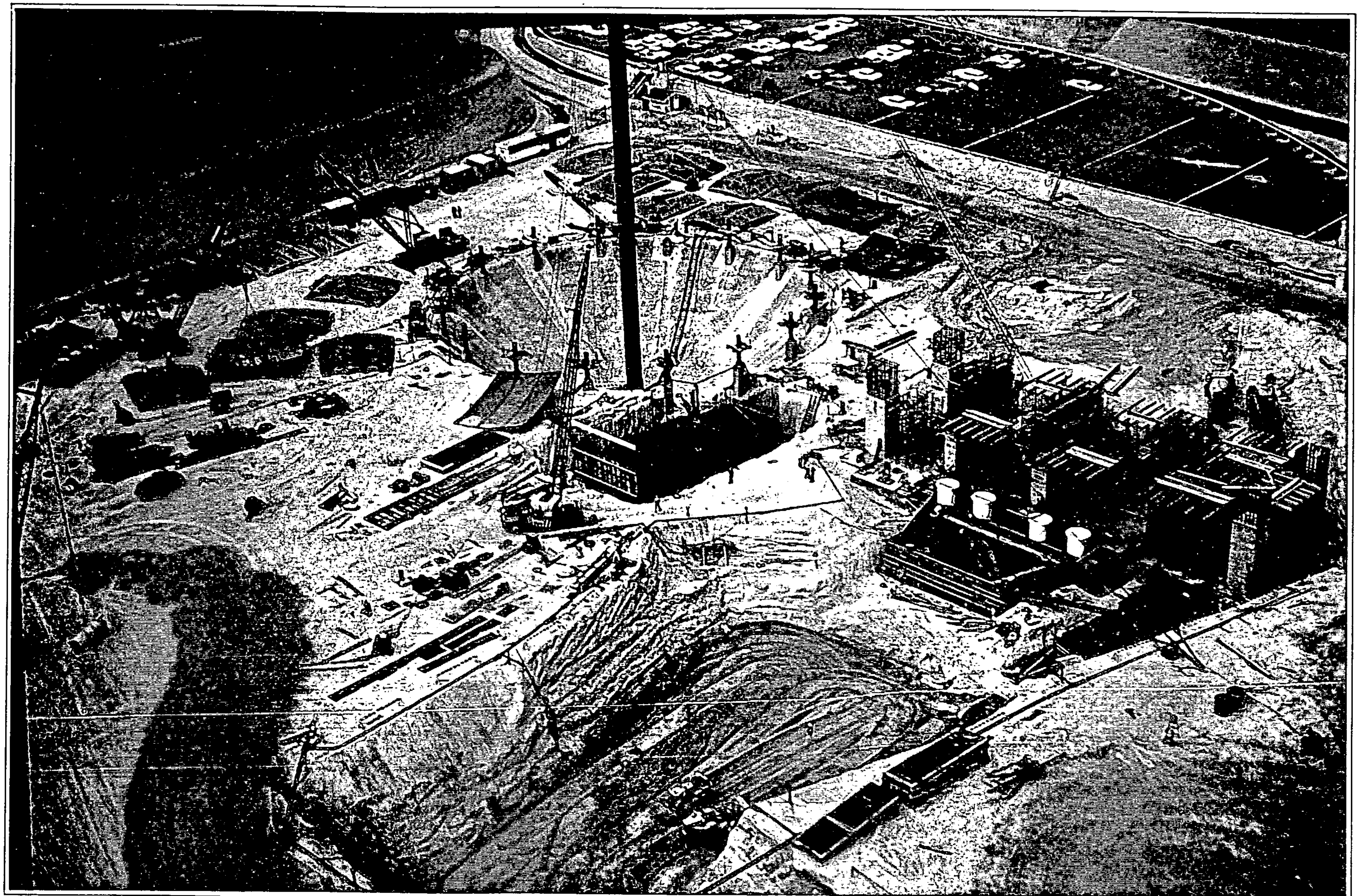
Fig. A-4



PHOTOGRAPH P13 VIEW WEST 10/27/64

Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	TURBINE GENERATOR PEDESTAL AND INTAKE STRUCTURE EXCAVATION	Fig. A-5

WOODWARD–CLYDE CONSULTANTS



PHOTOGRAPH P14 VIEW NORTHEAST 10/27/64

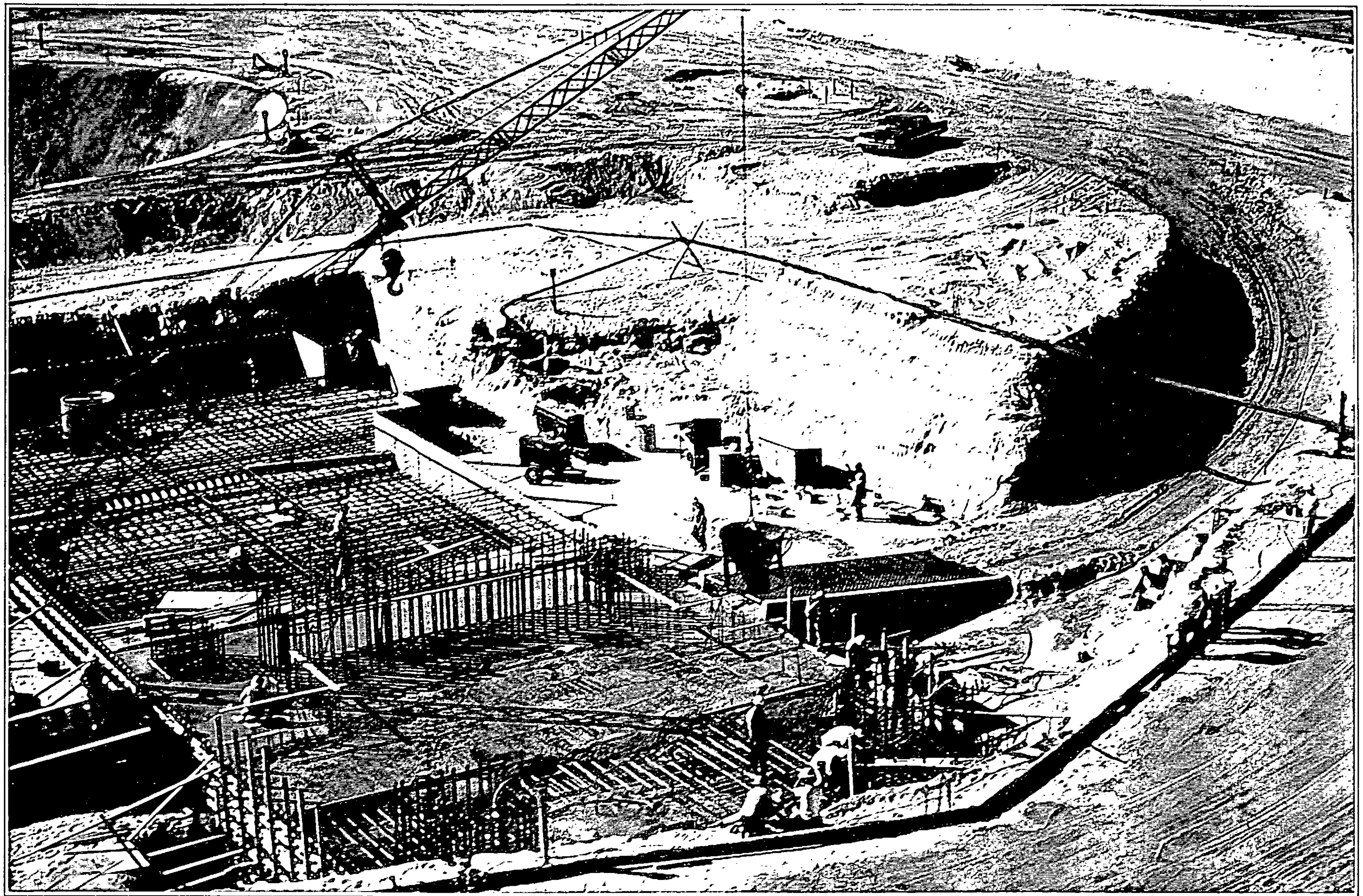
Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	TURBINE GENERATOR PEDESTAL AND INTAKE STRUCTURE EXCAVATION	Fig. A-6

WOODWARD–CLYDE CONSULTANTS

PHOTOGRAPH P15 VIEW NORTHWEST 8/26/64

Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	TURBINE GENERATOR PEDESTAL AND INTAKE STRUCTURE EXCAVATION	Fig. A-7

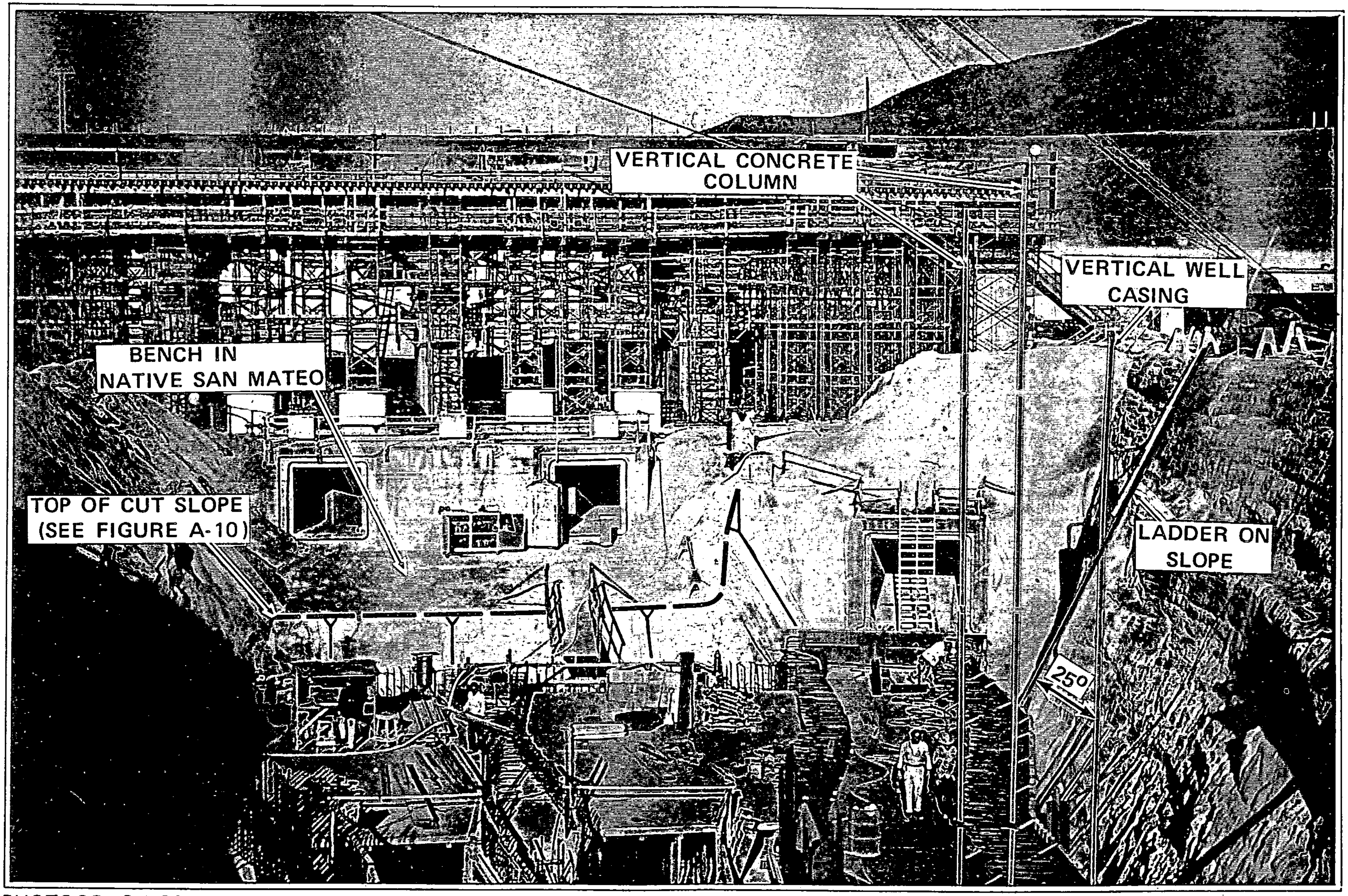
WOODWARD–CLYDE CONSULTANTS



PHOTOGRAPH P16 VIEW NORTHEAST 8/26/64

Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	TURBINE GENERATOR PEDESTAL EXCAVATION	Fig. A-8

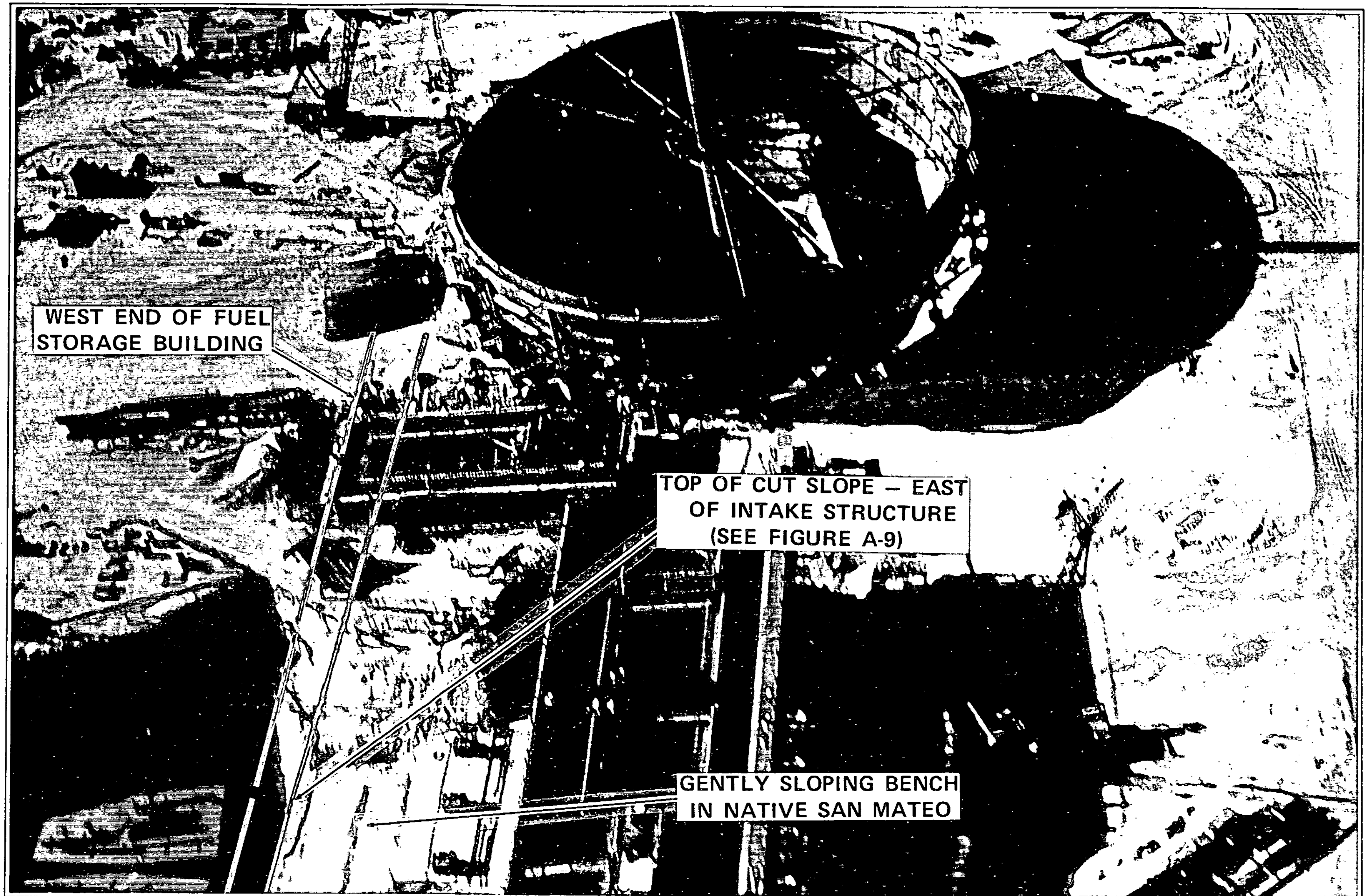
WOODWARD–CLYDE CONSULTANTS



PHOTOGRAPH P6 VIEW EAST 12/14/64

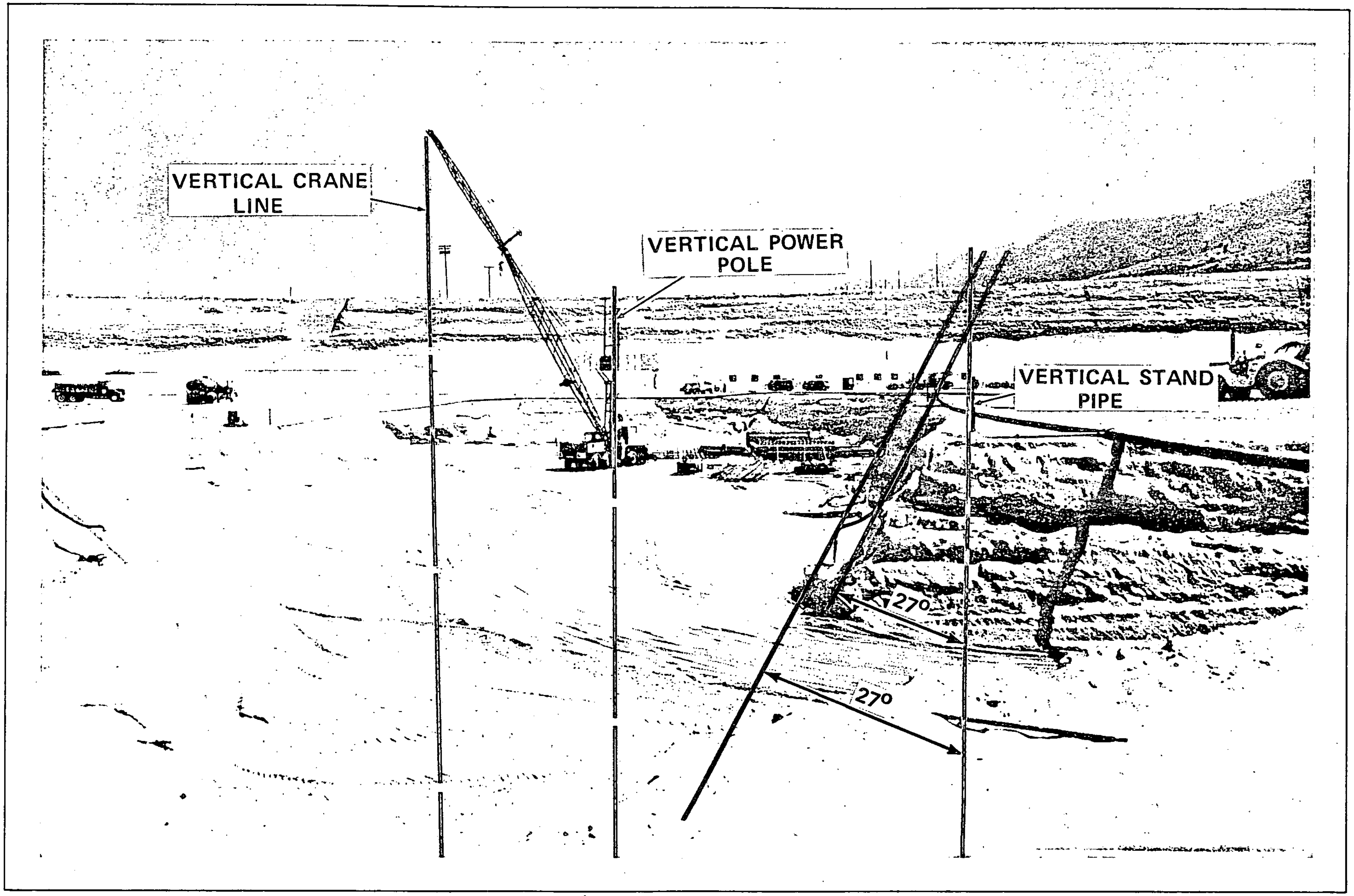
Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	TURBINE GENERATOR PEDESTAL AND INTAKE STRUCTURE EXCAVATION	Fig. A-9

WOODWARD–CLYDE CONSULTANTS



PHOTOGRAPH P18 VIEW NORTH 11/30/64

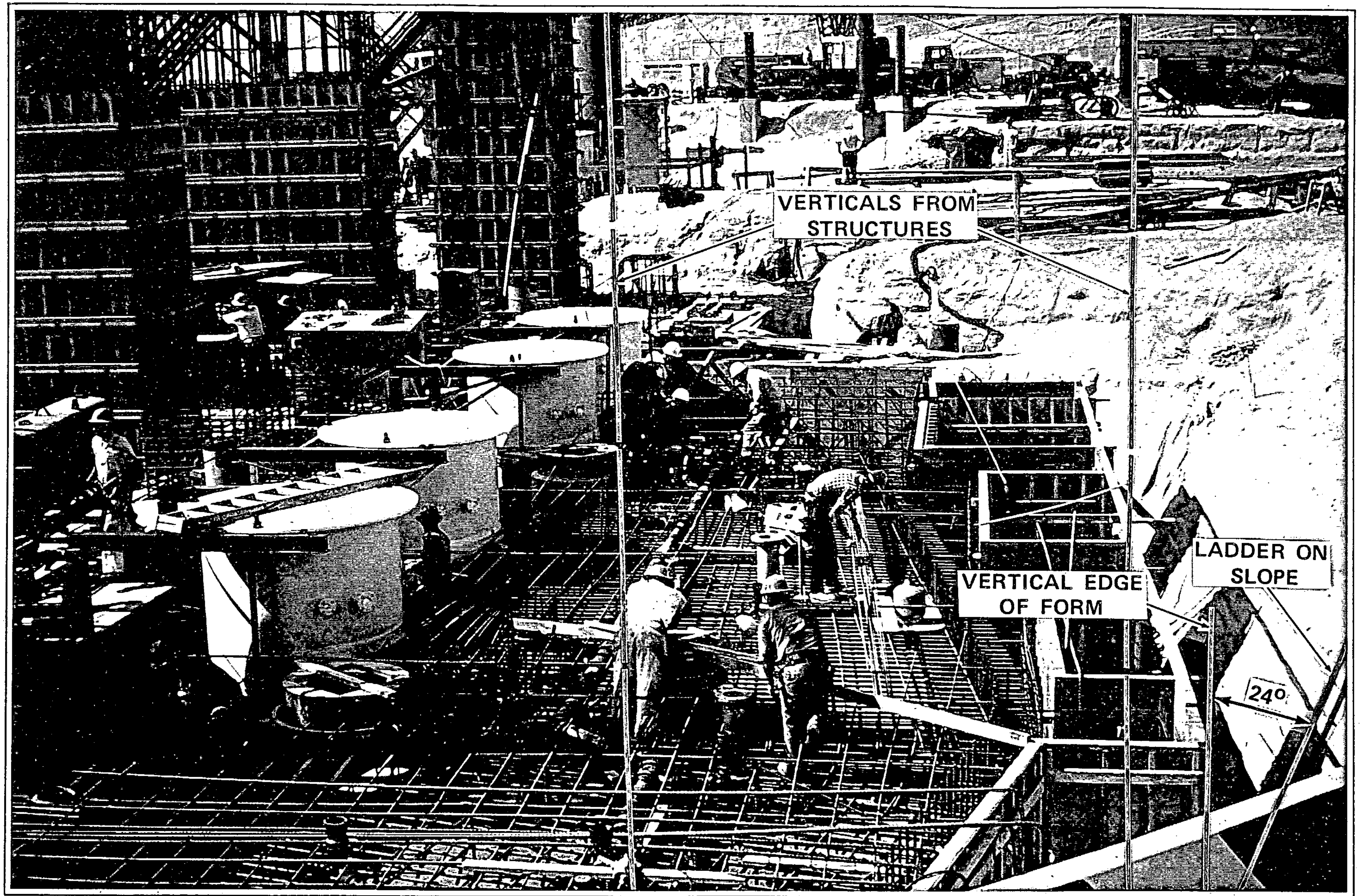
Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 41352I	TURBINE GENERATOR PEDESTAL AND INTAKE STRUCTURE EXCAVATION	Fig. A-10



PHOTOGRAPH P4 VIEW EAST 8/21/64

Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	TURBINE GENERATOR PEDESTAL AND INTAKE STRUCTURE EXCAVATION	Fig. A-11

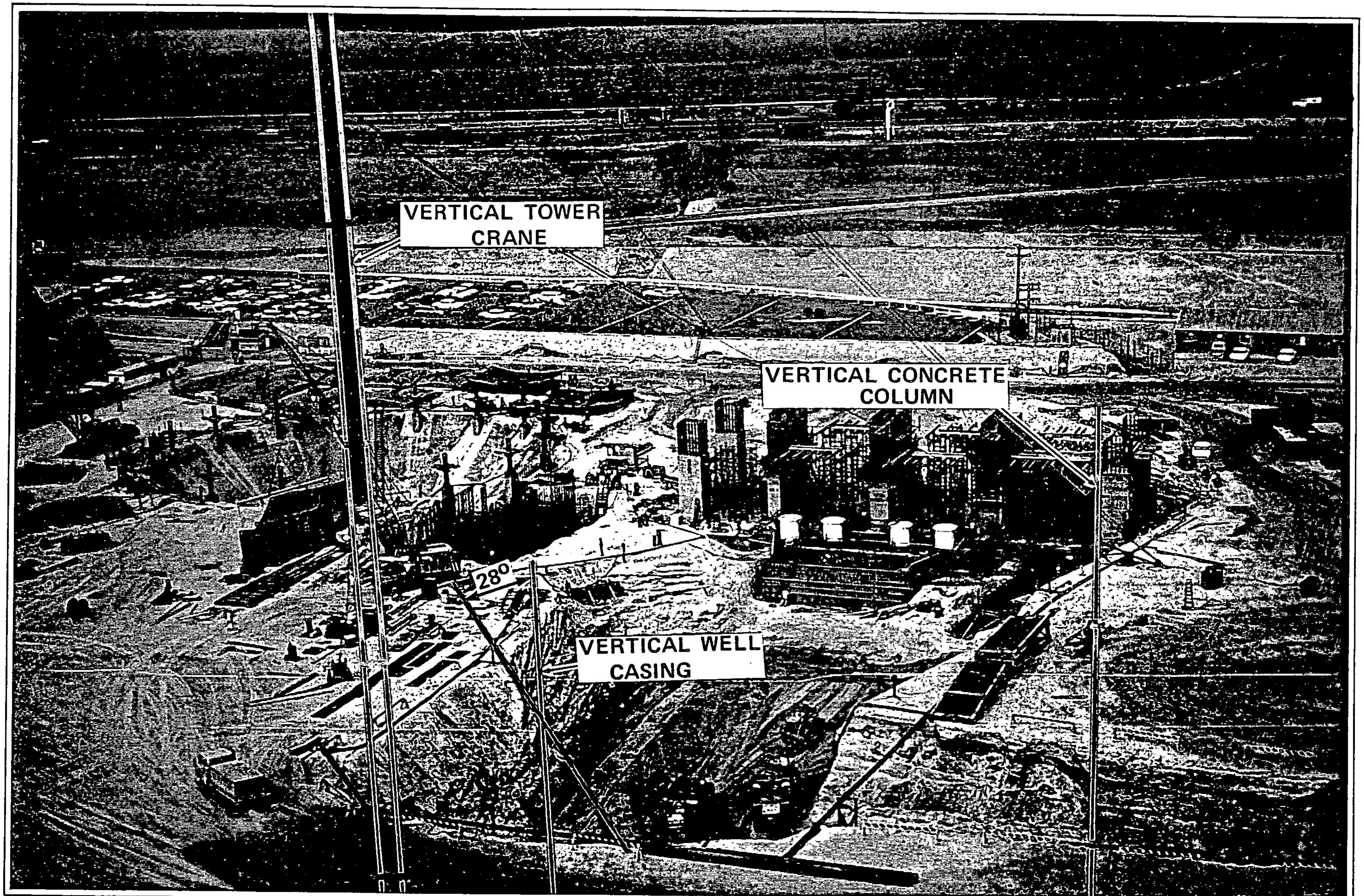
WOODWARD–CLYDE CONSULTANTS



PHOTOGRAPH P7 VIEW NORTH 10/14/64

Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	TURBINE GENERATOR PEDESTAL AND ANCHOR BLOCK EXCAVATION	Fig. A-12

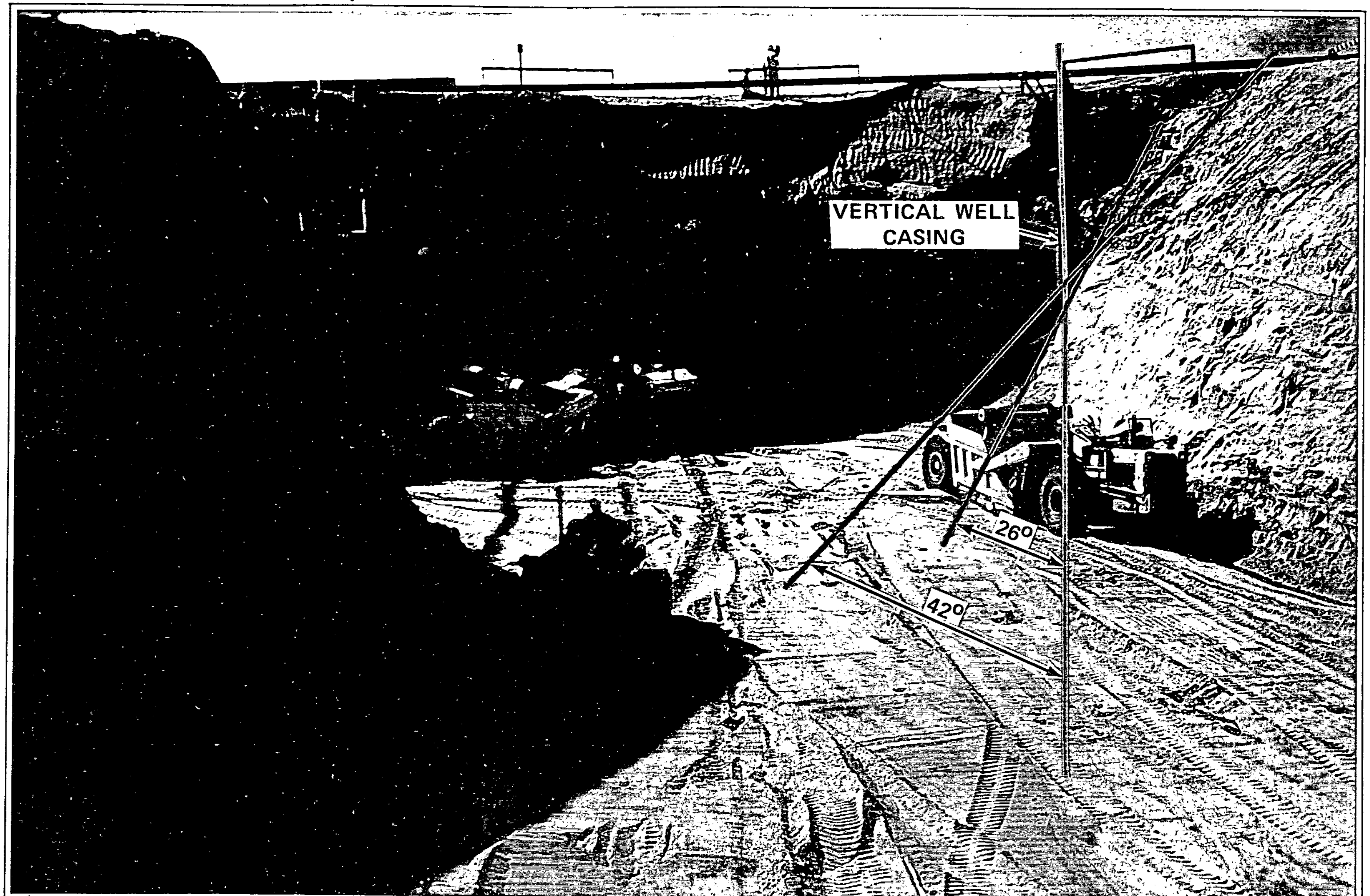
WOODWARD–CLYDE CONSULTANTS



PHOTOGRAPH P8 VIEW NORTHEAST 10/27/64

Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	INTAKE STRUCTURE EXCAVATION	Fig. A-13

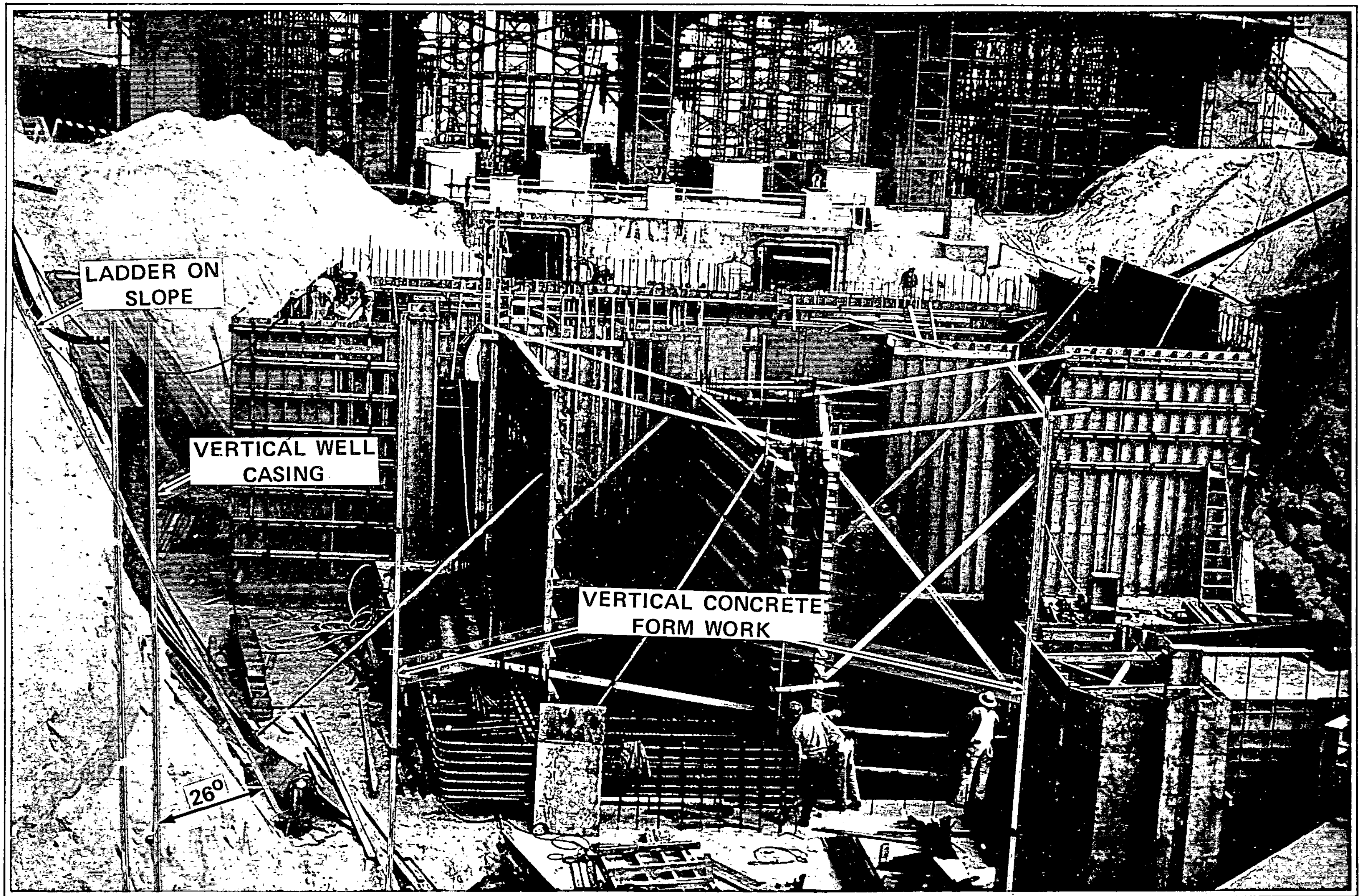
WOODWARD–CLYDE CONSULTANTS



PHOTOGRAPH P3 VIEW WEST 10/29/64

Project: SONGS UNIT 1 SEISMIC RE-EVALUATION **Project No.** 413521	INTAKE STRUCTURE EXCAVATION	Fig. A-14

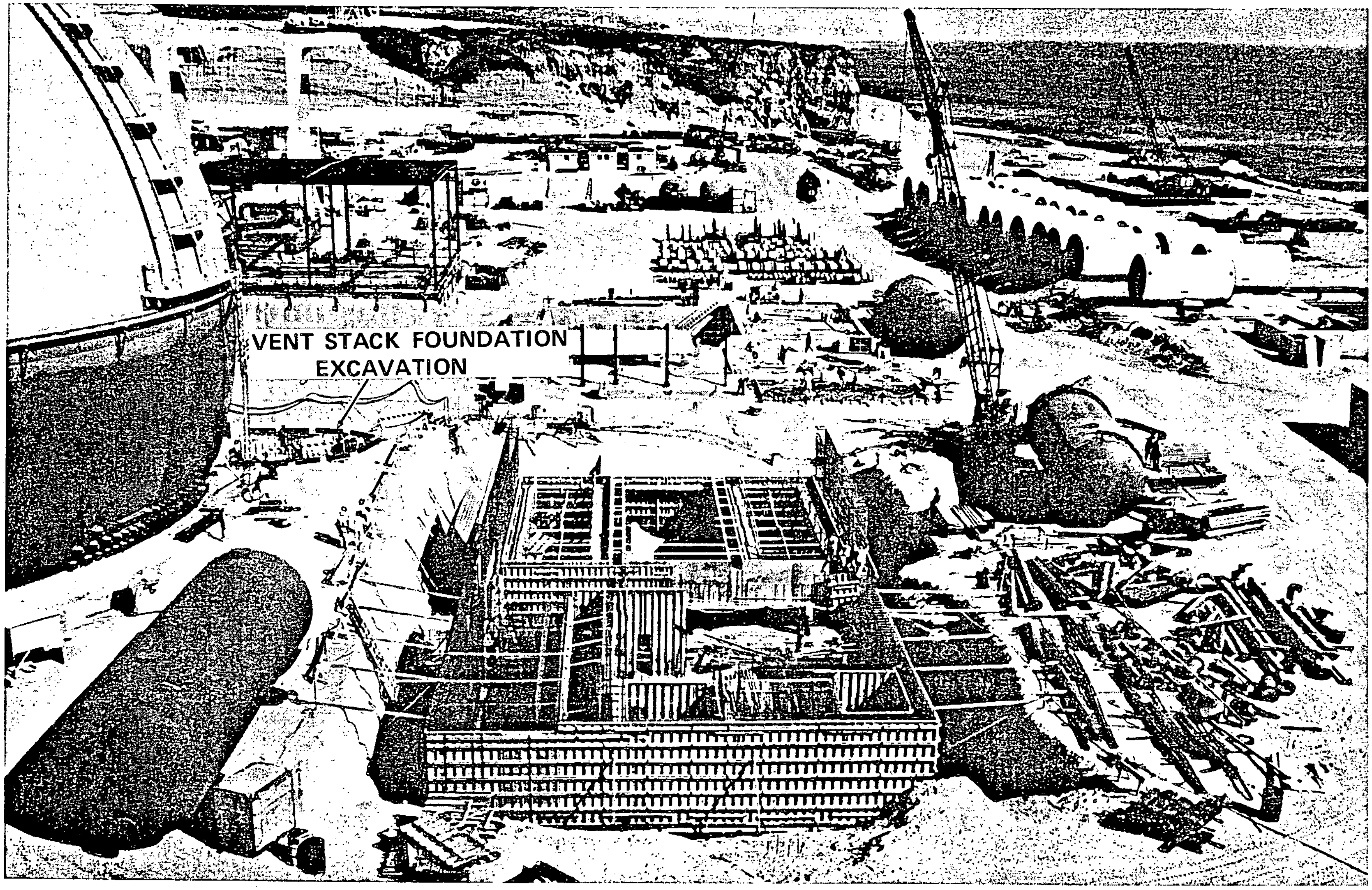
WOODWARD–CLYDE CONSULTANTS



PHOTOGRAPH P5 VIEW EAST 1/14/65

Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	INTAKE STRUCTURE EXCAVATION	Fig. A-15

WOODWARD–CLYDE CONSULTANTS



PHOTOGRAPH P10 VIEW SOUTH 8/23/65

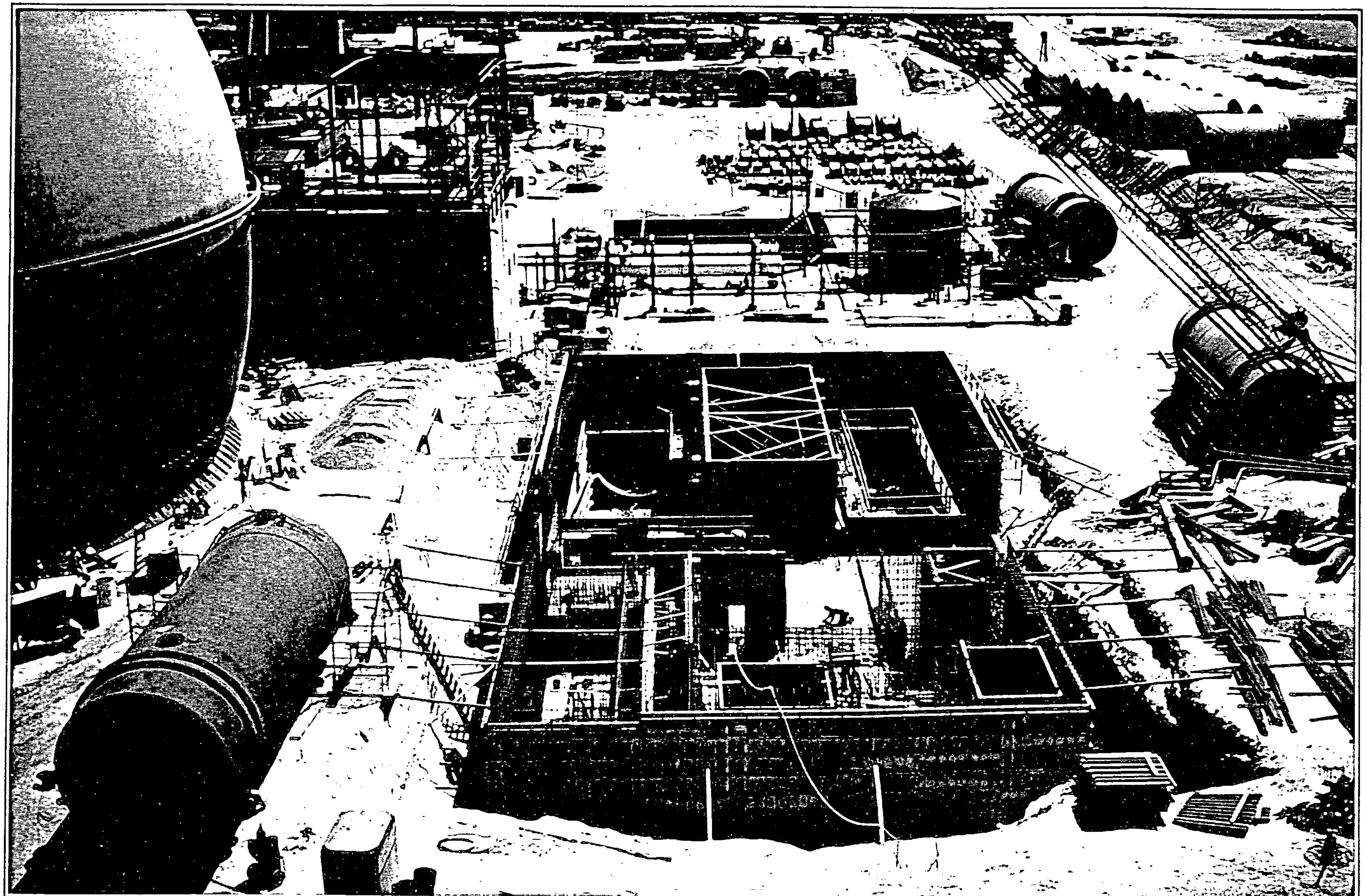
Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	REACTOR AUXILIARY BUILDING	Fig. A-16

WOODWARD–CLYDE CONSULTANTS

PHOTOGRAPH P17 VIEW NORTH 7/30/65

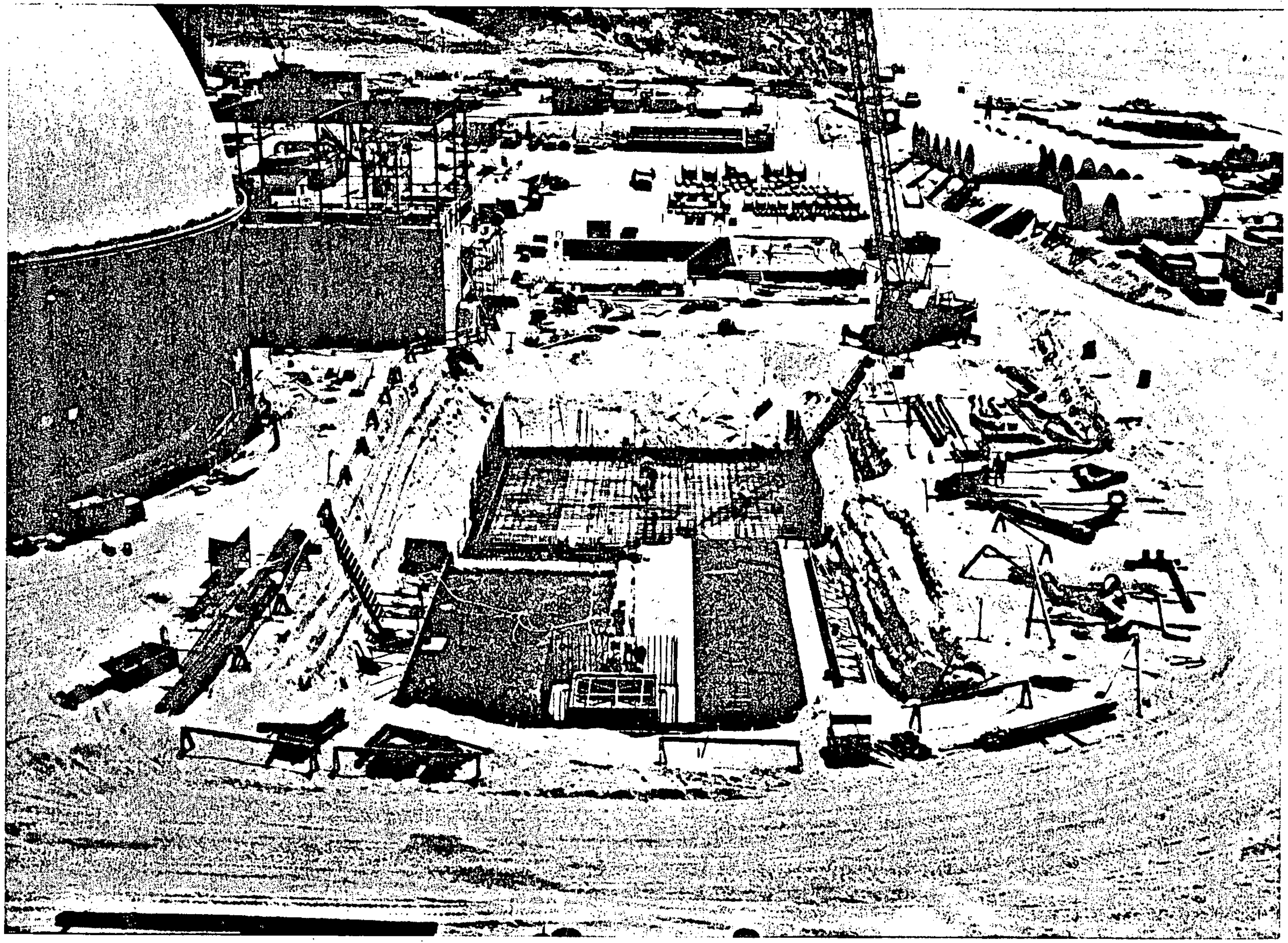
Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 41352I	REACTOR AUXILIARY BUILDING EXCAVATION	Fig. A-17

WOODWARD–CLYDE CONSULTANTS



PHOTOGRAPH P11 VIEW SOUTH 9/3/65

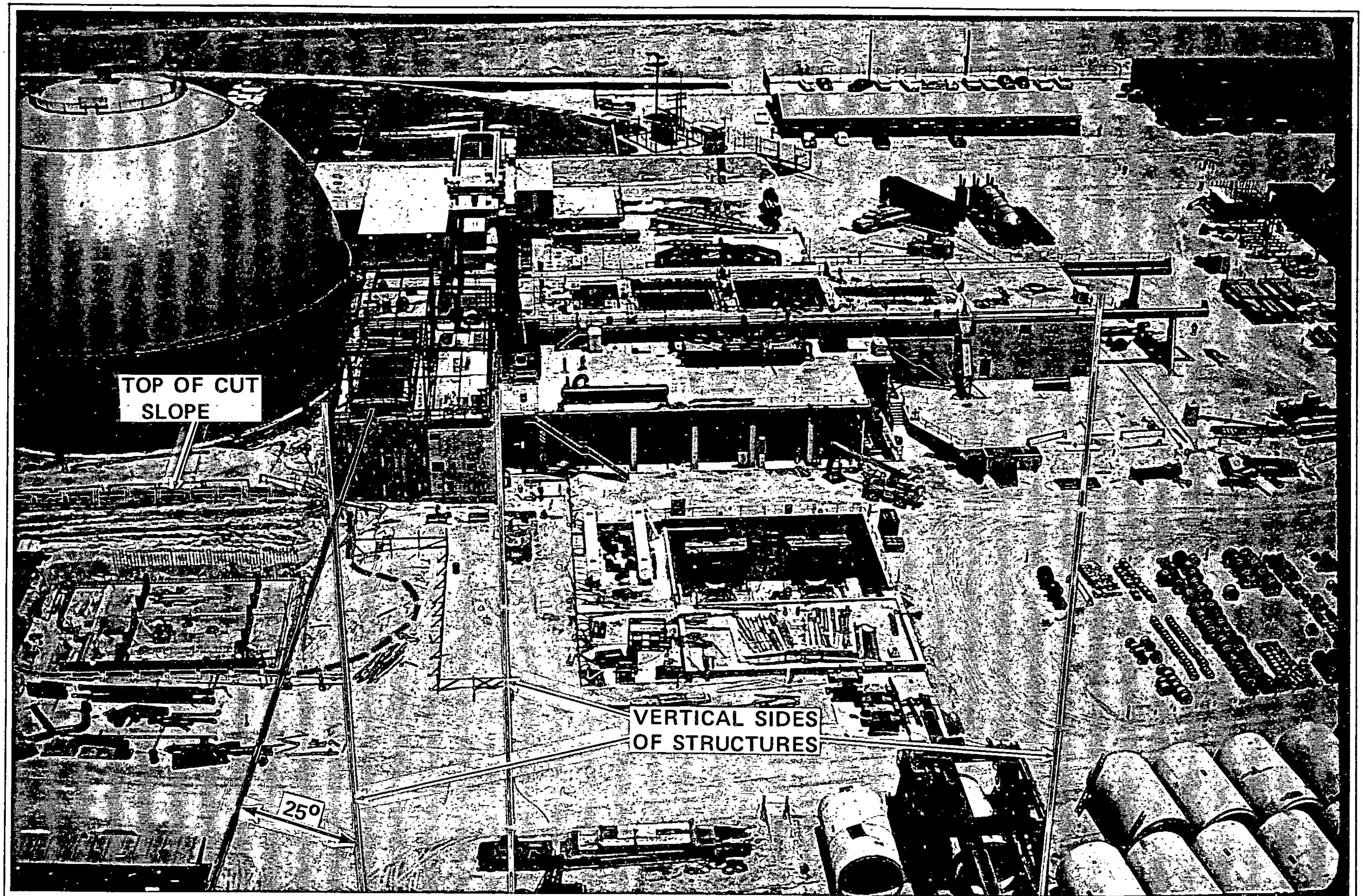
Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	REACTOR AUXILIARY BUILDING	Fig. A-18



PHOTOGRAPH P12 VIEW SOUTH 7/23/65

Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	REACTOR AUXILIARY BUILDING EXCAVATION	Fig. A-19

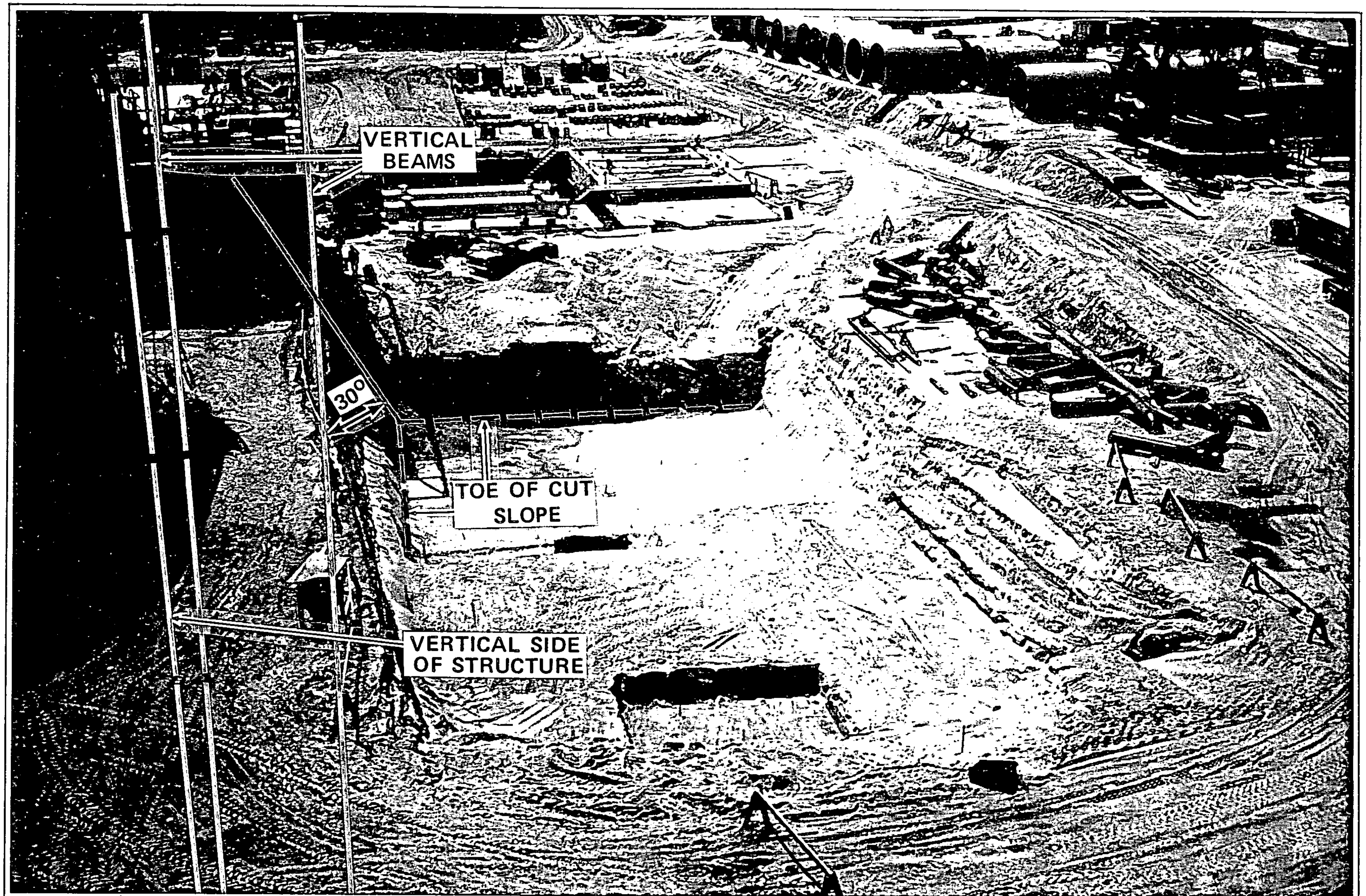
WOODWARD–CLYDE CONSULTANTS



PHOTOGRAPH P9 VIEW EAST 7/30/65

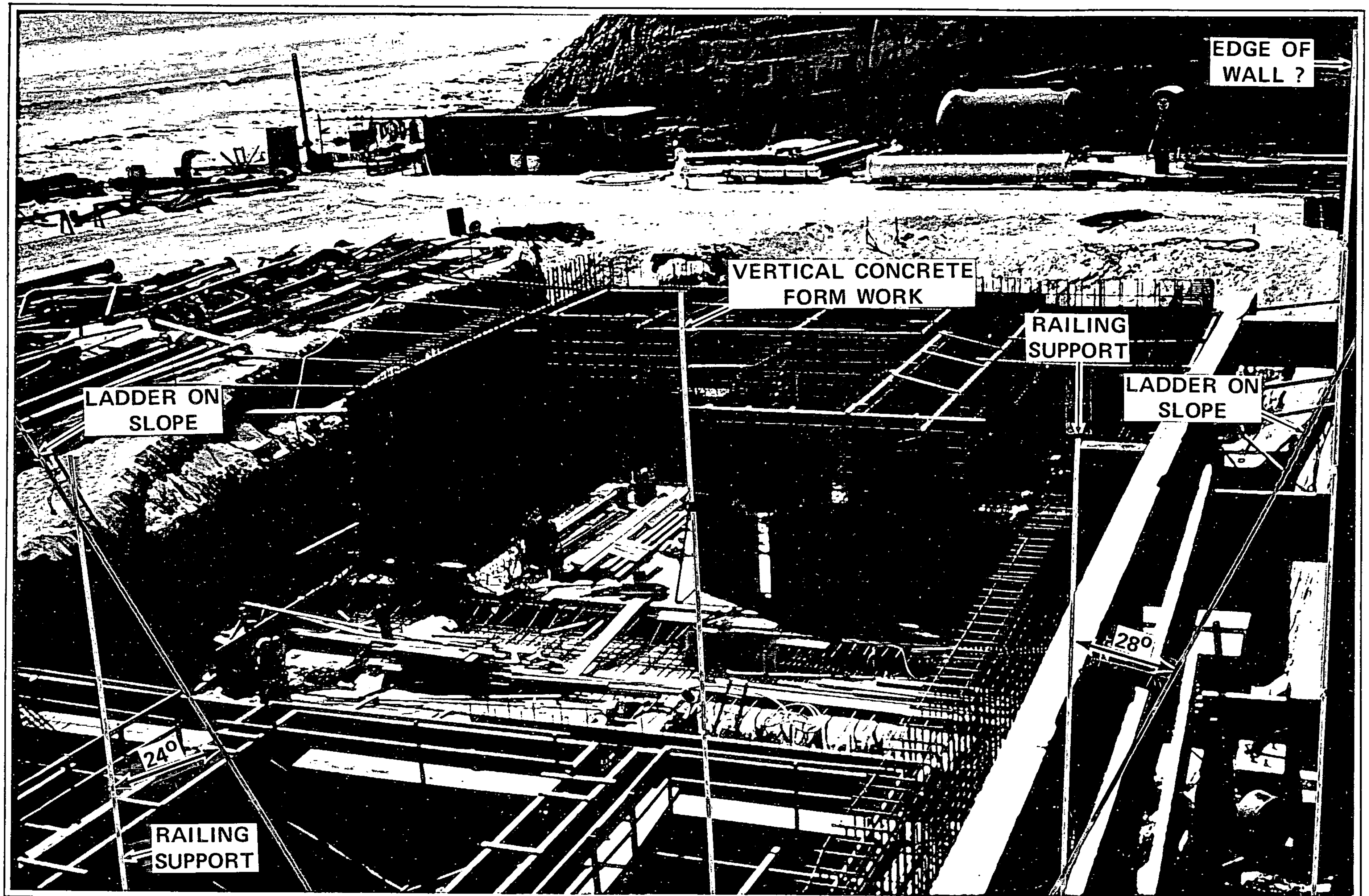
Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	REACTOR AUXILIARY BUILDING	Fig. A-20

WOODWARD–CLYDE CONSULTANTS



PHOTOGRAPH P1 | VIEW SOUTH | 6/30/65

Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	REACTOR AUXILIARY BUILDING	Fig. A-21



PHOTOGRAPH P2 VIEW NORTH 8/28/65

Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	REACTOR AUXILIARY BUILDING	Fig. A-22

WOODWARD–CLYDE CONSULTANTS

APPENDIX B

SUMMARY OF FIELD OBSERVATIONS
AND BACKFILL CHARACTERIZATION

SEISMIC UPGRADE PROGRAM
FOOTING MODIFICATIONS AND SOIL EXCAVATIONS

APPENDIX B

SUMMARY OF FIELD OBSERVATIONS
AND BACKFILL CHARACTERIZATION

SEISMIC UPGRADE PROGRAM
FOOTING MODIFICATIONS AND SOIL EXCAVATIONS

B-1 INTRODUCTION

This appendix presents a description and results of the observations and testing provided by Woodward-Clyde Consultants (WCC) between 5 February 1982 and 21 January 1983. Based on these results, the backfill and soil bearing conditions for the Turbine Building Extensions, the Ventilation Equipment Building, the Reactor Auxiliary Building, the Refueling Water Storage Tank, and the Auxiliary Feedwater Tank were characterized for developing dynamic stiffness parameters for those foundations. The sections that follow describe the observation and testing completed, characterize the foundation bearing soils for each footing, and give a general summary of findings.

B-2 OBSERVATION AND TESTING

An experienced soil technician from WCC observed excavations and backfill placement throughout the construction of the footing modifications. Field work performed by the soil technician was supervised by the project engineer. Laboratory tests were performed in support of field testing, as required.

Areas where these observations and testing were made included the following:

- North and northwest foundations
- West foundation along column line 13
- Southwest foundation along column line K
- Outrigger foundation west of column line 13
- East foundations along column lines F and J
- Northeast foundation along column line D
- Southeast foundation along column line 5
- South foundation along column line P
- Auxiliary Feedwater Tank foundation
- Miscellaneous utility trench backfills in the project area
- Refueling Water Storage Tank, Ventilation Equipment Building and Reactor Auxiliary Building exploratory test pits.

Field density tests were performed in accordance with ASTM Test Method No. D1556-74, and the results are summarized in Table B-1. Figure B-1 shows the location of all foundations, the auxiliary feedwater piping trench and the exploratory test pits. Figures B-2 through B-14 show a plan of each foundation, the Refueling Water Storage Tank and the auxiliary feedwater piping trench along with the test locations. Tests not located in these areas (therefore not located in Figures B-1 through B-14) were generally associated with utility trenches outside the area of interest. All maximum densities were determined by ASTM Test Method No. D1557-A and all relative compaction discussed below are determined relative to the maximum density thus determined.

B-3 CHARACTERIZATION OF BACKFILL CONDITIONS

Results of field density data, summarized in Figures B-2 to B-14 and Table B-1, supplemented with observations and by probing (with a 3-ft long, 3/8-inch diameter steel probe), were used to characterize soil conditions at the bases and sides of excavations made for constructing new foundations or additions to existing foundation. This characterization was made in terms of variation of relative compaction of soil along the perimeter of the footing and in different areas at the base of the footing. A summary of these conditions is presented in Table B-2. These conditions formed the basis for developing stiffness parameters for the various foundations.

B-4 SUMMARY

Test results and observations made in the field by WCC engineers and technicians indicate that the foundations were constructed with the bearing and backfill conditions as indicated in Table B-1. In cases where exposed soils at the base of footings were found not to meet project specifications (i.e., 95 percent relative compaction), the soil was overexcavated to the native soil and replaced with concrete or left in place if the area was very small compared to the total base area of the footing. For some of the footings, medium dense soils were encountered at the base of the foundation excavations, but were left in place. These footings were either supported on top of existing structures, founded on native soil or were structurally connected to foundations (resting on native soil) at one end and supported on native soil at the other end. Based on a careful review of the data with Bechtel engineering personnel, all footing revisions constructed as located in Figure B-1 were found to be satisfactory for their intended use. Furthermore, the results of the observations documented in Table B-2 formed the basis of characterizing soil conditions used in evaluating stiffness parameters for these foundations.

TABLE B-1 - SUMMARY OF FIELD TEST RESULTS

WOODWARD-CLYDE CONSULTANTS

Job Name: Songs 1-Turbine Building Seismic Modification Short Term Outage Project

Field Data Sheet

Sheet No.: 1

Job Number: 41009K

1982 Date	Test Number	Retest by	Retest of	Grid Number	Location of Test	Elev.	Field Dry Density (pcf)	Moist. %	Method	Max. Lab. (pcf)	Rel. Comp %	Spec Reg. %	Drawing No., Spec.	Quality Class
Feb 05	451**			S13+05 W5+80	Electrical Trench	+13'	119	6	S/C	120	99	95		2 *2
Feb 05	452**			S12+50 W5+87	Electrical Trench	+13'	120	6	"	"	100	95		" *2
Mar 04	453**			S10+20 W3+91	East of Sphere Electrical Trench	+18'	115	12	"	"	96	95	5149352	" *2
Mar 05	454**			S8+71 W4+97	Entry Drain Sump 3rd Point	+12'	119	9	"	"	99	95	8570+B	" *2
Mar 08	455**			S8+74 W4+95	Entry Drain Sump 3rd Point	+14'	114	10	"	"	95	95	"	" *2
Mar 10	456			S12+26 W5+30	Turbine K-12 (Fill)	+12'	105	8	"	"	88	95	5166413	" *2
Mar 11	457**	458		S8+65 W5+02	Entry Drain Sump 3rd Point	+15'	117	11	"	"	93	95	8570+B	" *2
Mar 11	458**		457	S8+65 W5+02	Entry Drain Sump 3rd Point	+15'	114	10	"	"	95	95	"	" *2
Mar 11	459			S10+83 W4+92	Turbine A-8 (Fill)	+13'	102	6	"	"	85	95	5166413	" *2
Mar 11	460			S10+79 W4+54	Turbine A-6 (Native)	+13'	120	6	"	"	100	95	"	" *2
Mar 11	461			S10+82 W4+42	Turbine A-8 (Fill)	+12'	101	6	"	"	84	95	"	" *2
Mar 12	462			S12+18 W5+47	Turbine J-13 (Fill)	+13'	111	5	"	"	93	95	"	" *2
Mar 13	463			S10+98 W4+84	Turbine B-8 (Native)	+12'	115	3	"	"	97	95	"	" *2
Mar 13	464			S10+98 W4+88	Turbine B-8 (Fill)	+12'	98	3	"	"	82	95	"	" *2
Mar 13	465			S10+54 W4+93	Turbine A-8 (Fill)	+9'	111	6	"	"	93	95	"	" *2

Remarks: *2 Test requested by Bechtel

** Test outside the Turbine Building Area.

Class 1 & 2 Reviewed By: ____________

Class 3 & 4 Reviewed By: ____________

TABLE B-1 - SUMMARY OF FIELD TEST RESULTS (CONTINUED)

WOODWARD-CLYDE CONSULTANTS

Job Name: Songs 1-Turbine Building
Seismic Modification Short Term Outage Project

Field Data Sheet

Sheet No.: 2
Job Number: 41009K

Date	Test Number	Retest by	Retest of	Grid Number	Location of Test	Elev.	Field Dry Density (pcf)	Moist. %	Method	Max. Lab. (pcf)	Rel. Comp %	Spec Req. %	Drawing No., Spec.	Quality Class
Mar 13	466			S10+91 W4+93	Turbine B-8 (Fill)	+9'	107	5	S/C	120	89	95	5166413	2 *2
Mar 15	467			S10+82 W4+57	Turbine A-6 (Native)	+9'	116	5	"	"	97	95	"	" *2
Mar 15	468			S11+01 W4+66	Turbine B-7 (Native)	+9'	116	4	"	"	97	95	"	" *2
Mar 15	469			S12+29 W5+36	Turbine K-12 (Fill)	+11'	115	5	"	"	95	95	"	" *2
Mar 17	470			S10+88 W4+57	(Trench) Turbine B-6 (Fill)	+9'	118	9	"	"	98	95	"	" *2
Mar 17	471			S11+80 W5+46	Turbine G-13 (Fill)	+9'	107	5	"	"	89	95	"	" *2
Mar 18	472			S11+08 W4+92	(Fill) Turbine Bldg B-8	+9'	109	8	"	"	91	95	"	" *2
Mar 19	473 **			S12+37 W6+76	(Fill) Sump Detector Skids	+12'	116	10	"	"	96	95	9168637	" *2
Mar 19	474			S12+14 W5+59	Footing J-13 (Fill)	+13'	114	9	"	"	95	95	5166413	" *2
Mar 19	475			S10+86 W4+93	(Fill) Turbine Building A&B-8	+8'	104	14	"	"	87	95	"	" *2
Mar 19	476			S10+92 W4+91	(Fill) Turbine Bldg B-8	+8'	101	12	"	"	85	95	"	" *2
Mar 23	477			S11+13 W5+01	(Fill) Turbine Bldg C-9	+12'	102	3	"	"	85	95	"	" *2
Mar 23	478			S11+10 W5+00	(Fill) Turbine Bldg C-9	+10'	104	4	"	"	87	95	"	" *2
Mar 24	479			S12+28 W5+44	(Fill) Turbine Bldg K-13	+7'	117	9	"	"	98	95	"	" *2
Mar 24	480			S11+14 W5+04	(Native) Turbine Bldg C-9	+8'	116	6	"	"	97	95	"	" *2

Remarks: *2 Test requested by Bechtel

** Test outside the Turbine Building Area

Class 1 & 2 Reviewed By: ________

Class 3 & 4 Reviewed By: ________

TABLE B-1 - SUMMARY OF FIELD TEST RESULTS (CONTINUED)

WOODWARD-CLYDE CONSULTANTS

Job Name: Songs 1-Turbine Building Seismic Modification Short Term Outage Project

Field Data Sheet

Sheet No.: 3

Job Number: 41009K

Date	Test Number	Retest by	Retest of	Grid Number	Location of Test	Elev.	Field Dry Density (pcf)	Moist. %	Method	Max. Lab. (pcf)	Rel. Comp %	Spec Reg. %	Drawing No., Spec.	Quality Class	
Mar 25	481			S11+21 W5+05	(Fill) Turbine Bldg C-9	+11'	114	3	S/C	120	95	95	5166413	2	*2
Mar 26	482			S11+26 W5+47	(Native) Turbine Bldg C-13	+9'	119	4	"	"	99	95	"	"	*2
Mar 26	483			S11+41 W5+49	(Fill) Turbine Bldg E-13	+9'	101	4	"	"	85	95	"	"	*2
Mar 27	484			S11+10 W5+02	(Fill) Turbine Bldg C-9 Recomf	+8'	116	6	"	"	97	95	"	"	*2
Mar 30	485**			S12+38 W5+66	Fireline Repair Trench	+12'	117	11	"	"	97	95	"	"	*2
Mar 30	486			S11+29 W5+18	(Fill) Turbine Bldg E-11	+11'	98	5	"	"	82	95	"	"	*2
Mar 30	487			S11+26 W5+26	(Native) Turbine Bldg E-11	+11'	115	3	"	"	96	95	"	"	*2
Mar 30	488**			S14+65 W6+70	south side Ground Cable Trench	+18'	102	7	"	"	85	95	–	"	*2
Apr 01	489			S12+21 W5+52	South (Fill) Turbine Bldg SW K-13	+6'	121	9	"	"	101	95	5166413	"	*2
Apr 01	490			S12+32 W5+44	East (Fill) Turbine Bldg SW K-13	+6'	119	9	"	"	99	95	"	"	*2
Apr 02	491			S12+27 W5+27	East (Fill) Turbine Bldg SW K-12	+7'	100	6	"	"	83	95	"	"	*2
Apr 03	492			S12+29 W5+61	(Fill) Turbine Bldg WW K-13	+9'	110	6	"	"	91	95	"	"	*2
Apr 03	493			S12+27 W5+22	SW Area (Fill) Turbine Bldg K-11	+7'	102	7	"	"	85	95	"	"	*2
Apr 05	494			S11+43 W5+46	West Area (Fill) Turbine Bldg H-13	+11'	107	4	"	"	89	95	"	"	*2
Apr 05	495			S12+24 W5+62	SWW Area (Fill) Turbine Bldg K-13	+7'	114	8	"	"	95	95	"	"	*2

Remarks: *2 Test requested by Bechtel

** Test outside the Turbine Building Area.

Class 1 & 2 Reviewed By: ____________

Class 3 & 4 Reviewed By: ____________

TABLE B-1 - SUMMARY OF FIELD TEST RESULTS (CONTINUED)

WOODWARD-CLYDE CONSULTANTS

Job Name: Songs 1-Turbine Building
Seismic Modification Short Term Outage Project

Field Data Sheet

Sheet No.: 4
Job Number: 41009K

Date	Test Number	Retest by	Retest of	Grid Number	Location of Test	Elev.	Field Dry Density (pcf)	Moist. %	Method	Max. Lab. (pcf)	Rel. Comp %	Spec Req. %	Drawing No., Spec.	Quality Class
Apr 05	496			S12+48 W5+05	S Area (Fill) Turbine Bldg L-9	+9'	103	7	S/C	120	85	95	5166413	2 *2
Apr 05	497			S11+90 W5+65	WW Area (Fill) Outside Turbine Bldg 13	+11'	105	6	"	"	87	95	"	" *2
Apr 05	498			S10+88 W4+95	N Area (Fill) Turbine Bldg B-8	+7'	96	9	"	"	80	95	"	" *2
Apr 06	499			S11+73 W5+62	WW Area (Fill) Outside Turbine Bldg	+8'	105	6	"	"	87	95	"	" *2
Apr 06	500			S10+89 W4+91	N Area (Fill) Turbine Bldg B-8	+5'	98	7	"	"	82	95	"	" *2
Apr 06	501			S11+32 W5+17	NW Area (Fill) Turbine Bldg E-11	+7'	97	11	"	"	81	95	"	" *2
Apr 07	502			S12+68 W5+04	S Project M-9 (Fill)	+11'	108	5	"	"	90	95	"	" *2
Apr 07	503			S11+00 W4+94	N Project (Fill) Turbine Bldg B-8	+6'	100	5	"	"	83	95	"	" *2
Apr 08	504			S11+30 W5+16	NW Area (Native) Turbine Bldg E-11	+5'	116	6	"	"	97	95	"	" *2
Apr 08	505			S11+73 W5+65	WW Project (Fill) Turbine Area G-13+	+9'	111	4	"	"	92	95	"	" *2
Apr 09	506			S11+24 W5+32	NW Project (Native) Turbine Bldg C-11	+7'	116	3	"	"	97	95	"	" *2
Apr 10	507			S11+85 W5+48	W Project (Fill) Turbine Bldg G-13	+9'	115	6	"	"	96	95	"	" *2
Apr 12	508			S12+29 W5+36	SW Project (Native) Turbine Bldg K-12	+3'	116	6	"	"	97	95	"	" *2
Apr 12	509			S11+38 W5+45	W Project (Fill) Turbine Bldg E-13	+10'	101	4	"	"	84	95	"	" *2
Apr 12	510			S11+99 W5+64	Outside Turbine Bldg G-11 WW Project (Fill)	+8'	112	6	"	"	93	95	"	" *2

Remarks: *2 Test requested by Bechtel

Class 1 & 2 Reviewed By: ____________

Class 3 & 4 Reviewed By: ____________

TABLE B-1 - SUMMARY OF FIELD TEST RESULTS (CONTINUED)

WOODWARD-CLYDE CONSULTANTS

Field Data Sheet

Job Name: Songs 1-Turbine Building
Seismic Modification Short Term Outage Project

Sheet No.: 5

Job Number: 41009K

Date	Test Number	Retest by	Retest of	Grid Number	Location of Test	Elev.	Field Dry Density (pcf)	Moist. %	Method	Max. Lab. (pcf)	Rel. Comp %	Spec Req. %	Drawing No., Spec.	Quality Class
Apr 12	511			S11+55 W5+47	W Project (Fill) Turbine Bldg F-13	+9'	119	4	S/C	120	99	95	5166413	2 *2
Apr 13	512			S11+45 W5+50	W Project (Fill & Native) Turbine Bldg E-F&13	+9'	113	3	"	"	94	95	"	" *2
Apr 13	513			S11+83 W5+63	WW Project (Fill) Outside Turbine Bldg	G-H +5'	110	7	"	"	91	95	"	" *2
Apr 13	514			S11+83 W5+61	WW Project (Fill) Turbine Bldg G-H	+3'	111	9	"	"	93	95	"	" *2
Apr 14	515			S12+19 W5+47	SW Project (Fill) Turbine Bldg J-13	+7'	117	6	"	"	98	95	"	" *2
Apr 14	516			S12+31 W5+57	SW Project (Fill) Turbine Bldg K-13	+7'	119	6	"	"	99	95	"	" *2
Apr 14	517			S11+87 W5+62	WW Project (Fill) Outside Turbine Bldg	G-H +3'	110	15	"	"	92	95	"	" *2
Apr 14	518			S11+97 W5+62	WW Project (Fill) Outside Turbine Bldg	G-H +3'	114	8	"	"	95	95	"	" *2
Apr 15	519			S11+85 W5+66	WW Project (Fill) Outside Turbine Bldg	G-H +3'	107	12	"	"	89	95	"	" *2
Apr 15	520			S11+98 W5+66	WW Project (Fill) Outside Turbine Bldg	G-H +3'	109	11	"	"	91	95	"	" *2
Apr 15	521			S12+00 W5+60	WW Project (Fill) Outside Turbine Bldg	G-H +3'	110	11	"	"	91	95	"	" *2
Apr 15	522			S12+28 W5+65	SW Project (Fill) Turbine Bldg K-13	+7'	114	5	"	"	95	95	"	" *2
Apr 16	523			S11+65 W5+42	W Project (Fill) Turbine Bldg F-13	+9'	112	5	"	"	93	95	"	" *2
Apr 16	524			S11+73 W5+50	W Project (Fill) Turbine Bldg F-G&13	+8'	107	6	"	"	89	95	"	" *2
Apr 17	525			S11+74 W5+49	W Project (Fill) Turbine Bldg F-G&13	+7'	106	5	"	"	89	95	"	" *2

Remarks: *2 Test requested by Bechtel

Class 1 & 2 Reviewed By: ____________

Class 3 & 4 Reviewed By: ____________

TABLE B-1 - SUMMARY OF FIELD TEST RESULTS (CONTINUED)

WOODWARD-CLYDE CONSULTANTS

Field Data Sheet

Job Name: Songs 1-Turbine Building
Seismic Modification Short Term Outage Project

Sheet No.: 6
Job Number: 41009K

Date	Test Number	Retest by	Retest of	Grid Number	Location of Test	Elev.	Field Dry Density (pcf)	Moist. %	Method	Max. Lab. (pcf)	Rel. Comp %	Spec Reg. %	Drawing No., Spec.	Quality Class
Apr 17	526			S11+94 W5+47	W Project (Fill) Turbine Bldg H-13	+7'	108	6	S/C	120	90	95	5166413	2 *2
Apr 21	527			S12+06 W5+64	WW Project (Fill) Outrigger Footing	+11'	105	4	"	"	87	95	5166420	" *2
Apr 22	528			S12+03 W5+46	W Project (Fill) Turbine Bldg H-13	+10'	106	5	"	"	88	95	5166413	" *2
Apr 22	529			S12+07 W5+47	W Project (Fill) Turbine Bldg H-13	+9'	112	5	"	"	94	95	"	" *2
Apr 28	530			S12+14 W5+64	WW Project (Fill) Outrigger Footing J	+12'	117	5	"	"	97	95	5166417	" *2
Apr 28	531			S12+53 W6+24	Exploration Trench Elec. Duct Trench	+13'	117	6	"	"	97	95	No Drawing	" *2
Apr 30	532			S12+41 W5+02	S Project (Fill) Turbine Bldg L-9	+7'	112	10	"	"	93	95	5166419	" *2
May 05	533			S11+94 W5+60	WW Project (Fill) Outrigger Footing	+12'	115	5	"	"	96	95	5166420	" *2
May 05	534			S11+73 W5+65	WW Project (Fill) Outrigger Footing	+13'	117	6	"	"	98	95	"	" *2
May 14	535			S12+55 W6+10	Electrical Duct Trench Security (Fill)	+13'	116	9	"	"	96	95	No Drawing	" *2
May 14	536			S12+52 W6+80	Electrical Duct Trench Security (Fill)	+13'	116	8	"	"	96	95	No Drawing	" *2
May 21	537			S12+39 W5+63	Fire Water Trench (Fill)	+13'	120	8	"	"	100	95	5677794	" *2
May 24	538**			S9+30 W2+12	Electrical Duct Com. Trench	+18'	114	6	"	"	95	95	No Drawing	" *2
May 24	539**			S10+20 W2+45	Electrical Duct Com. Trench	+19'	114	7	"	"	95	95	No Drawing	" *2
May 24	540**			S8+50 W1+99	Electrical Duct Com. Trench	+19'	114	6	"	"	95	95	No Drawing	" *2

Remarks: ~~*2 Test requested by Bechtel~~
**Test outside the area as shown in Figure B-1

Class 1 & 2 Reviewed By: ____________

Class 3 & 4 Reviewed By: ____________

TABLE B-1 - SUMMARY OF FIELD TEST RESULTS (CONTINUED)

WOODWARD-CLYDE CONSULTANTS

Job Name: Songs 1-Turbine Building Field Data Sheet Sheet No.: 7
Seismic Modification Short Term Outage Project Job Number: 41009K

Date	Test Number	Retest by	Retest of	Grid Number	Location of Test	Elev.	Field Dry Density (pcf)	Moist. %	Method	Max. Lab. (pcf)	Rel. Comp %	Spec Req. %	Drawing No., Spec.	Quality Class
Jun 09	541**			S12+35 W0+64	Electrical Duct Com. Trench	+50'	115	6	S/C	120	96	95	No Drawing	2 *2
Jun 09	542**			S11+26 W0+66	Electrical Duct Com. Trench	+53'	114	7	"	"	95	95	"	" *2
Jun 15	543**			S7+39 W1+70	Electrical Duct Com. Trench	+28'	108	5	"	"	90	95	"	" *2
Aug 09	544			S12+23 W4+03	(Fill) Foundation "B" Line K-1	+12'	102	4	"	"	85	95	Plot Plan	" *2
Aug 10	545			S12+09 W4+05	(Fill) Foundation "B" Line H-1	+10'	106	4	"	"	88	95	"	" *2
Aug 12	546			S12+20 W4+22	(Fill) Foundation "B" Line J-2	+10'	100	5	"	"	83	95	"	" *2
Aug 15	547			S12+11 W4+25	(Fill) Foundation "B" Line J-2	+8'	108	6	"	"	90	95	"	" *2
Aug 16	548			S12+13 W4+16	(Fill) Foundation "B" Line J-2	+6'	97	14	"	"	81	95	"	" *2
Aug 16	549			S12+10 W4+06	(Native) Foundation "B" Line J-1	+8.5'	115	3	"	"	96	95	"	" *2
Aug 16	550			S11+39 W4+38	(Native) Foundation "A" Line B-3	+12'	122	5	"	"	102	95	"	" *2
Aug 17	551			S11+31 W4+39	(Fill) Foundation "A" Line B-3	+10'	103	5	"	"	85	95	"	" *2
Aug 17	552			S11+33 W4+38	(Fill) Foundation "A" Line B-3	+7'	96	5	"	"	80	95	"	" *2
Aug 19	553			S11+73 W4+00	(Native) Foundation "A" Line C-1	+9'	118	3	"	"	98	95	"	" *2
Aug 20	554			S12+25 W3+99	(Fill) Foundation "B" Line K-1	+8.5'	105	5	"	"	88	95	"	" *2
Aug 20	555			S11+99 W4+04	(Native) Foundation "B" Line H-1	+5.5'	119	13	"	"	99	95	"	" *2

Remarks: *2 Test requested by Bechtel
**Test outside the area as shown in Figure B-1

Class 1 & 2 Reviewed By: ____________

Class 3 & 4 Reviewed By: ____________

TABLE B-1 - SUMMARY OF FIELD TEST RESULTS (CONTINUED)

WOODWARD-CLYDE CONSULTANTS

Job Name: Songs 1-Turbine Building Seismic Modification Short Term Outage Project

Field Data Sheet

Sheet No.: 8

Job Number: 41009K

Date	Test Number	Retest by	Retest of	Grid Number	Location of Test	Elev.	Field Dry Density (pcf)	Moist. %	Method	Max. Lab. (pcf)	Rel. Comp %	Spec Reg. %	Drawing No., Spec.	Quality Class
Aug 20	556			S11+75 W4+22	(Fill) Foundation "A" Line G-2	+10'	96	4	S/C	120	80	95	Plot Plan	2 *2
Aug 20	557			S11+68 W4+05	(Native) Foundation "A" Line F-1	+10'	117	3	"	"	97	95	"	" *2
Aug 20	558			S11+29 W4+05	(Native) Foundation "A" Line E-1	+13'	115	3	"	"	96	95	"	" *2
Aug 21	559			S12+26 W4+01	(Fill) Foundation "B" Line K-1	+6'	109	12	"	"	91	95	"	" *2
Aug 22	560			S11+62 W4+20	(Fill) Foundation "A" Line F-2	+9'	105	4	"	"	88	95	"	" *2
Aug 22	561			S11+69 W4+20	(Fill) Foundation "A" Line F-2	+9'	105	3	"	"	88	95	"	" *2
Aug 23	562			S11+57 W4+13	(Native) Foundation "A" Line F-2	+9'	120	3	"	"	100	95	"	" *2
Aug 24	563			S11+33 W4+37	(Native) Foundation "A" Line E-3	+3'	113	13	"	"	95	95	"	" *2
Aug 24	564			S11+68 W4+17	(Fill) Foundation "A" Line F-2	+8'	103	5	"	"	86	95	"	" *2
Aug 24	565			S12+25 W4+05	(Native) Foundation "B" Line K-1	+4'	120	4	"	"	100	95	"	" *2
Aug 24	566			S11+66 W4+27	(Native) Foundation "A" Line F-2	+5'	119	4	"	"	99	95	"	" *2
Aug 25	567			S12+20 W4+19	(Fill) Foundation "B" Line J-2	+2'	107	14	"	"	89	95	"	" *2
Aug 25	568			S12+08 W4+16	(Fill) Foundation "B" Line J-2	+2'	107	13	"	"	89	95	"	" *2
Aug 25	569				AWS Building East Line 3	+22.5'	119	7	"	"	99	95	"	" *2
Aug 25	570				AWS Building East Line 7	+22.5'	119	6	"	"	99	95	"	" *2

Remarks: *2 Test requested by Bechtel

Class 1 & 2 Reviewed By: ______

Class 3 & 4 Reviewed By: ______

TABLE B-1 - SUMMARY OF FIELD TEST RESULTS (CONTINUED)

WOODWARD-CLYDE CONSULTANTS

Job Name: Songs 1-Turbine Building
Seismic Modification Short Term Outage Project

Field Data Sheet

Sheet No.: 9
Job Number: 41009K

Date	Test Number	Retest by	Retest of	Grid Number	Location of Test	Elev.	Field Dry Density (pcf)	Moist. %	Method	Max. Lab. (pcf)	Rel. Comp %	Spec Reg. %	Drawing No., Spec.	Quality Class
Aug 27	571				AWS Building (Fill) West Line 2.5	+22.5'	116	3	S/C	120	96	95	Plot Plan	2 *2
Aug 27	572			S11+69 W4+30	(Native) Foundation "A" Line F-2	+6'	116	3	"	"	97	95	"	" *2
Aug 28	573			S10+74 W6+20	(Fill) Refueling Water Tank	+12'	113	4	"	"	94	95	"	" *2
Aug 28	574			S10+81 W6+27	(Fill) Refueling Water Tank	+12'	113	4	"	"	94	95	"	" *2
Aug 28	575			S10+90 W6+29	(Fill) Refueling Water Tank	+12'	110	3	"	"	92	95	"	" *2
Aug 28	576			S11+05 W6+21	(Native) Refueling Water Tank	+12'	118	3	"	"	99	95	"	" *2
Aug 28	577			S11+04 W5+99	(Native) Refueling Water Tank	+12'	115	2	"	"	96	95	"	" *2
Aug 28	578			S10+97 W5+94	(Native) Refueling Water Tank	+12'	114	3	"	"	95	95	"	" *2
Aug 28	579			S13+16 W4+79	(Fill) Foundation "D"	+17'	102	3	"	"	85	95	"	" *2
Aug 28	580			S13+17 W4+64	(Native) Foundation "D"	+17'	120	2	"	"	100	95	"	" *2
Sep 02	581			S11+73 W4+24	(Fill) Foundation "A" Line G-2	+9'	103	3	"	"	86	95	"	" *2
Sep 04	582			S11+58 W4+06	(Native) Foundation "A" Line F-1	+6'	121	4	"	"	101	95	"	" *2
Sep 08	583			S11+36 W4+04	(Native) Foundation "A" Line E-1	+11'	119	3	"	"	99	95	"	" *2
Sep 10	584			S11+58 W3+96	(Native) Foundation "A" Line F-1	+5'	115	3	"	"	96	95	"	" *2
Sep 10	585			S11+43 W4+03	(Native) Foundation "A" Line E-1	+7'	118	3	"	"	98	95	"	" *2

Remarks: *2 Test requested by Bechtel

Class 1 & 2 Reviewed By: ________

Class 3 & 4 Reviewed By: ________

TABLE B-1 - SUMMARY OF FIELD TEST RESULTS (CONTINUED)

WOODWARD-CLYDE CONSULTANTS

Job Name: Songs 1-Turbine Building
Seismic Modification Short Term Outage Project

Field Data Sheet

Sheet No.: 10
Job Number: 41009K

Date	Test Number	Retest by	Retest of	Grid Number	Location of Test	Elev.	Field Dry Density (pcf)	Moist. %	Method	Max. Lab. (pcf)	Rel. Comp %	Spec Req. %	Drawing No., Spec.	Quality Class
Sep 10	586			S11+65 W4+00	(Native) Foundation "A" Line F-1	+6'	116	3	S/C	120	96	95	Plot Plan	2 *2
Sep 10	587			S11+65 W3+97	(Native) Foundation "A" Line F-1	+6'	115	2	"	"	96	95	"	" *2
Sep 11	588			S11+64 W4+07	(Native) Foundation "A" Line F-1	+6'	119	3	"	"	99	95	"	" *2
Sep 12	589			S11+42 W3+97	(Native) Foundation "A" Line E-1	+6'	119	3	"	"	100	95	"	" *2
Sep 12	590			S11+76 W4+04	(Native) Foundation "A" Line G-1	+6'	115	3	"	"	96	95	"	" *2
Sep 12	591			S11+33 W4+03	(Native) Foundation "A" Line E-1	+5'	115	3	"	"	96	95	"	" *2
Sep 13	592			S12+08 W4+07	(Native) Foundation "B" Line J-1	+5'	116	3	"	"	96	95	"	" *2
Sep 13	593			S11+54 W4+02	(Native) Foundation "A" Line F-1	+6'	117	3	"	"	97	95	"	" *2
Sep 14	594			S12+20 W4+06	(Native) Foundation "B" Line J-1	+5'	123	3	"	"	103	95	"	" *2
Sep 17	595			S14+40 W5+73	(Fill) Aux. Feedwater Tank	+16'	119	4	"	"	99	95	"	" *2
Sep 17	596			S14+40 W5+56	(Native) Aux. Feedwater Tank	+17'	121	2	"	"	101	95	"	" *2
Sep 19	597			S14+55 W5+58	(Native) Aux. Feedwater Tank	+15'	118	2	"	"	99	95	"	" *2
Sep 24	598			S12+47 W4+47	(Fill) Foundation "C" Line I-5	+17'	104	4	"	"	86	95	"	" *2
Sep 25	599			S14+43 W5+28	(Fill) Septic Tank Sewer	+12'	103	5	"	"	86	95	"	" *2
Sep 29	600			S12+62 W4+48	(Fill) Foundation "C" Center	+16'	104	4	"	"	86	95	"	" *2

Remarks: *2 Test requested by Bechtel

Class 1 & 2 Reviewed By: ____________

Class 3 & 4 Reviewed By: ____________

TABLE B-1 - SUMMARY OF FIELD TEST RESULTS (CONTINUED)

WOODWARD-CLYDE CONSULTANTS

Job Name: Songs 1-Turbine Building Field Data Sheet Sheet No.: 11

Seismic Modification Short Term Outage Project Job Number: 41009K

Date	Test Number	Retest by	Retest of	Grid Number	Location of Test	Elev.	Field Dry Density (pcf)	Moist. %	Method	Max. Lab. (pcf)	Rel. Comp %	Spec Req. %	Drawing No., Spec.	Quality Class
Sep 30	601			S12+71 W4+45	(Fill) Foundation "C" South	+14'	102	5	S/C	120	85	95	Plot Plan	2 *2
Sep 30	602			S13+43 W5+75	(Native) Piping Trench	+14'	117	5	"	"	97	95	"	" *2
Sep 30	603			S12+94 W5+75	(Native) Piping Trench	+11'	118	3	"	"	99	95	"	" *2
Oct 01	604			S12+54 W4+41	(Fill) Foundation "C"	+10'	106	4	"	"	88	95	"	" *2
Oct 01	605			S12+57 W4+47	(Fill) Foundation "C"	+12'	106	6	"	"	88	95	"	" *2
Oct 02	606			S12+48 W5+76	(Fill) Piping Trench	+12'	116	4	"	"	97	95	"	" *2
Oct 02	607			S12+65 W4+47	(Fill) Foundation "C"	+8'	105	5	"	"	87	95	"	" *2
Oct 03	608			S12+20 W5+77	(Fill) Piping Trench	+9'	105	6	"	"	87	95	"	" *2
Oct 04	609			S12+68 W4+48	(Fill) Foundation "C" South	+7'	101	7	"	"	85	95	"	" *2
Oct 04	610			S12+70 W4+41	(Fill) Foundation "C" South	+8'	100	5	"	"	84	95	"	" *2
Oct 05	611			S12+46 W5+76	(Fill) Piping Trench	+8'	108	5	"	"	90	95	"	" *2
Oct 06	612			S13+75 W5+71	(Native) Piping Trench	+12'	120	3	"	"	100	95	"	" *2
Oct 07	613			S11+95 W5+75	(Fill) Piping Trench	+8.5'	102	6	"	"	85	95	"	" *2
Oct 07	614			S14+28 W5+73	(Native) Fire Water Line	+8'	124	8	"	"	104	95	"	" *2
Oct 07	615			S14+68 W5+79	(Fill) Fire Water Line	+13'	116	7	"	"	97	95	"	" *2

Remarks: *2 Test requested by Bechtel

Class 1 & 2 Reviewed By: ____________

Class 3 & 4 Reviewed By: ____________

TABLE B-1 - SUMMARY OF FIELD TEST RESULTS (CONTINUED)

WOODWARD-CLYDE CONSULTANTS

Job Name: Songs 1-Turbine Building Field Data Sheet
Seismic Modification Short Term Outage Project

Sheet No.: 12
Job Number: 41009K

Date	Test Number	Retest by	Retest of	Grid Number	Location of Test	Elev.	Field Dry Density (pcf)	Moist. %	Method	Max. Lab. (pcf)	Rel. Comp %	Spec Req. %	Drawing No., Spec.	Quality Class
Oct 08	616			S11+74 W5+76	(Fill) Piping Trench	+9'	108	6	S/C	120	90	95	Plot Plan	2 *2
Oct 10	617			S12+38 W5+76	(Fill) Piping Trench	+9'	105	8	"	"	87	95	"	" *2
Oct 10	618			S14+05 W6+20	(Native) Electrical Duct 12KV	+15'	119	2	"	"	99	95	"	" *2
Oct 11	619	621		S13+60 W5+28	(Fill) Electrical Duct 12KV	+17'	102	3	"	"	85	95	"	" *2
Oct 12	620			S14+13 W7+09	(Fill) Electrical Duct 12KV	+11'	101	2	"	"	84	95	"	" *2
Oct 13	621		619	S13+63 W5+28	(Fill Recompacted) Electrical Duct 12KV	+17'	120	9	"	"	100	95	"	" *2
Oct 13	622			S14+55 W5+36	(Native) Septic Tank Sewer	+7'	121	6	"	"	101	95	"	" *2
Oct 13	623			S14+46 W5+31	(Native) Septic Tank Sewer	+7'	117	6	"	"	98	95	"	" *2
Oct 13	624			S14+53 W5+22	(Native) Septic Tank Sewer	+7'	118	6	"	"	98	95	"	" *2
Oct 14	625			S13+50 W5+06	(Fill) Level Transmitter Trench	+18'	116	9	"	"	97	95	"	" *2
Oct 14	626			S12+05 W5+80	(Fill) Aux. Piping Trench	+8.5'	106	6	"	"	85	95	"	" *2
Oct 14	627			S12+20 W5+80	(Fill) Aux. Piping Trench	+8.5'	99	5	"	"	82	95	"	" *2
Oct 14	628			S12+27 W5+80	(Fill) Aux. Piping Trench	+8.5'	97	6	"	"	81	95	"	" *2
Oct 15	629			S14+52 W5+57	(Fill) Aux. Feedwater Tank	+14.5'	120	6	"	"	100	95	"	" *2
Oct 16	630			S14+28 W5+58	(Native) Aux. Feedwater Tank	+14'	120	2	"	"	100	95	"	" *2

Remarks: *2 Test requested by Bechtel

Class 1 & 2 Reviewed By: ______

Class 3 & 4 Reviewed By: ______

TABLE B-1 - SUMMARY OF FIELD TEST RESULTS (CONTINUED)

WOODWARD-CLYDE CONSULTANTS

Job Name: Songs 1-Turbine Building Seismic Modification Short Term Outage Project

Field Data Sheet

Sheet No.: 13

Job Number: 41009K

Date	Test Number	Retest by	Retest of	Grid Number	Location of Test	Elev.	Field Dry Density (pcf)	Moist. %	Method	Max. Lab. (pcf)	Rel. Comp %	Spec Reg. %	Drawing No., Spec.	Quality Class
Oct 16	631			S14+52 W5+56	(Fill) Aux. Feedwater Tank	+14'	120	5	S/C	120	100	95	Plot Plan	2 *2
Oct 16	632			S14+51 W5+69	(Fill) Aux. Feedwater Tank	+10.5'	120	8	"	"	100	95	"	" *2
Oct 17	633			S14+48 W5+48	(Native) Aux. Feedwater Tank	+10.5'	117	3	"	"	97	95	"	" *2
Oct 17	634			S14+50 W5+70	(Fill) Aux. Feedwater Tank	+10.5'	118	6	"	"	98	95	"	" *2
Oct 17	635			S14+28 W5+67	(Native) Aux. Feedwater Tank	+10.5'	121	5	"	"	101	95	"	" *2
Oct 18	636			S11+81 W5+75	(Fill) Aux. Piping Trench	+9.5'	110	6	"	"	92	95	"	" *2
Oct 18	637			S12+12 W5+75	(Fill) Aux. Piping Trench	+9'	103	5	"	"	86	95	"	" *2
Oct 18	638			S12+33 W5+76	(Fill) Aux. Piping Trench	+9'	107	7	"	"	89	95	"	" *2
Oct 19	639			S14+35 W5+38	(Native) Aux. Feedwater Tank	+10.5'	118	3	"	"	99	95	"	" *2
Oct 19	640			S12+00 W5+75	(Fill) Aux. Piping Trench	+9'	100	4	"	"	83	95	"	" *2
Oct 19	641			S12+28 W5+75	(Fill) Aux. Piping Trench	+9'	105	5	"	"	88	95	"	" *2
Oct 20	642			S14+26 W5+51	(Native) Aux. Feedwater Tank	+10.5'	121	3	"	"	101	95	"	" *2
Oct 22	643			S14+51 W5+65	(Native) Aux. Feedwater Tank	+9'	117	5	"	"	97	95	"	" *2
Oct 24	644			S11+81 W5+74	(Fill) Piping Trench	+6'	104	7	"	"	87	95	"	" *2
Oct 24	645			S11+86 W5+77	(Fill) Piping Trench	+7'	105	8	"	"	87	95	"	" *2

Remarks: *2 Test requested by Bechtel

Class 1 & 2 Reviewed By: ____________

Class 3 & 4 Reviewed By: ____________

TABLE B-1 - SUMMARY OF FIELD TESTS RESULTS (CONTINUED)

WOODWARD-CLYDE CONSULTANTS

Job Name: Songs 1-Turbine Building
Seismic Modification Short Term Outage Project

Field Data Sheet

Sheet No.: 14
Job Number: 41009K

Date	Test Number	Retest by	Retest of	Grid Number	Location of Test	Elev.	Field Dry Density (pcf)	Moist. %	Method	Max. Lab. (pcf)	Rel. Comp %	Spec Reg. %	Drawing No., Spec.	Quality Class
Oct 25	646			S12+05 W5+76	(Fill) Piping Trench	+7.5'	103	6	S/C	120	86	95	Plot Plan	2 *2
Oct 25	647			S14+32 W5+77	(Fill) Aux. Feedwater Tank	+17'	117	4	"	"	97	95	"	" *2
Oct 26	648			S13+91 W5+71	(Native) Piping Trench	+13.5'	120	4	"	"	100	95	"	" *2
Oct 26	649			S14+01 W5+72	(Fill) Piping Trench	+13.5'	116	4	"	"	96	95	"	" *2
Oct 26	650			S14+15 W5+73	(Native) Piping Trench	+14'	119	3	"	"	99	95	"	" *2
Oct 27	651**			S8+43 W2+30	(Fill) Electrical Duct Trench	+24'	114	9	"	"	95	95	"	" *2
Oct 28	652**			S2+62 W3+45	(Native) Reservoir Electrical Duct Trench	+94'	112	1	"	141	86	N/A	"	" *2
Oct 28	653**			S5+03 W2+60	(Native) Reservoir Electrical Duct Trench	+97'	104	8	"	126	82	N/A	"	" *2
Oct 28	654**			S2+31 W4+28	(Native) Reservoir Electrical Duct Trench	+95'	113	4	"	141	85	N/A	"	" *2
Oct 29	655**			S8+40 W2+20	(Fill) Electrical Duct N. Guard Tower	+23'	116	9	"	120	97	N/A	"	" *2
Oct 29	656**			S8+15 W1+87	(Fill) Electrical Duct N. Guard Tower	+24'	115	7	"	"	96	N/A	"	" *2
Oct 31	657			S13+71 W5+08	(Fill) Level Transmitter Trench	+18'	113	5	"	"	94	85	"	" *2
Oct 31	658			S14+64 W5+41	(Fill) Trench South of Aux. Tank	+19'	121	10	"	"	101	95	"	" *2
Nov 02	659			S13+50 W5+06	(Fill) Level Transmitter Trench	+19'	113	6	"	"	94	85	"	" *2
Nov 03	660			S14+35 W5+23	(Fill) Level Transmitter Trench	+18.5'	117	7	"	"	97	85	"	" *2

Remarks: *2 Test requested by Bechtel
**Test outside the area as shown in Figure 1

Class 1 & 2 Reviewed By: ____________

Class 3 & 4 Reviewed By: ____________

TABLE B-1 - SUMMARY OF FIELD TESTS RESULTS (CONTINUED)

WOODWARD-CLYDE CONSULTANTS

Field Data Sheet

Job Name: Songs 1-Turbine Building Seismic Modification Short Term Outage Project

Sheet No.: 15

Job Number: 41009K

Date	Test Number	Retest by	Retest of	Grid Number	Location of Test	Elev.	Field Dry Density (pcf)	Moist. %	Method	Max. Lab. (pcf)	Rel. Comp %	Spec Reg. %	Drawing No., Spec.	Quality Class
Nov 03	661			S14+19 W5+18	(Fill) Level Transmitter Trench	+18'	115	8	S/C	120	96	85	Plot Plan	2 *2
Nov 03	662			S13+99 W5+10	(Fill) Level Transmitter Trench	+18'	106	7	"	"	88	85	"	" *2
Nov 08	663			S14+16 W5+53	(Native) Piping Trench	+14'	120	5	"	"	100	95	"	" *2
Nov 09	664			S11+89 W5+76	(Fill) Piping Trench	+3'	104	16	"	"	87	95	"	" *2
Nov 09	665			S11+73 W5+75	(Fill) Piping Trench	+6'	101	9	"	"	84	95	"	" *2
Nov 17	666			S11+74 W5+64	(Fill) Piping Trench	+10'	108	6	"	"	90	95	"	" *2
Nov 19	667			S14+19 W5+42	(Fill) Level Transmitter Trench	+18'	114	8	"	"	95	95	"	" *2
Nov 29	668			S12+80 W5+72	(Fill) Electrical Trench	+12'	118	8	"	"	98	95	"	" *2
Dec 03	669			S12+81 W5+70	(Fill) Electrical Trench	+13.5'	117	7	"	"	98	95	"	" *2
Dec 03	670			S13+27 W5+71	(Fill) Electrical Trench	+13.5'	118	7	"	"	98	95	"	" *2
Dec 06	671			S11+74 W5+65	(Fill) Piping Trench	+7.0'	105	11	"	"	87	95	"	" *2
Dec 08	672			S8+37 W1+92	(Fill) Fire Water Line 6"	+22'	118	10	"	"	98	95	"	" *2
Dec 09	673	675		S9+90 W2+36	(Fill) Fire Water Line 6"	+21'	111	11	"	"	93	95	"	" *2
Dec 10	674			S10+56 W2+57	(Fill) Fire Water Line 6"	+21'	115	8	"	"	96	95	"	" *2
Dec 10	675		673	S9+92 W2+36	(Fill) Fire Water Line 6"	+21"	119	10	"	"	99	95	"	" *2

Remarks: *2 Test requested by Bechtel

Class 1 & 2 Reviewed By: ____________

Class 3 & 4 Reviewed By: ____________

TABLE B-1 - SUMMARY OF FIELD TESTS RESULTS (CONTINUED)

WOODWARD-CLYDE CONSULTANTS

Field Data Sheet

Job Name: Songs 1-Turbine Building Seismic Modification Short Term Outage Project

Sheet No.: 16

Job Number: 41009K

Date	Test Number	Retest by	Retest of	Grid Number	Location of Test	Elev.	Field Dry Density (pcf)	Moist. %	Method	Max. Lab. (pcf)	Rel. Comp %	Spec Reg. %	Drawing No., Spec.	Quality Class
Dec 13	676			S10+30 W2+47	(Fill) Fire Water Line 6"	+23'	115	8	S/C	120	96	95	Plot Plan	2 *2
Dec 13	677			S9+32 W2+18	(Fill) Fire Water Line 6"	+24'	115	8	"	"	96	95	"	" *2
Dec 17	678			S9+71 W5+72	(Fill) East Side Reactor Aux. Building	+19'	104	5	"	"	86	95	"	" *2
Dec 17	679			S9+70 W5+71	(Fill) East Side Reactor Aux. Building	+18'	110	6	"	"	91	95	"	" *2
Dec 17	680			S9+71 W5+69	(Fill) East Side Reactor Aux. Building	+17'	97	6	"	"	81	95	"	" *2
Dec 17	681			S9+70 W5+70	(Fill) East Side Reactor Aux. Building	+16'	106	6	"	"	88	95	"	" *2
Dec 17	682			S10+60 W5+35	(Fill) South Side Ventilation Stack Area	+18'	102	8	"	"	85	95	"	" *2
Dec 20	683			S10+60 W5+35	(Fill) South Side Ventilation Stack Area	+17'	101	12	"	"	84	95	"	" *2
Dec 20	684			S10+60 W5+35	(Fill) South Side Ventilation Stack Area	+16'	99	10	"	"	83	95	"	" *2
Dec 20	685			S10+60 W5+35	(Fill) South Side Ventilation Stack Area	+15'	99	10	"	"	83	95	"	" *2
Dec 21	686			S11+81 W5+80	(Fill) West Side Piping Trench	+11'	119	10	"	"	99	95	"	" *2
Dec 22	687**			S7+80 W2+64	(Fill) Light Pool Footing	+25'	117	10	"	"	97	95	"	" *2
Dec 23	688			S11+77 W5+80	(Fill) West Side Piping Trench	+12'	116	10	"	"	97	95	"	" *2
Dec 29	689			S12+90 W4+55	(Fill) N-6 Turbine Bldg Ftg. "E"	+19'	103	9	"	"	86	95	"	" *2
Jan 04	690			S12+79 W4+52	(Fill) Line M-6 Turbine Ftg. E	+19"	101	8	"	"	84	95	"	" *2

Remarks: *2 Test requested by Bechtel

** Test outside the area as shown in Figure B-1

Class 1 & 2 Reviewed By: ____________

Class 3 & 4 Reviewed By: ____________

TABLE B-1 - SUMMARY OF FIELD TESTS RESULTS (CONTINUED)

WOODWARD-CLYDE CONSULTANTS

Field Data Sheet

Job Name: Songs 1-Turbine Building Seismic Modification Short Term Outage Project

Sheet No.: 17

Job Number: 41009K

Date	Test Number	Retest by	Retest of	Grid Number	Location of Test	Elev.	Field Dry Density (pcf)	Moist. %	Method	Max. Lab. (pcf)	Rel. Comp %	Spec Req. %	Drawing No., Spec.	Quality Class
Jan 04	691			S12+88 W4+58	(Fill) Line M-6 Turbine Footing E	+18'	108	8	S/C	120	91	95	Plot Plan	2 *2
Jan 05	692			S12+92 W4+50	(Fill) Line M-6 Turbine Footing E	+17'	98	4	"	"	82	95	"	" *2
Jan 05	693			S9+71 W12+84	(Fill) Line MN-6 Turbine Footing E	+15'	96	4	"	"	80	95	"	" *2
Jan 06	694			S12+85 W4+51	(Fill) Line MN-6 Turbine Footing E	+14.5'	96	3	"	"	80	95	"	" *2
Jan 07	695			S12+82 W4+57	(Fill) Line MN-6 Turbine Footing E	+12'	97	3	"	"	81	95	"	" *2
Jan 07	696			S12+88 W4+51	Native Turbine Footing E	+14'	121	3	"	"	101	95	"	" *2
Jan 10	697			S12+76 W4+47	(Fill) M-5 Turbine Footing E	+9.5'	98	4	"	"	82	95	"	" *2
Jan 10	698			S12+99 W4+51	Native N-S&G Turbine Footing E	+14.5'	120	3	"	"	100	95	"	" *2
Jan 12	699			S12+79 W4+57	(Fill) M-6 Turbine Footing E	+8.5'	99	5	"	"	82	95	"	" *2
Jan 14	700			S12+92 W4+97	(Fill) N-8 Turbine Footing "F"	+19'	106	6	"	"	88	95	"	" *2
Jan 17	701			S12+80 W4+97	(Fill) M-8 Turbine Footing "F"	+19'	105	5	"	"	87	95	"	" *2
Jan 18	702			S12+98 W4+42	Native Outrigger Turbine Bldg Ftg. "E"	+16'	121	4	"	"	101	95	"	" *2
Jan 18	703			S12+96 W4+48	(Fill) M-5 Turbine Bldg Ftg. "E"	+17'	100	5	"	"	83	95	"	" *2
Jan 19	704			S12+93 W4+43	Native Outrigger Turbine Bldg Ftg. "E"	+14.5'	120	3	"	"	100	95	"	" *2
Jan 20	705			S12+91 W4+92	(Fill) N-8 Turbine Bldg Ftg. "F"	+17.5'	109	5	"	"	91	95	"	" *2

Remarks: *2 Test requested by Bechtel

Class 1 & 2 Reviewed By: ____________

Class 3 & 4 Reviewed By: ____________

TABLE B-1 - SUMMARY OF FIELD TESTS RESULTS (CONTINUED)

WOODWARD-CLYDE CONSULTANTS

Job Name: Songs 1-Turbine Building Seismic Modification Short Term Outage Project

Field Data Sheet

Sheet No.: 18

Job Number: 41009K

Date	Test Number	Retest by	Retest of	Grid Number	Location of Test	Elev.	Field Dry Density (pcf)	Moist. %	Method	Max. Lab. (pcf)	Rel. Comp %	Spec Reg. %	Drawing No., Spec.	Quality Class
Jan 20	706			S12+82 W4+92	(Fill) M-8 Turbine Bldg. Ftg. "F"	+17.5	103	4	S/C	120	85	95	Plot Plan	2 *2
Jan 20	707			S12+93 W4+99	(Fill) N-6 Turbine Bldg Ftg. "F"	+15.5	102	6	"	"	85	95	"	" *2
Jan 21	708			S12+85 W4+95	(Fill) MN-9 Turbine Bldg Ftg. "F"	+16'	98	5	"	"	82	95	"	" *2
Jan 21	709			S12+76 W4+98	(Fill) M-8 Turbine Bldg Ftg. "F"	+16'	97	4	"	"	81	95	"	" *2
Jan 21	710			S12+88 W4+99	Native MN-8 Turbine Bldg Ftg. "F"	+14.5	120	3	"	"	100	95	"	" *2

Remarks: *2 Test requested by Bechtel

Class 1 & 2 Reviewed By:

Class 3 & 4 Reviewed By:

TABLE B-2

SUMMARY OF SOIL CHARACTERIZATION FOR EXCAVATIONS TESTED

Foundation	Soil Characterization
1. North Footing	As shown in Figure B-1, most of the footing is founded on native soil and the backfill or native soil exposed against the footing sides is dense (minimum 95 percent relative compaction), except for the western portion. In this area the foundation is founded on backfill with relative compaction varying from 80 to 93 percent. Backfill encountered in the west end of the excavation was removed and replaced with concrete extending down to about elevation +1 foot. Backfill below elevation +1 ft was compacted by vibration and probed and was left in place. Native soil was encountered, based on probing and evaluation of construction photos and excavation plans, at about elevation -2.8 feet.
2. Northwest Footings ° Footing E-11	As shown in Figure B-1, most of the footing is founded on native soil except in a small portion along the east wall. The backfill against the walls is dense, equivalent to a relative compaction of 95 percent except for small portions of the east and south walls where backfill having a relative compaction of 80 to 82 percent was encountered and left in place.
° Footing C-9	As shown in Figure B-1, most of the footing is founded on native soil except for a small width near the north wall. The density of backfill against the north end ranges from 85 to 87 percent compaction. The density of backfill against the other footing sides also varies from 85 to 87 percent compaction.

3. West Footing

Footing north of column line F is founded on the native soil while the remaining footing is founded on backfill placed in the intake-culverts area. This fill varies in relative compaction from 90 to 95 percent. The density of backfill against the sides of the footing has an average relative compaction of about 90 percent. The west footing was placed as one continuous foundation from column C13 to column K13. The depth of excavation south of column F13 and north of J13 was extended to the top of the intake culverts. The depth of the footing in that area was increased such that the footing rested on both intake culverts. The footing was designed such that the building loads will be transferred to the intake culvert walls. The footing structurally span over the backfill between the two culverts.

4. Southwest Footing

Northern portion of the footing along column line 13 is founded on backfill with a relative compaction of 98 to 100 percent. Approximately the western half of the remaining footing is founded on backfill with relative compaction of 95 to 99 percent. In the remainder of the footing backfill having a density ranging between 83 and 85 percent was encountered at elevation +7 ft, the planned footing base elevation. The excavation was deepened in this area to approximately elevation +3 where native soil was encountered in most of the area except in a small area (about 4 ft x 6 ft) in the northeast corner. Backfill in that area was left in place. The overexcavated area was backfilled with concrete. The density of backfill against the east side of the footing and half of the north side was found to have a relative compaction of about 90 percent. For the remaining walls the backfill varied in compaction from 90 to 95 percent.

5. Outrigger Footing

This footing, as originally planned was founded on the backfill above the intake culverts. Tests in the excavation showed the backfill to have a relative compaction varying from 87 percent to 93 percent, down to elevation +3. As a result of this observation, the footing design was changed and was modified to be supported on both ends on the intake culverts. The overexcavation below the base of the footing, at elevation +5 was backfilled with concrete. The loads from the footing are transferred directly to the culvert walls and do not rely on any subgrade support between the two culverts.

6. Northeast Footing
   ° Footing E-3

As shown in Figure B-1, most of the footing is founded on native soil at the design base elevation, +6.0 ft, except for a small portion in the western end. In this area backfill with a relative compaction of 80 to 85 percent was removed and replaced with concrete extending down to about elevation +3 ft, at which elevation the native soil was encountered based on probing and field density test data.

7. East Footings
   ° Footing A

The footing is founded on native soil at the design base elevation +6 feet. Because of a design change to the footing the excavation made at the location of G-2 was excluded from the main footing and backfilled with concrete.

   ° Footing B

Most of the footing is founded on native soil except in a 10 ft wide area at the west end. In that area the excavation was made to elevation -1.0 ft, and the soil exposed at the base of the excavation was found to be backfill with a relative compaction of about 89 percent or lower, down to approximately elevation -5.0 feet. The overexcavation below the base of the footing, at elevation +5.5 ft, was backfilled with concrete. The eastern part of the footing rests on native soil and the west end is structurally connected to the anchor block. Therefore, the loads from the footing will be transferred to the native soil and to the anchor block without having to rely on the support of the backfill.

8. Southeast Footing

° Footing C

Approximately two thirds of the northern portion of the footing is founded on the existing anchor block at elevation +8.5 feet. In the south end, the footing is founded on backfill with a relative compaction of about 88 percent or lower. Results of probing indicate that backfill exists to elevation +5.0 ft underlain by native soil in this area. For this condition the loads are transferred directly to the anchor block without having to rely on the support of the backfill.

° Footing E

As shown in Figure B-1, the southern portion of the footing is founded on native soil at elevation +14.5 feet. In the northern portion the backfill with a relative compaction of 80 to 91 percent was excavated to elevation +12.0 ft except for an approximate 5 ft wide area at the north end. In that area the excavation was made to the turbine mat at elevation +8.5 feet. Therefore, the loads from the footing are transferred to the native soil and to the turbine mat without having to rely on the support of backfill.

9. Foundation of Auxiliary Feedwater Tank

The foundation is founded on native soil except for a small portion at the west end. In that area the field density tests indicated that the backfill has a relative compaction of about 97 percent or higher. The demolished septic tank area at the east end was excavated to elevation +7.0 ft, and the soil exposed at the base of the excavation was found to be native soil. The overexcavation was backfilled with lean concrete to elevation +10.5 ft, the base of the footing.

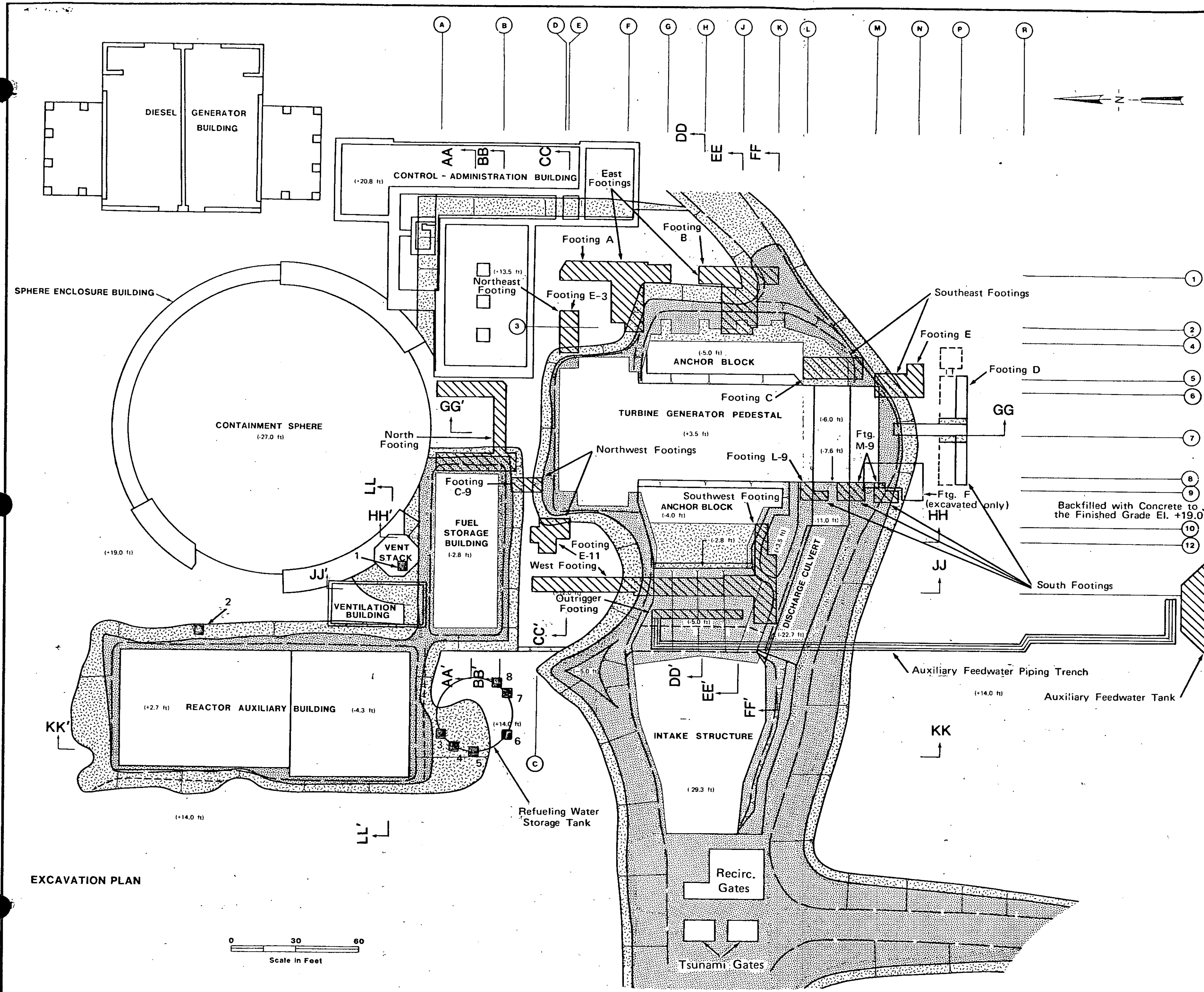
10. Auxiliary Feedwater Piping Trench

As shown in Figure B-1, most of the trench is founded on native soil at approximately elevation +8, except for the northern portion in the intake culvert area. In that area the excavation was made approximately to elevation +4, and the soil exposed at the base of the excavation was found to be backfill with relative compaction varying from 82 to 88 percent. Probing in this area indicated that the backfill was a minimum of 6 ft deep. As a result of this observation, a concrete, u-shaped trench was constructed. The load will be transferred to the intake culverts without having to rely on subgrade support between the two culverts.

11. Refueling Water Storage Tank

Approximately 60 percent of the footing is founded on native soil with the remaining 40 percent on recompacted soil in the northwest section as shown in Figures B-1 and B-13. In this area the backfill has a relative compaction of about 92 percent or higher.

12. Test Pits near Reactor Auxiliary Building and Ventilation Equipment Building	Tests in these areas showed low values of relative compaction ranging from 81 to 91 percent relative compaction.



EXCAVATION PLAN

LEGEND

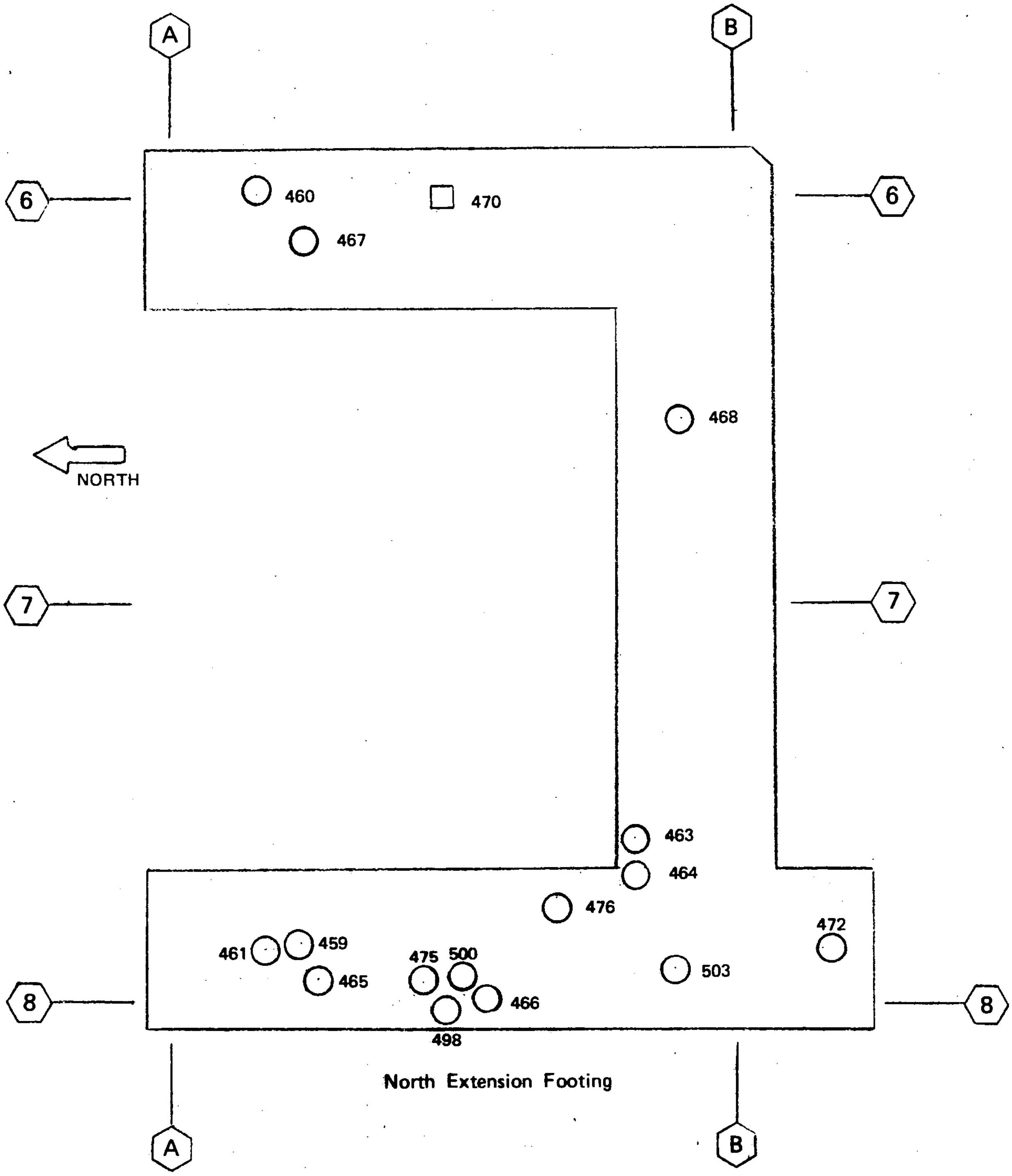
- Estimated toe of cut slope
- Indicates break between flatter upper slope and steeper lower slope
- Top of Cut for Construction / Water Table / Estimated Toe of Cut Slope / Edge of Structure — Fill above water table; Fill below water table
- HH'— HH Cross section location (Figures 2-18 to 2-21, Section 2.0)
- Location of footing (completed)
- Location of footing (proposed)
- Location of footing (excavated only)
- Location of trench
- (+3.5 ft) Elevation of native San Mateo Sand at the bottom of the foundation.
- Observation Pits
  - 1 — Ventilation Equipment Building
  - 2 — Reactor Auxiliary Building
  - 3–8 — Refueling Water Storage Tank

Woodward-Clyde Consultants

SITE PLAN SHOWING LOCATIONS INVESTIGATED DURING CURRENT SEISMIC UPGRADE PROGRAM

Project No. 413521
SONGS UNIT 1
SEISMIC RE-EVALUATION

Fig. B-1

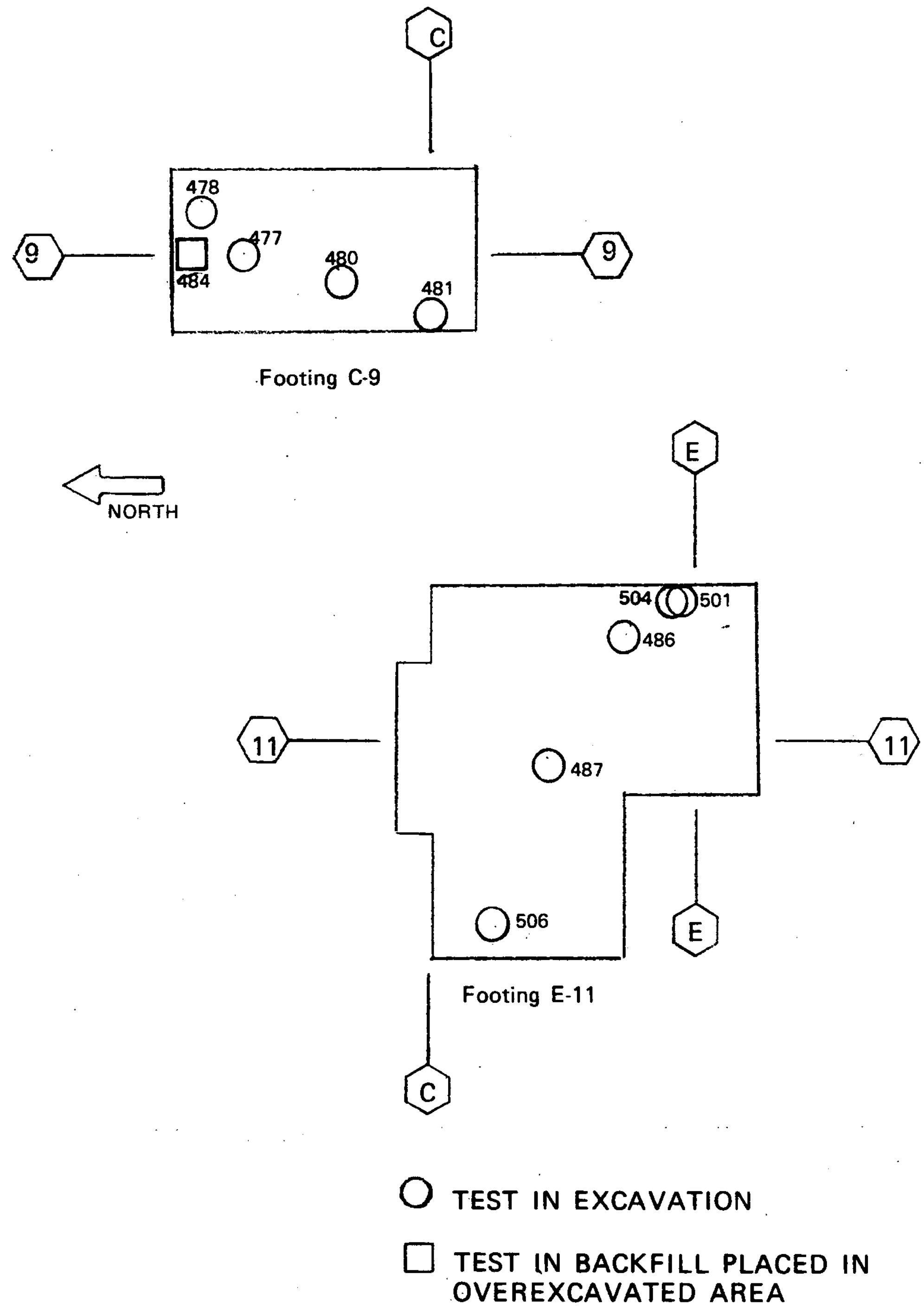


○ TEST IN EXCAVATION

□ TEST IN BACKFILL PLACED IN OVEREXCAVATED AREA

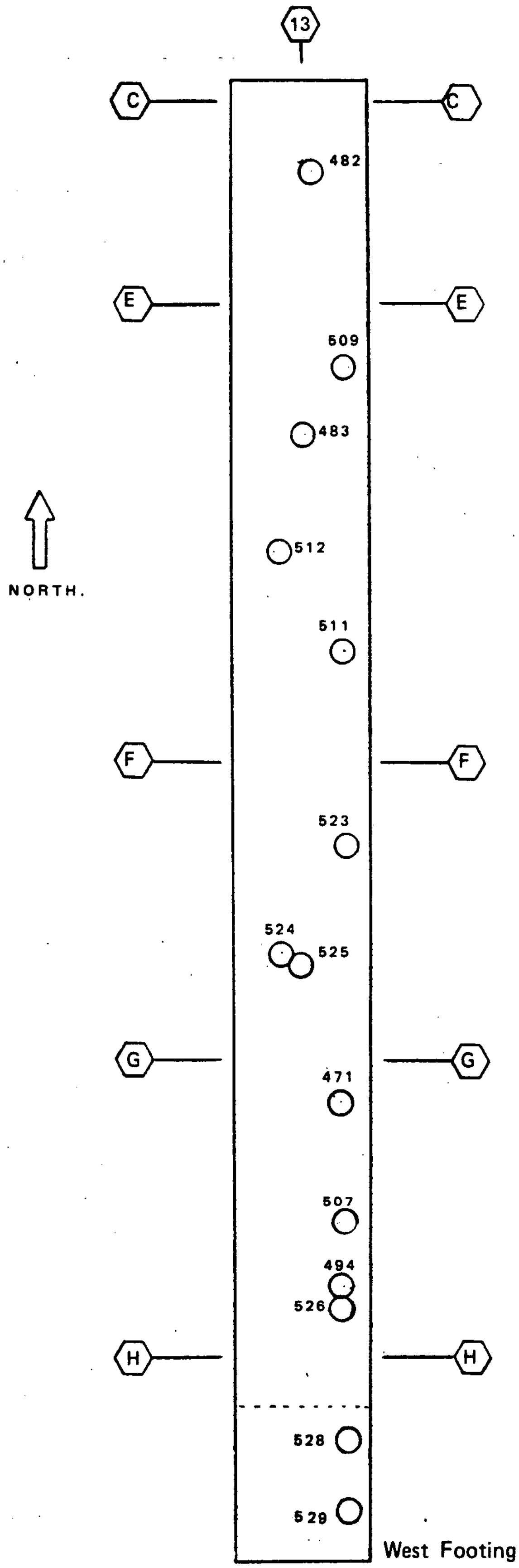
Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	LOCATION OF FIELD DENSITY TESTS NORTH PROJECT: LINE A–B & / 6–8	Fig. B-2

WOODWARD–CLYDE CONSULTANTS



Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	LOCATION OF FIELD DENSITY TESTS NORTH–WEST PROJECT: LINE C–9 & E–11	Fig. B-3

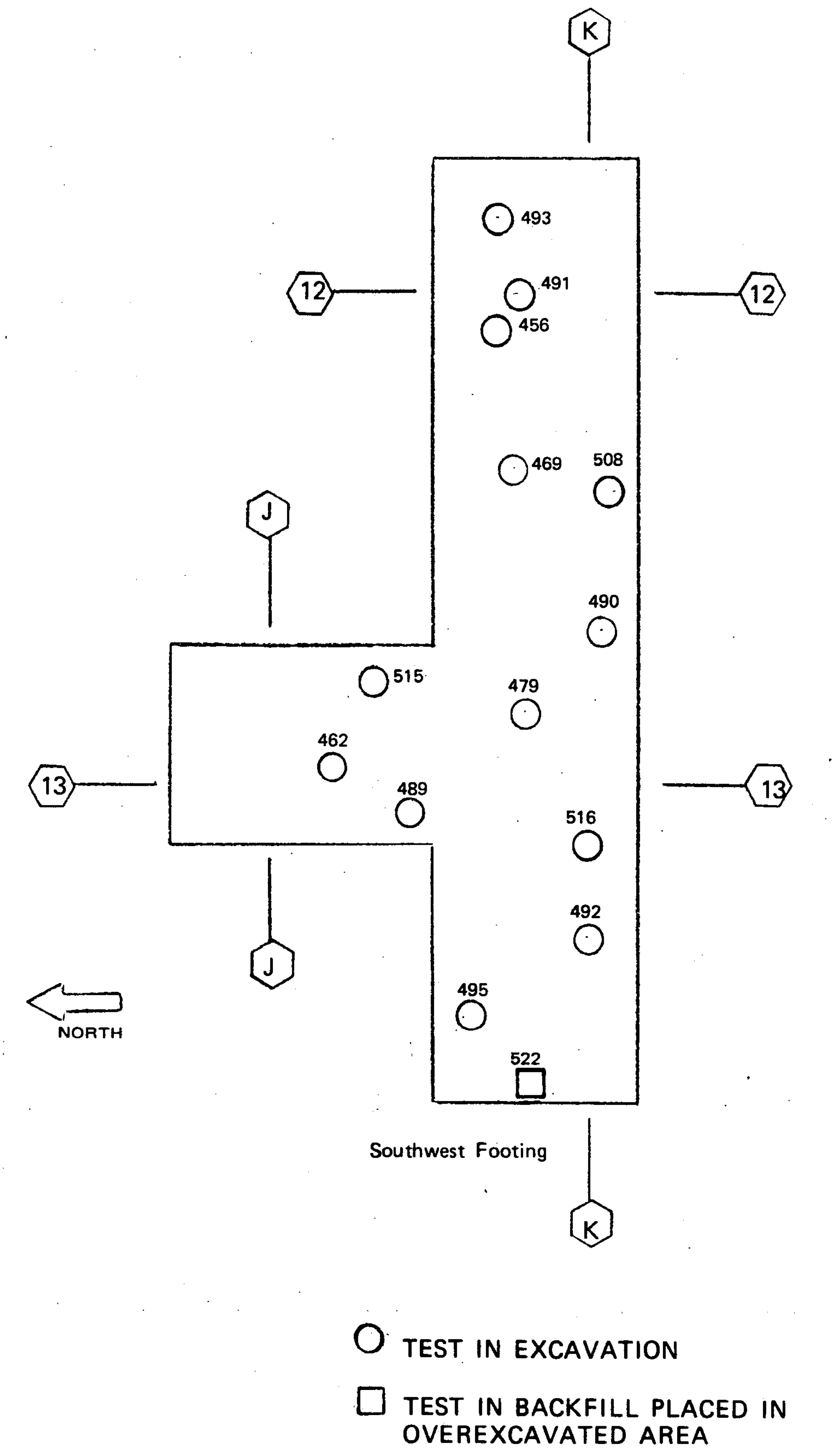
WOODWARD–CLYDE CONSULTANTS



O TEST IN EXCAVATION

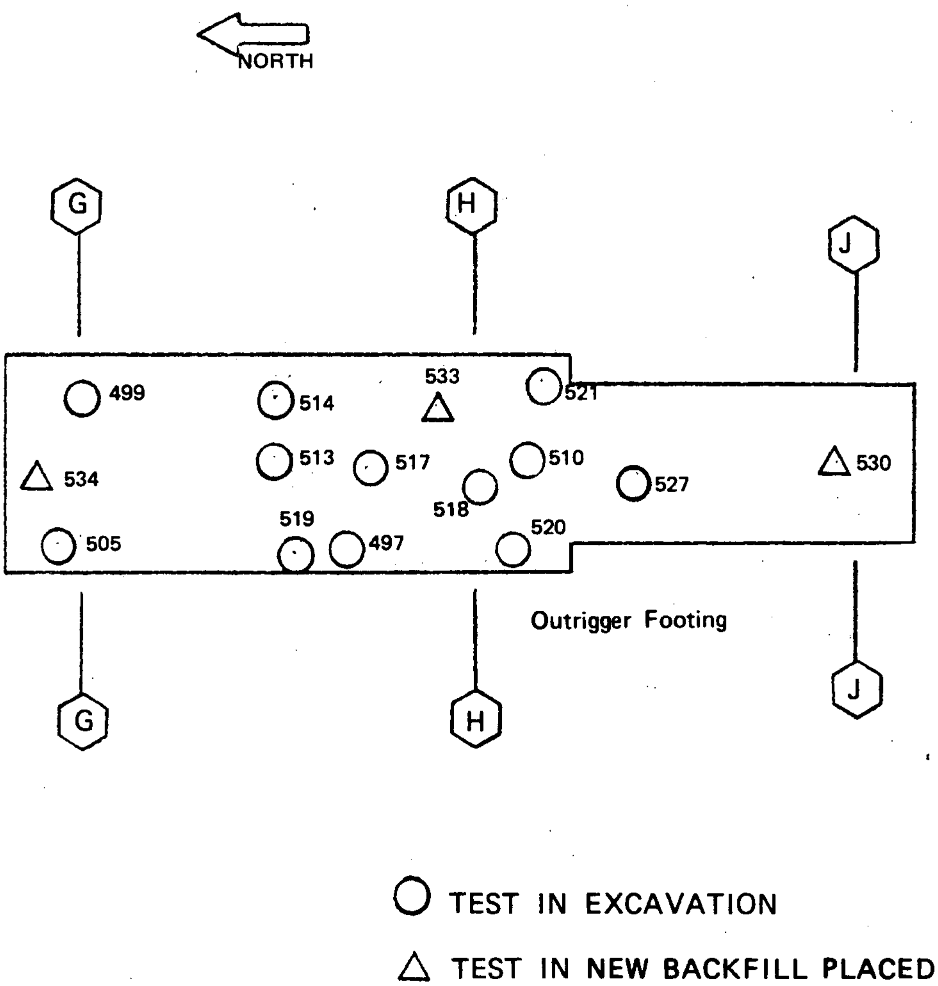
Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	LOCATION OF FIELD DENSITY TESTS WEST PROJECT: LINE C–H+ & 13	Fig. B-4

WOODWARD–CLYDE CONSULTANTS



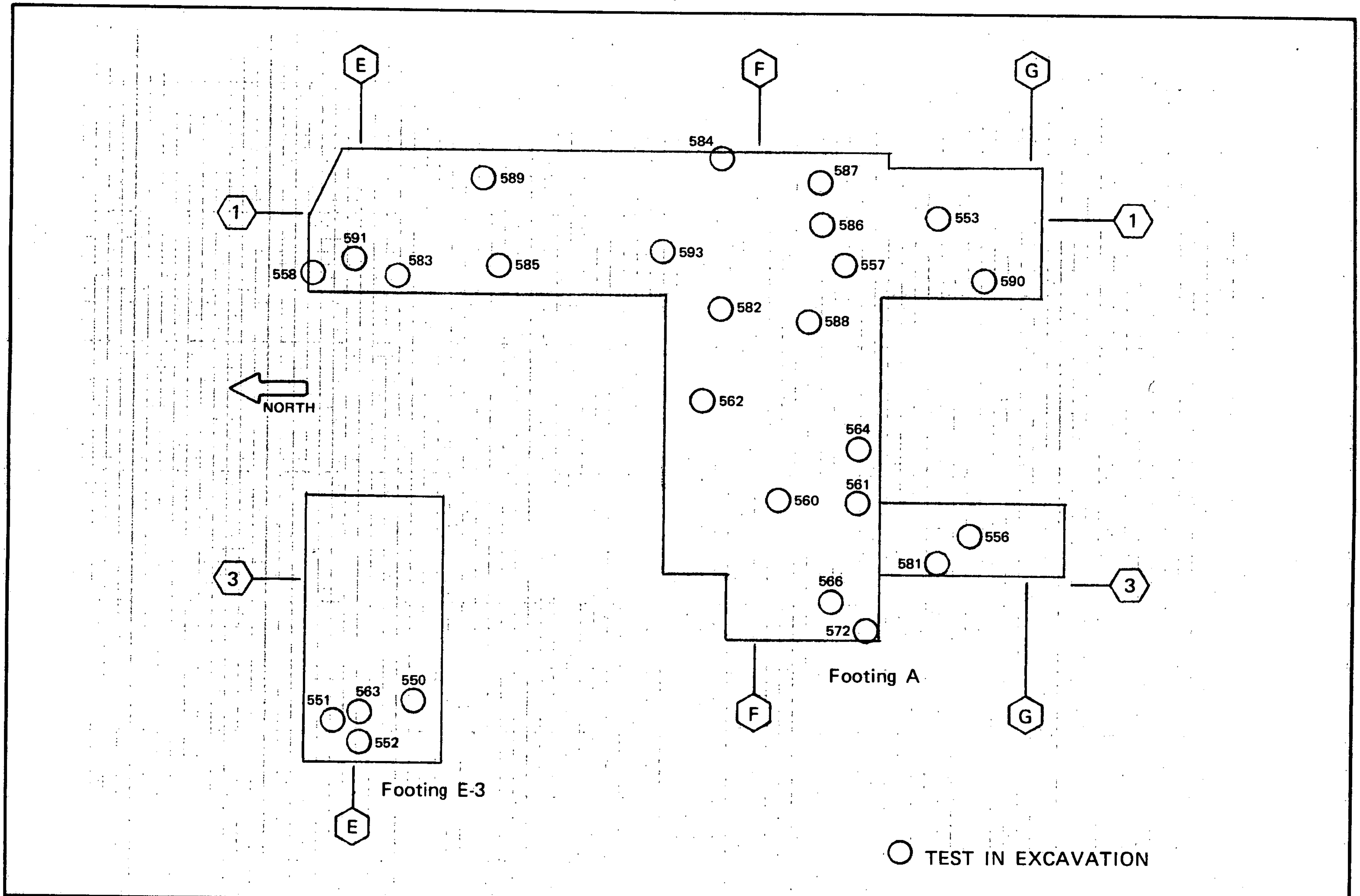
Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	LOCATION OF FIELD DENSITY TESTS SOUTH–WEST PROJECT: LINE K & 11–13+	Fig. B-5

WOODWARD–CLYDE CONSULTANTS



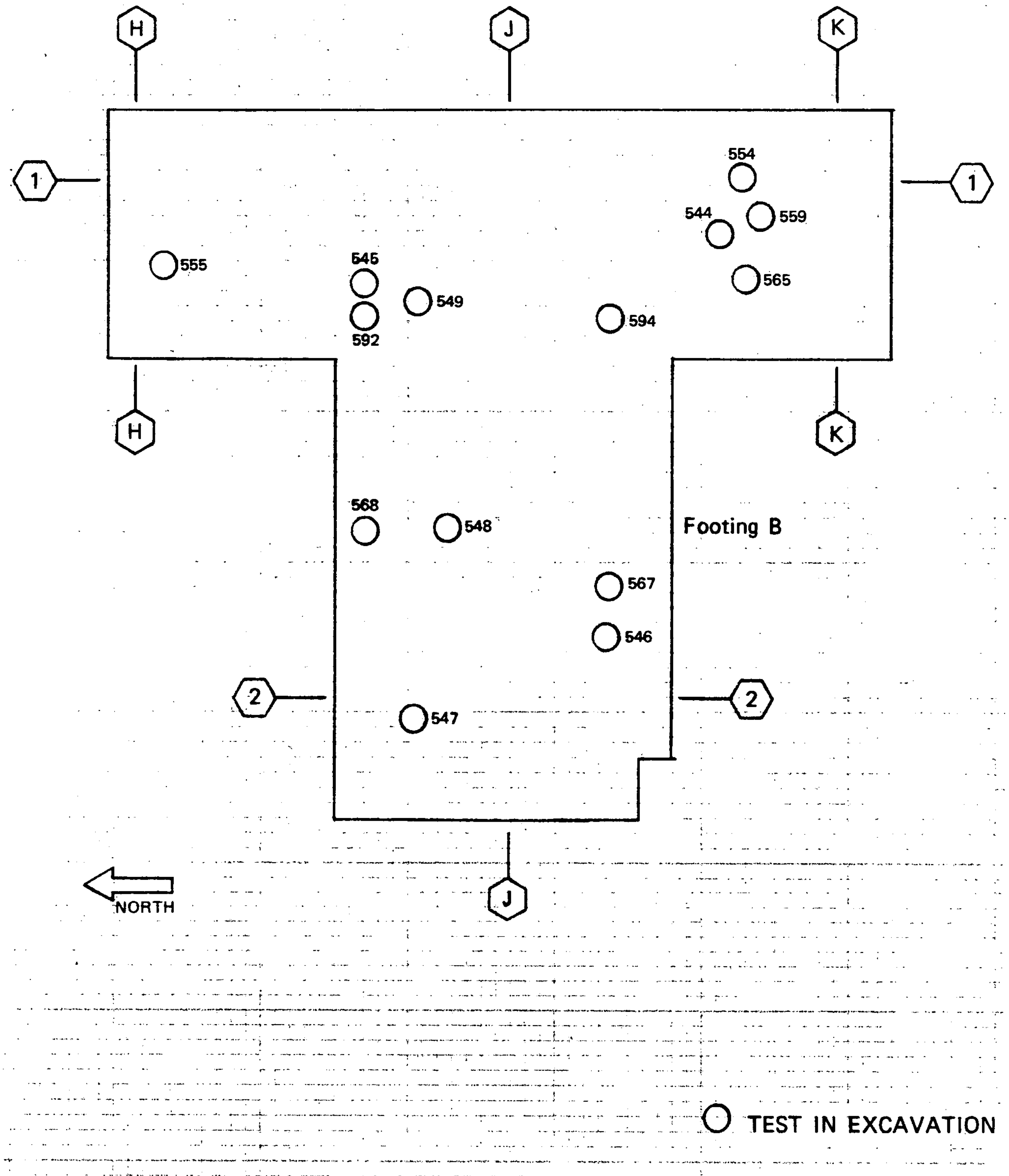
Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	LOCATION OF FIELD DENSITY TESTS OUTRIGGER FOOTING: LINE G–J	Fig. B-6

WOODWARD–CLYDE CONSULTANTS



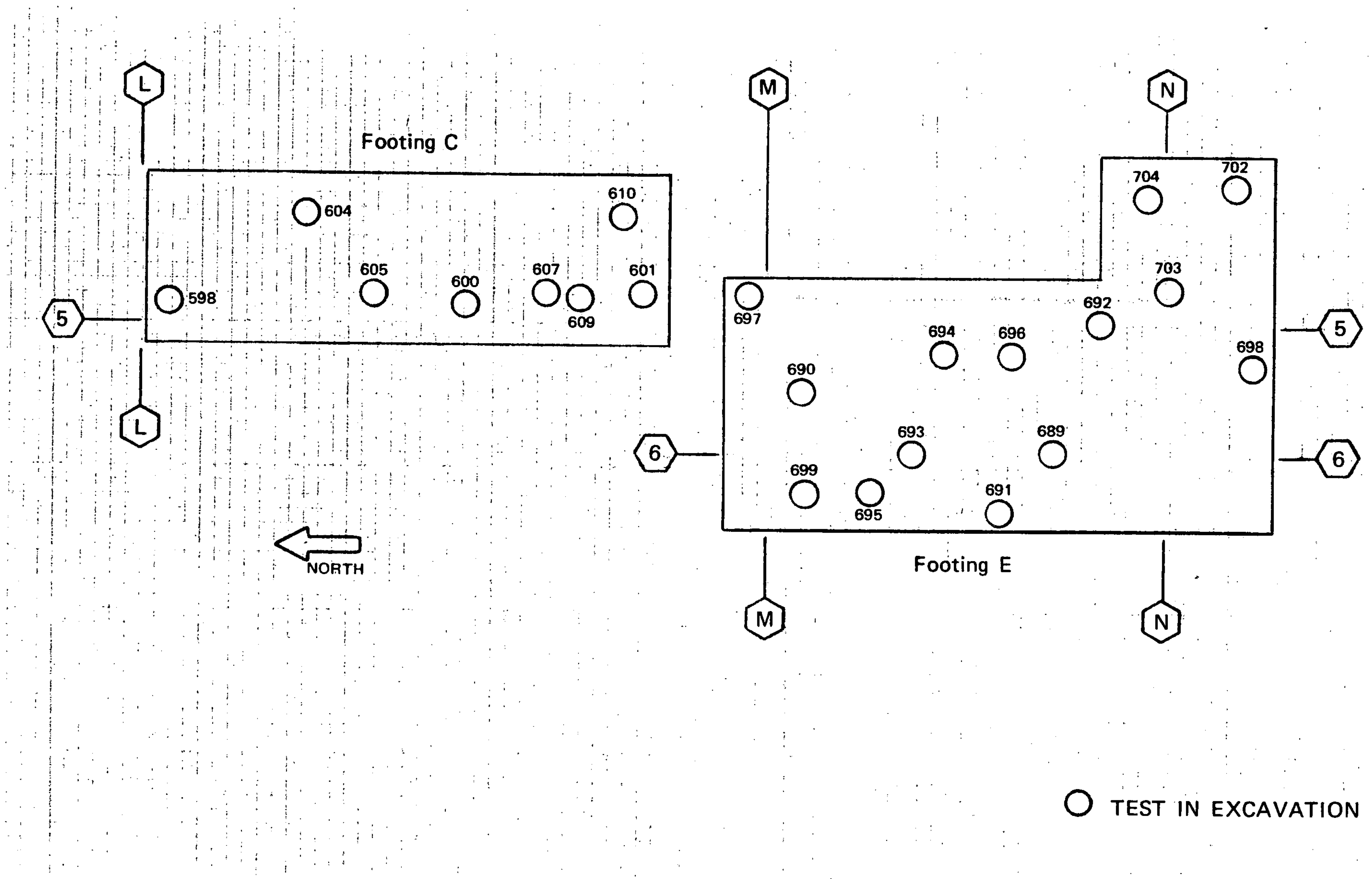
Project: SONGS UNIT 1 SEISMIC RE-EVALUATION **Project No.** 413521	LOCATION OF FIELD DENSITY TESTS NORTHEAST AND EAST PROJECT: LINE E–G AND 1–3	**Fig.** B-7

WOODWARD–CLYDE CONSULTANTS



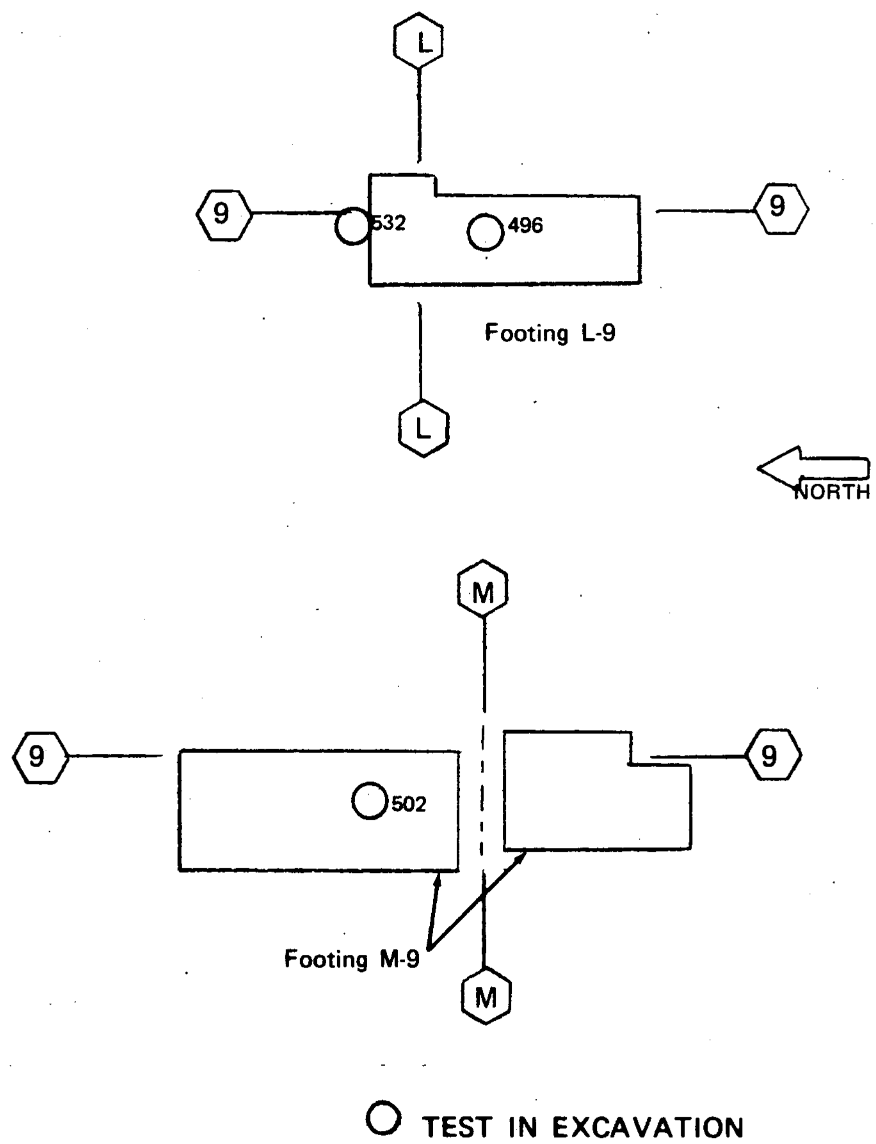
Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	LOCATION OF FIELD DENSITY TESTS EAST PROJECT: LINE H–K AND 1–2	Fig. B-8

WOODWARD–CLYDE CONSULTANTS



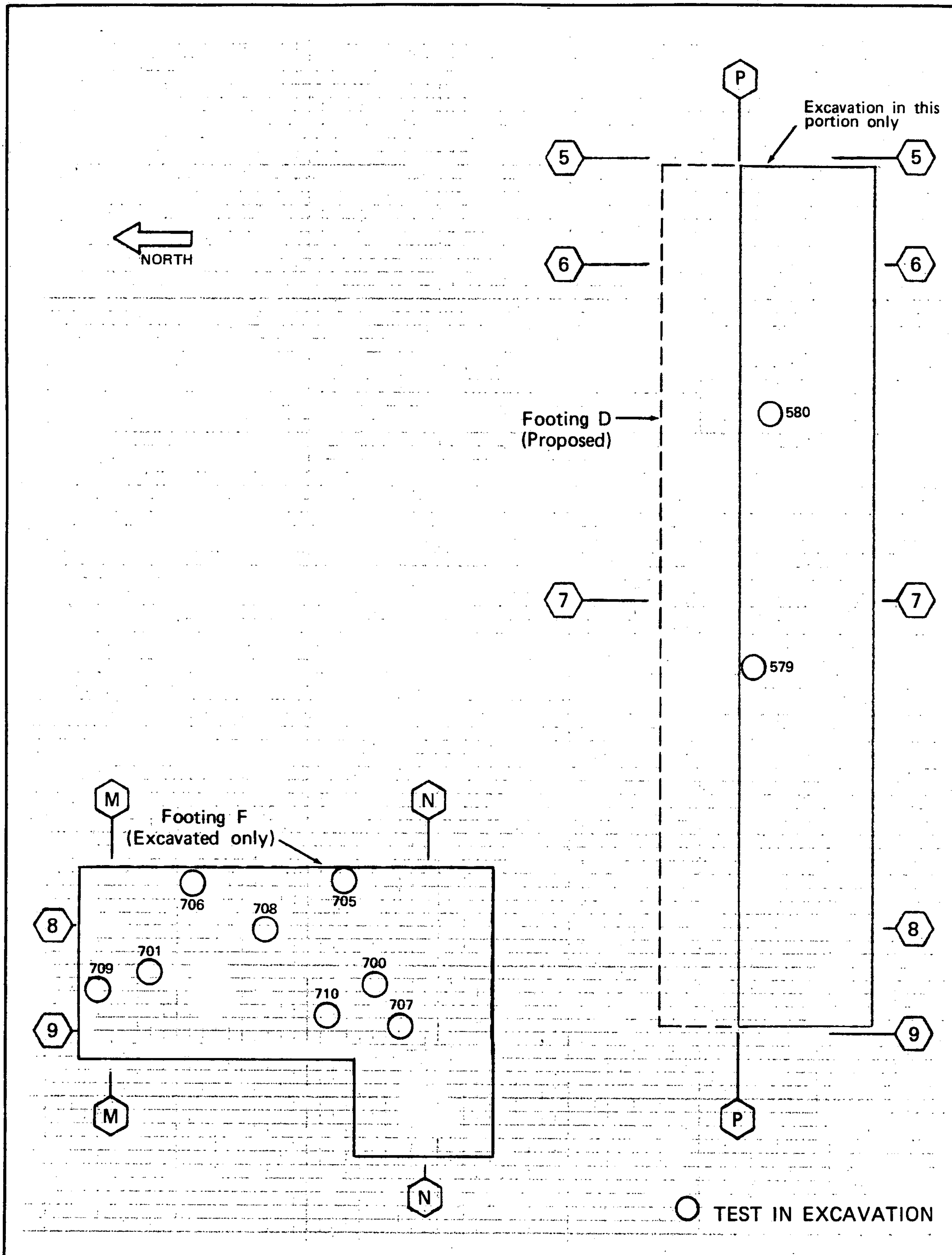
Project: Project No.	SONGS UNIT 1 SEISMIC RE-EVALUATION 41352I	LOCATION OF FIELD DENSITY TESTS SOUTHEAST PROJECT: LINE L–N AND 5–6	Fig. B-9

WOODWARD–CLYDE CONSULTANTS



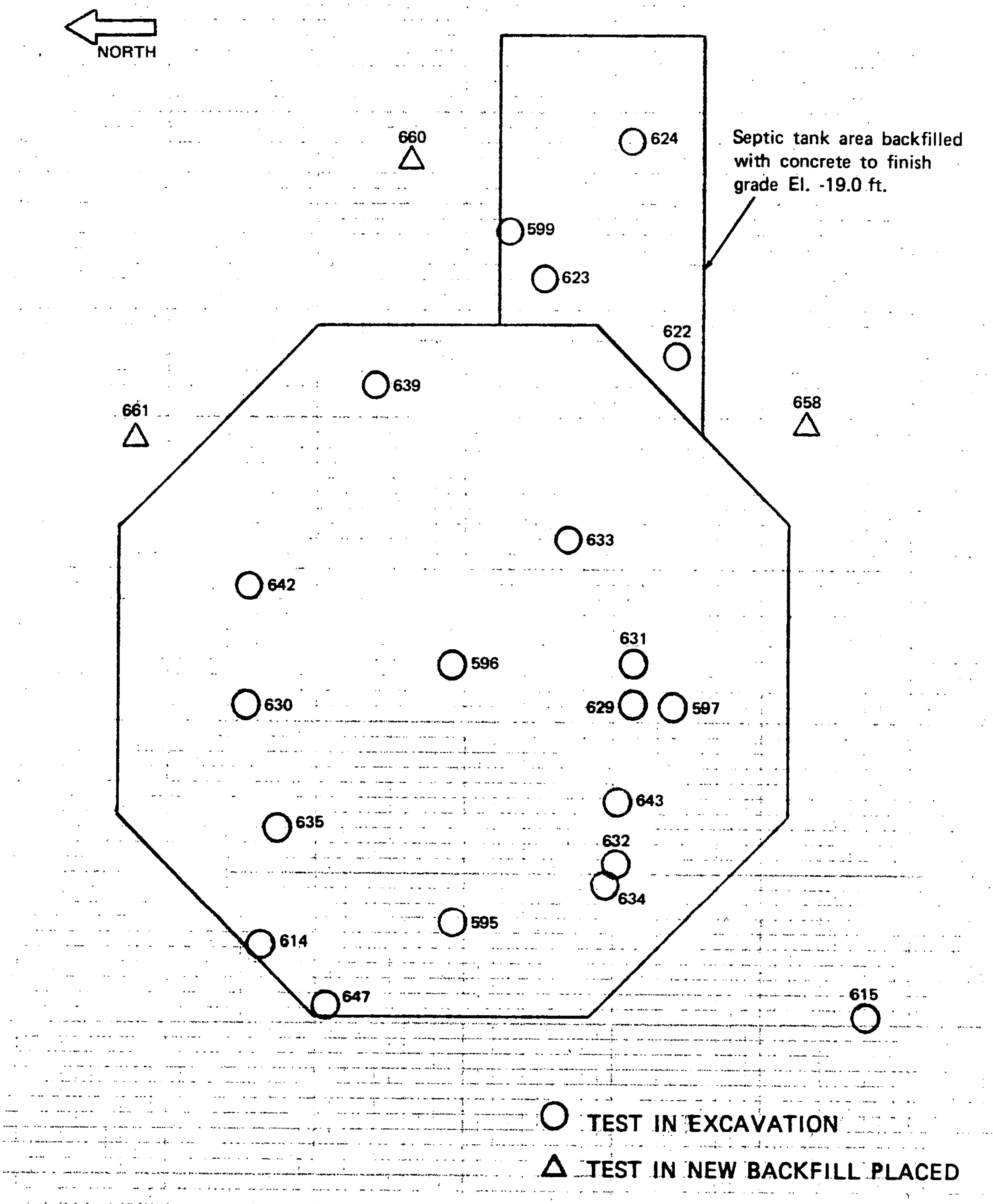
Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	LOCATION OF FIELD DENSITY TESTS SOUTH PROJECT: LINE L–M & 9	Fig. B-10

WOODWARD–CLYDE CONSULTANTS

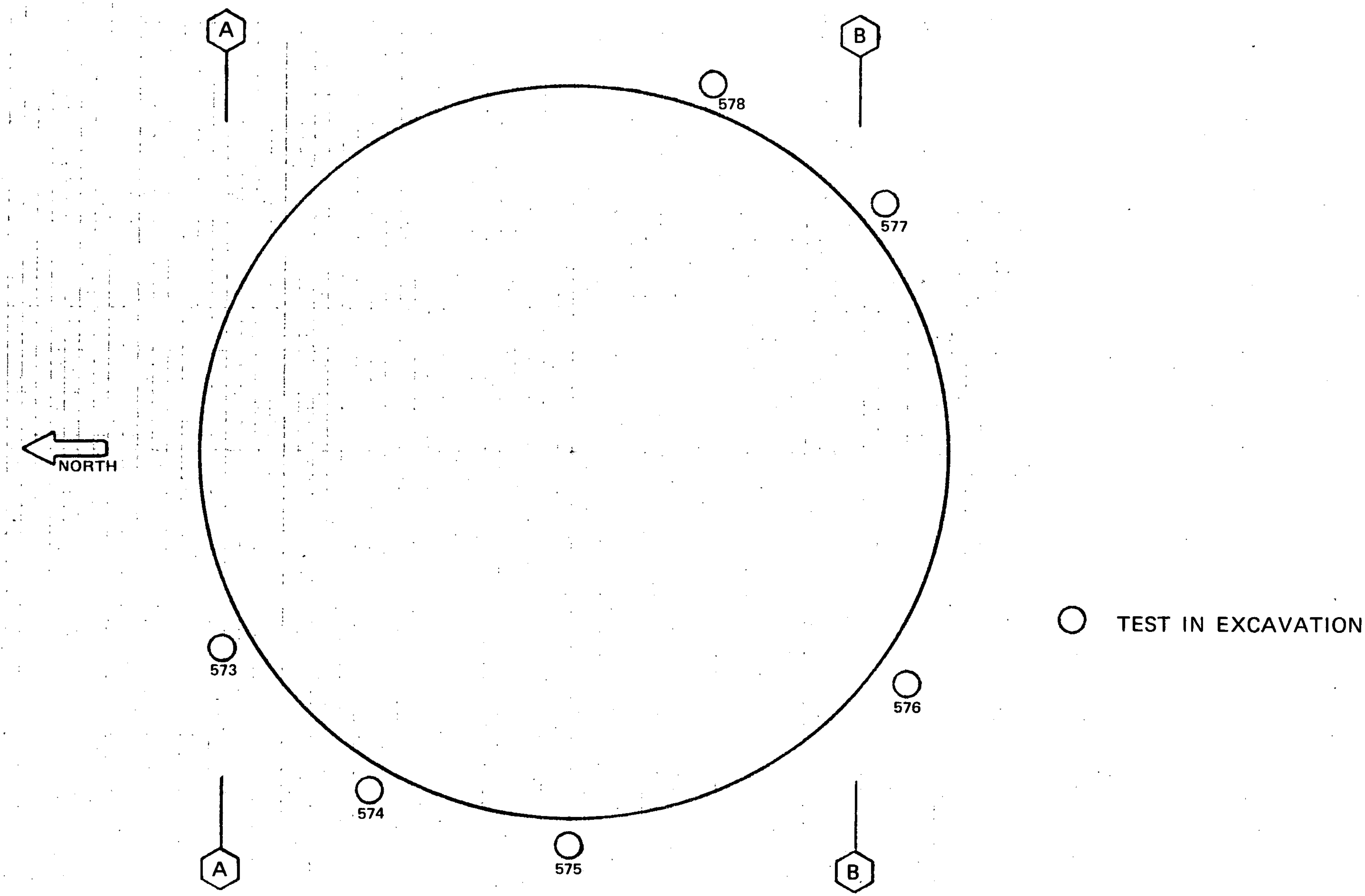


Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	LOCATION OF FIELD DENSITY TESTS SOUTH PROJECT: LINE N–P AND 5–6	Fig. B-11

WOODWARD–CLYDE CONSULTANTS

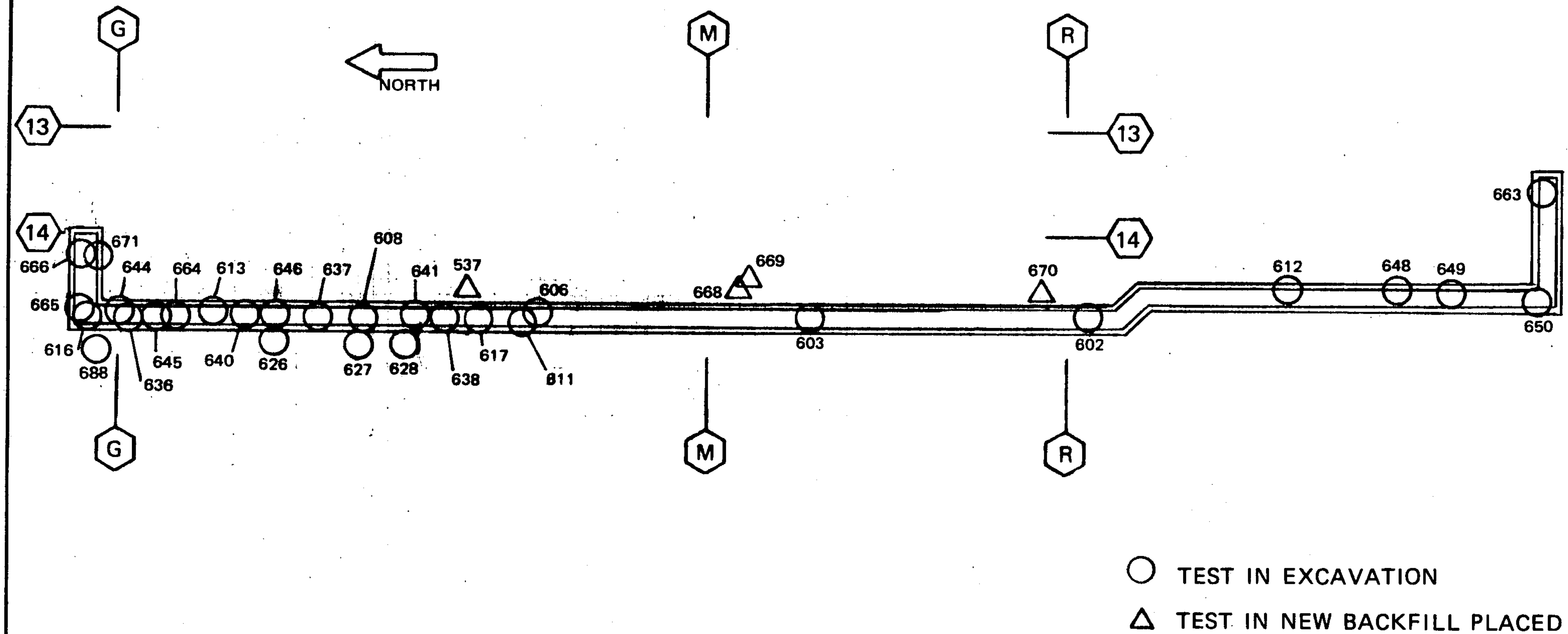


Project: Project No.	SONGS UNIT 1 SEISMIC RE-EVALUATION 41352I	LOCATION OF FIELD DENSITY TESTS AUXILIARY FEEDWATER TANK PROJECT	Fig. B-12



Project: SONGS UNIT 1 SEISMIC RE-EVALUATION **Project No.** 413521	LOCATION OF FIELD DENSITY TESTS REFUELING WATER STORAGE TANK PROJECT	**Fig.** B-13

WOODWARD–CLYDE CONSULTANTS



Project: Project No.	SONGS UNIT 1 SEISMIC RE-EVALUATION 413521	LOCATION OF FIELD DENSITY TESTS, EAST OF TURBINE PEDESTAL AUXILIARY FEEDWATER PIPING TRENCH	Fig. B-14

WOODWARD–CLYDE CONSULTANTS

APPENDIX C
LIQUEFACTION EVALUATION

C-1 INTRODUCTION

An evaluation of the liquefaction potential was made of SONGS 1 backfills considering the areal distribution and characterization of backfill soils shown in Figure 2-22 of the text of this report. The sections that follow describe the key elements of the analyses and summarize the results of liquefaction potential as a function of backfill density, geometry, and elevation relative to the water table. Section C-2 describes the earthquake ground motion considered in the analysis. The soil properties assigned to backfill as a function of density, confinement and geometry are presented in Section C-3. The liquefaction analyses and potential for liquefaction are presented in Section C-4 and a brief summary is presented in Section C-5.

C-2 GROUND MOTION

The SONGS Unit 1 FSAR seismic design criteria is characterized by the 2 percent damped response spectrum designated in Figure C-1 as the 1/2 g Housner spectrum. The 2/3 g Housner seismic reevaluation spectrum is also shown in Figure C-1. Based on the work completed for the SONGS 2 and 3 FSAR on ground motions, the seismic event which would control the response spectrum would be a postulated M7 earthquake at a distance of 8 km from the site on the hypothesized OZD. Recently, a report entitled "Comparison of 2/3 g Housner Reanalysis Spectrum with Multiple Regression Analyses of Spectral Values, San Onofre Nuclear Generating Station" dated June 1982 (based on studies by

Woodward-Clyde Consultants and Tera Corporation) showed that the reanalysis spectrum lies above the 84th percentile instrumental spectra for the San Onofre site. Figure C-2 shows the range of the 84th percentile instrumental spectra from the June 1982 study compared to the Housner reanalysis spectrum. Four accelerograms were chosen for liquefaction analysis whose spectra generally characterize the 2/3 g Housner response spectra as shown in Figure C-3. These accelerograms include Trace A and Trace B synthetic accelerograms used in the reevaluations described in the document entitled "Seismic Reevaluation and Modification" dated April 1977 and two IV-79 accelerograms as identified in Figure C-3. The range of the spectra of these four accelerograms is compared to the June 1982 84th percentile instrumental spectra range in Figure C-4. That range exceeds the 84th percentile instrumental spectra. The accelerograms are therefore considered conservative for use in the liquefaction analysis.

The curves of uniform stress cycles as a function of earthquake magnitude presented in Figure C-5 show that, for an M7 earthquake, 10 equivalent uniform stress cycles are a mean of the empirical data. Because the accelerographs used in the analysis represent at least an 84th percentile of the amplitude of ground motion, it is appropriate to consider a mean number of applied cycles in the analysis. Therefore, N = 10 equivalent uniform stress cycles has been chosen for the liquefaction analysis of fill soils at the SONGS Unit 1 site consistent with the conservative accelerograms used to obtain induced shear stresses.

C-3 SOIL PROPERTIES AND SITE CONDITIONS

Plant grade at SONGS 1 is at elevation +14 ft to +20 ft and has native San Mateo Sand exposed over most of the plant area except in those areas adjacent to major structures where backfills have been placed. The backfill areas have been characterized by density (relative compaction) as indicated in Figure 2-22 of this report and as discussed in Section 2. The dynamic strength of the backfill soils are therefore characterized in the paragraphs that follow, in terms of relative compaction and backfill geometry.

For the SONGS 1 site the cyclic shear strength of the soil can be developed from the following relationship:

$$\tau = \frac{\sigma_d}{2\sigma_{3c}} \quad C_1 \; C_r \; A_f \; B_f \; C_f \; \sigma_v'$$

where τ = cyclic shear stress to cause $\pm$ 5 percent strain for 10 uniform stress cycles in the field

$\frac{\sigma_d}{2\sigma_{3c}}$ = laboratory cyclic shear stress ratio required to cause $\pm$ 5 percent strain for 10 uniform stress cycles in cyclic triaxial tests, normalized to a confining pressure of 4 ksf

C_1 = correction factor for confining pressures other than 4 ksf

C_r = correction factor between cyclic triaxial and simple shear test results related to K_o and the relative compaction

A_f = aging correction factor on strength

B_f = fill geometry correction factor

C_f = compaction correction factor for in-situ stress conditions

σ_v' = effective field confining pressure

σ_d = cyclic deviator stress

σ_{3c} = initial consolidation pressure

Extensive laboratory testing has been completed for the San Mateo Sand and the results are summarized in Figure C-6 in the form of $\sigma_d/2\sigma_{3c}$ versus number of cycles for various values of relative compaction ranging from 85 to 100 percent. These curves are for a σ_{3c} of 4 ksf. The correction factor C_1 used to modify $\sigma_d/2\sigma_{3c}$ as a function of σ_{3c} is summarized in Figure C-7 based on data for other values of σ_{3c}.

Effects of Aging on SONGS 1 Backfill Materials

The effects of aging on the cyclic strength of sand are summarized in Figure C-8 from which the following observations are made:

1. The strength gain from initial deposition corresponding from point A to point B in Figure C-8 is not considered in this evaluation because of difference in soil deposition in the laboratory and in the field. For the present evaluation, the starting point was taken as point B. Therefore the strength gain for the fill at SONGS was calculated as the ratio of the ordinate of the shaded area divided by ordinate of point B.

2. The lower bound curve BC was conservatively used as an estimate of strength gain due to aging of fill at SONGS.

3. The fill at SONGS was placed approximately 15 years ago. Therefore, the aging correction factor is as follows:

$$\frac{1.45}{1.2} = 1.2$$

Effects of Backfill Geometry (Length/Height Ratio) A comparison of shaking table test results indicates that cyclic strength test results are significantly influenced by the length/height ratio of the test samples (DeAlba, Seed and Chan, 1976, ref. C-1). In order to develop an approximate correction factor, B_f, to account for the length/height (L/H) ratio of test samples, reference is made to Figure C-9 in which shaking table test results for a relative density of 50 percent are compared. For a given number of cycles, the higher the L/H ratio, the lower the ratio of the applied shear stress to the initial effective stress, τ/σ_o' causing initial liquefaction or ± 5 percent strain. The data summarized in Figure C-9 were, however, obtained for samples prepared by different procedures. Influence of the method of sample preparation on cyclic strength was assessed using the data obtained by Mulilis and others (1975) (ref. C-2), and presented in Figure C-10. Thus, the data from Figure C-9 were corrected for the effects of sample preparation to evaluate the effect of L/H ratio on cyclic strength. For N = 10 cycles, this evaluation shows that the cyclic strength for L/H = 10.3 (based on corrected test results by Finn, et al., 1977, ref. C-3) would be about 15 percent more than that for L/H = 22.5 (based on corrected test results by DeAlba, et al., 1976, ref. C-1).

Using the data presented by O'Hara (1972) (ref. C-4), for fine sand with L/H = 3.4 and for N = 10 cycles, the cyclic stress ratio is found to be about 40 percent higher than for the data given for L/H = 22.5. This 40 percent difference was reduced to 35 percent to offset uncertainty in the method of sample preparation.

Finally, for the one data point reported by Ortigosa (1972), ref. C-5, for L/H = 2.3, assuming a curve parallel to the curves suggested by the above data could be reasonably drawn, the cyclic stress ratio is found to be about 70 percent higher than for the data presented for L/H = 22.5. For conservatism an overall geometry of only 50 percent has been used.

The stress ratios discussed above are summarized in Figure C-11 in the form of a correction factor B_f with respect to shaking table data on long thin samples versus the length/height ratio, L/H for a relative density of 50 percent. Various backfills are identified by numbered solid triangles in Figure C-12 and the L/H and resulting B_f values are tabulated in Table C-1.

Effects of Multidirectional Shaking - Studies have shown that the stress ratio required to cause a peak cyclic pore pressure ratio of 100 percent under multidirectional shaking conditions is about 10 percent less than that required under unidirectional shaking conditions (Seed, 1976, ref. C-6). Accordingly, cyclic triaxial test results can be corrected to obtain values of τ/σ_o' representative of large-scale simple shear conditions. For this purpose, the cyclic triaxial test data in terms of $\sigma_d/2\sigma_{3c}'$ should be multiplied by a correction factor, C_r, on the order of 0.54 to 0.58 depending on the number of stress cycles involved. For SONGS Unit 1, for N = 10 cycles, a C_r = 0.57 is appropriate. The value of C_r used for unidirectional shaking is 0.63 which is as indicated above, about 10 percent higher than that used for multi-directional shaking. DeAlba and Seed (1976) ref. C-1, found that the value of C_r was independent of the relative density of the soil tested.

<u>Effects of Field Compaction</u> - The correction factor C_r = 0.57 discussed above applies to multidirectional shaking for normally consolidated sands for which the coefficient of earth pressure at rest, K_o, may be taken as 0.4. For values of overconsolidation ratio, OCR, of the order of 6 to 8 (Hendron (1963), ref. C-7), has obtained results which indicate values of K_o of one or more. Based on data presented by Seed, Arango, and Chan (1975) (ref. C-8), Figure C-13 has been developed. The figure shows a linear relationship between C_r and OCR assuming a C_r = 0.57 for K_o = 0.4 (OCR = 1) and a C_r = 0.90 for K_o = 0.90 (OCR = 6).

Corresponding values for unidirectional shaking are a C_r of about 0.63 for K_o = 0.4 and a C_r of about one for K_o = 1. In other words, the cyclic stress ratio required to cause the same pore pressure ratio in the same number of cycles is about 50 to 60 percent greater for an overconsolidated sand (with an OCR of about 6 to 8 producing a K_o = 1) as compared to a normally consolidated sand with K_o = 0.4. Accordingly, a compaction correction factor, C_f, may be introduced to account for the overconsolidation of backfills. C_f is defined as the ratio of the value of C_r corresponding to an appropriate value of K_o for the compacted backfill divided by the value of C_r for the normally consolidated backfill material.

Because the value of K_o is greater for a compacted fill than the value of K_o for a normally consolidated fill, the value of C_f is expected to be greater than 1.0. D'Appolonia, et al. (1969), ref. C-9, have shown that sand fills compacted to high relative densities have high values of K_o, as great as 2 to 3, but with values typically being about 1.5. However, as the fill is increased in thickness, K_o values at depth do not remain at such high values. In fact, Lacroix and Horn (1973) (ref. C-10), have presented data (for heavily compacted sand fills with relative densities of about 97 percent) which show the value of K_o decreasing from a value between 2.0 and 2.5 near the surface to a value of about 0.5, corresponding to an "at rest" or normally consolidated condition, at a depth of several tens of feet as shown in Figure C-14.

The variation of K_o with vertical effective stress for SONGS Unit 1 backfill compacted to 95 percent relative compaction ($D_r \approx$ 85 percent) has been plotted in Figure C-14. This variation of K_o for $D_r \approx$ 85 percent with vertical effective stress is conservatively developed based on consideration of: 1) the amount of overconsolidation expected at this relative density during compaction (after D'Appolonia, et al., 1969, ref. C-9); 2) the subsequent final overconsolidation ratio (OCR) as a function of depth after the fill is placed; and 3) the determination of K_o from OCR based on Hendron (1963, ref. C-7).

The estimated variation of K_o with vertical effective stress for SONGS 1 backfills as shown in Figure C-14 and the relationship between C_r and K_o as shown in Figure C-13 were used to develop the relationship between C_f and effective vertical stress shown in Figure C-15. For each given depth, or vertical effective stress, the expected value of C_r corresponding to the expected value of K_o for the compacted backfill from Figure C-13 is divided by $C_r = 0.57$ which corresponds to $K_o = 0.4$ (normally consolidated sand) to obtain the factor C_f. Thus, the variation of C_f for a relative compaction of 95 percent is plotted in Figure C-15. For a normally consolidated soil, the compaction correction factor, C_f, is expected to be one for all depths. A backfill material with a relative compaction of about 85 percent is assumed to have a K_o = 0.4, thus producing the C_f = 1.0 vertical line shown in Figure C-15.

Also plotted in Figure C-15 are "interpolated" variations of C_f with vertical effective stress for backfill materials compacted to relative compactions of 90 and 92 percent. These curves were conservatively interpolated with a greater-than-linear decrease in C_f at a given effective vertical stress as a function of relative compaction between 85 and 95 percent relative compaction.

The curves shown in Figure C-15 should reasonably show the variations of C_f with vertical effective stress when applied to large areas of compacted fills. However, the following two exceptions are judged to be appropriate for the field conditions:

1. For areas where compaction is believed to have been obtained by a jetting process, the value of C_f should be taken as 1.0. It is believed that it is unlikely to obtain relative compaction values of higher than about 90 percent by jetting. Therefore, the possibility of compaction through jetting is disregarded for the areas of fill with higher than 90 percent relative compaction.

2. For relatively shallow backfills located in areas wide enough for compaction rollers to operate, yet narrow enough to develop high horizontal stresses, the values of K_o developed are believed to be high enough to correspond to values of C_f of the order of 1.5.

For the present case, backfills judged to have been compacted by jetting are assumed to have $C_f = 1$. For all other cases the curves on Figure C-15 are conservatively utilized realizing that for some narrow backfills, a higher value of C_f may be appropriate.

C-4 LIQUEFACTION ANALYSIS

The evaluation of liquefaction potential can be developed as a factor of safety against liquefaction (± 5 percent strain) by the following equation:

$$F.S. = \frac{\dfrac{\sigma_d}{2\sigma_{3c}} \quad C_1\ C_r\ A_f\ B_f\ C_f\ \sigma_v'}{\tau_i}$$

where $\dfrac{\sigma_d}{2\sigma_{3c}}$ = laboratory strength from Figure C-6

C_1 = correction factor for confining pressure from Figure C-7

C_r = 0.57

A_f = 1.2

B_f = fill geometry correction factor from Figures C-11 and C-12 and Table C-1.

C_f = compaction correction factor from Figure C-15

τ_i = average induced shear stress from an M7 earthquake at 8 km

σ_v' = effective overburden pressure.

The seismic induced shear stresses, τ_i, were determined by individually analyzing the response of a one-dimensional model of the SONGS site using the program SHAKE (Schnabel

[1972] ref. C-11), and the four accelerograms identified in Section C-2 above. The model of the SONGS site utilized strain-dependent modulus and damping parameters for the San Mateo Sand as developed in the SONGS 2 and 3 FSAR with appropriate modification for fills of lower density based on Seed and Idriss (1967, ref. C-12). The average induced stresses developed from these analyses and plotted as a function of depth for each relative compaction are presented in Figure C-16. Incorporating these stresses together with the other parameters into the above equations, plots of FS versus depth were developed for the category A, B, C, and D fills as shown in Figures C-17, C-18, C-19, and C-20 for specific locations typical of areal fills as shown in Figure C-12.

The understanding of the pore-water pressure induced by the earthquake can be developed from the factor of safety by consideration of the results of laboratory tests. As an example, for the 95-percent relative-compaction curve in Figure C-21, the ratio of N/N_ℓ (where N=10 cycles and N_ℓ is the number of cycles to liquefaction), can be developed for various calculated factors of safety as shown in Figure C-21. The pore pressure ratio ($r_u = u/\sigma_o'$ where u = pore water pressure and σ_o' = effective overburden pressure) is next determined by using the results of laboratory tests showing the rate of pore pressure increase in terms of pore pressure ratio, r_u, as a function of N/N_ℓ, as shown in Figure C-22. The relationship between pore pressure ratio, r_u, and factor of safety can thus be developed as shown in Figure C-23. Because the curves in Figure C-21 for lower values of relative compaction are equal to or flatter than that for 95 percent relative compaction, the curve on Figure C-23 is considered appropriately conservative for use at all relative compactions between 85 and 95 percent.

Using the results shown in Figures C-17 through C-19 together with Figure C-23, Figures C-24, C-25, and C-26 were developed showing the calculated induced maximum pore pressure ratio as a function of depth for category A, B, and C fills, respectively. Category D fills are not shown as the factors of safety are less than one as indicated in Figure C-20 indicating a pore pressure ratio of 1.0.

C-5 SUMMARY

The liquefaction potential has been quantified in terms of factor of safety against liquefaction for the various soil categories and locations at the site. These factors of safety have been summarized in Figures C-17 through C-20. The significance of these results is discussed in Section 3 of the text of this report. These results were also utilized to estimate the potential for seismic-induced pore-water pressures presented in terms of pore pressure ratio, r_u, for the various soil categories and locations at the site as summarized in Figures C-24 through C-26. The significance of these results is also discussed in Section 3. The results have also been used in the example calculation of the seismically induced settlement presented in Appendix D.

APPENDIX C - REFERENCES

C-1 De Alba, P., Seed, H. B., and Chan, C. K., Sand liquefaction in large-scale simple shear tests, Journal of the Geotechnical Engineering Division, ASCE, Vol. 102, No. GT9, Proc. Paper 12403, September 1976, pp. 909-927.

C-2 Mulilis, J. P., Chan, C. K., and Seed, H. B., The effects of method of sample preparation on the cyclic stress-strain behavior of sands, Report No. EERC 75-18, Earthquake Engineering Research Center, University of California, Berkeley, California, July 1975.

C-3 Finn, W. D. L., Emergy, J. J., and Gupta, Y. P., Soil liquefaction studies using a shaking table, Closed Loop Magazine, Fall/Winter, 1971, published by MTS Systems Corporation, Minneapolis, Minnesota.

C-4 O-Hara, S., The results of experiment on the liquefaction of saturated sands with a shaking box: comparison with other methods, Technology Reports on the Yamaguchi University, Vol. 1, No. 1, Yamaguchi, Japan, December 1972.

C-5 Ortigosa, P., Licuacion de arenas sometidas a vibraciones horizontales, Revista del Instituto de Investgaciones de Ensoyes de Materials, Vol. II, No. 3, December 1973.

C-6 Seed, H. B., Some aspects of sand liquefaction under cyclic loading, Proceedings, International Conference on Behavior of Offshore Structures, August 2-5, 1976, The Norwegian Institute of Technology, Trondheim, Norway.

C-7 Hendron, A. J. Jr., The behavior of sand in one-dimension compression, thesis presented to the University of Illinois, Urbana, Illinois, 1963, in partial fulfillment of the requirements for the degree of Doctor of Philosophy.

C-8 Seed, H. B., Arango, I., and Chan, C. K., Evaluation of soil liquefaction potential during earthquakes, Report No. EERC75-28, Earthquake Engineering Center, University of California, Berkeley, California, 1975.

C-9 D'Appolonia, D. J., Whitman, R. V., and D'Appolonia, E., Sand compaction with vibratory rollers, Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 95, No. SM1, Proc. Paper 6366, January 1969, pp. 263-284.

C-10 Lacroix, Y. and Horn, H. M., Direct determination and indirect evaluation of relative density and its use on earthwork construction projects, ASTM STP 523, American Society for Testing and Materials, 1973, pp. 251-280.

C-11 Schnabel, P. and Lysmer, J., SHAKE - a computer program for earthquake and response analysis of horizontally layered sites, Earthquake Engineering Research Center, University of California, Berkeley, Report EERC-72-12, 1972.

C-12 Seed, H. B. and Idriss, I. M., Analysis of soil liquefaction: Nigata earthquake, Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 93, No. SM3, Proc. Paper 5233, May 1967, pp. 83-108.

TABLE C-1

LENGTH/HEIGHT RATIOS OF BACKFILL AREAS

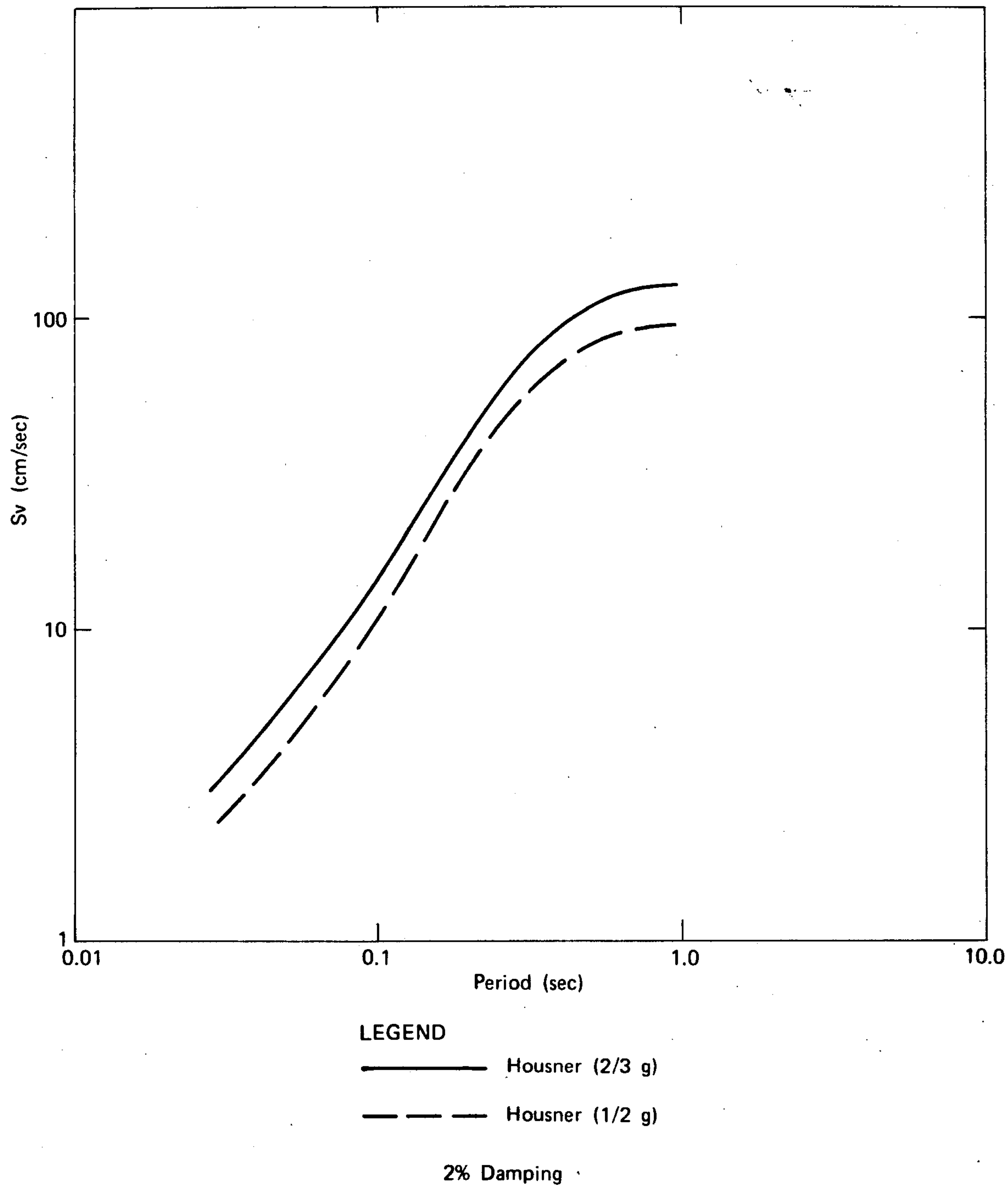
Area	Location	Soil Category(1)	L/H(2)	B_f(3)
51-52	North of Auxiliary Bldg.	D	3-3/4	1.32
52-53	West of Auxiliary Bldg.	D	8	1.18
51-54	East of Auxiliary Bldg.	D	10	1.15
53-55	South of Auxiliary Bldg. and North of Fuel Storage	D	9	1.17
55-56	East of Fuel Storage	D	12-1/2	1.10
54-56	South of Fuel Storage	D	5	1.27
57-58	Control-Administration Bldg.	D	-----------	(4)
58-59	Control-Administration Bldg.	D	-----------	(4)
60-62	North of Pump Well and West Footing	B	4	1.30
62-63	West of Turbine Generator and North of West Anchor Block	D	8	1.18
63-64	North of Turbine Generator	D	17	1.06
64-65	East of Turbine Generator and North of East Anchor Block	C	8	1.18
65-66	North of East Anchor Block	C	1.6	1.50
66-67	East of East Anchor Block	C	6	1.22
67-69	South of Anchor Blocks, Turbine Generator and Pump Well	B, C	7	1.20
61-68	East of Pump Well and West of West Anchor Block	A, B, C	3-1/3	1.35
60-70	West of Pump Well	C	wide open area	1.0

(1) See Figure C-12.

(2) Approximate length of fill to depth of fill ratio.

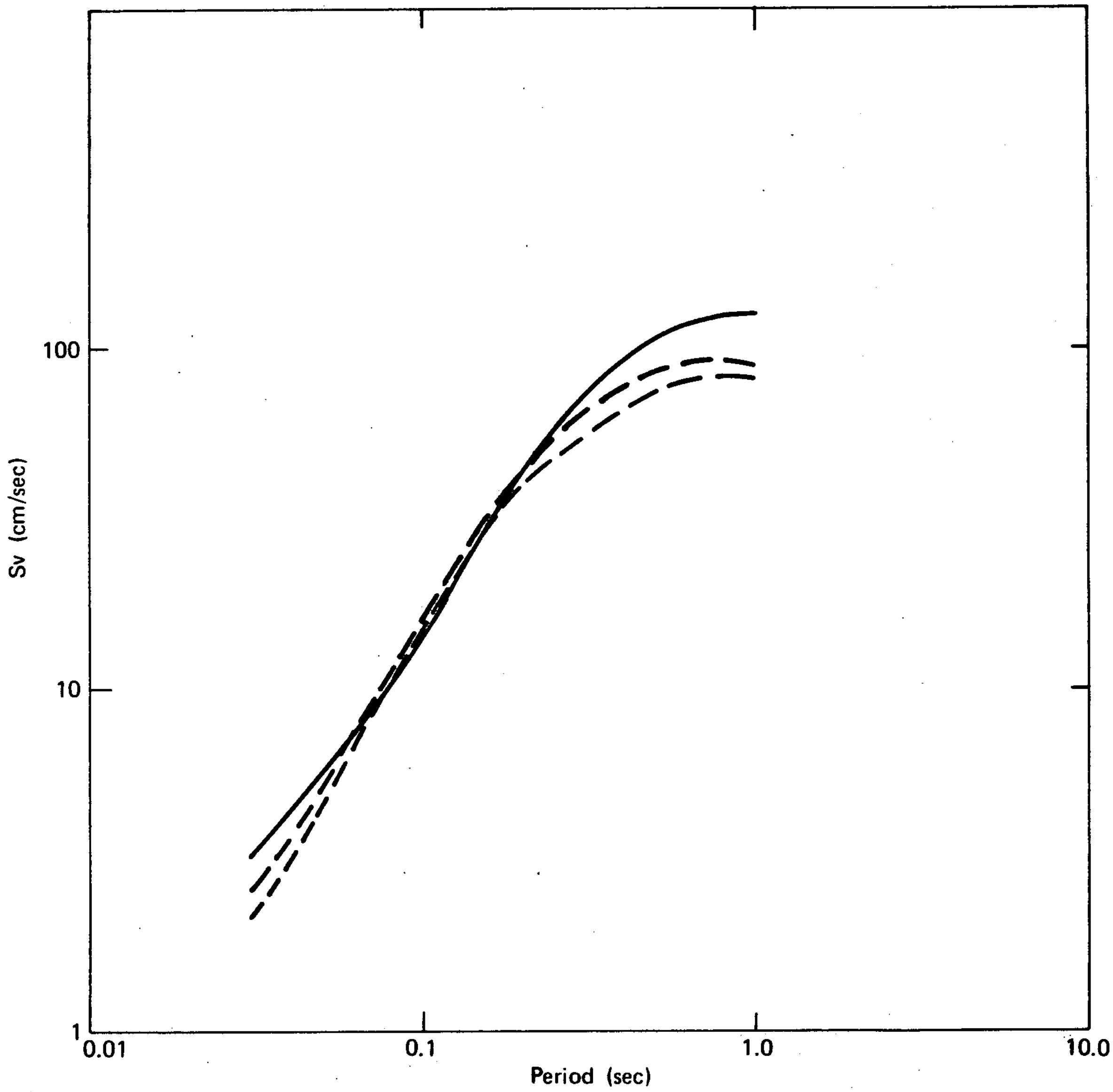
(3) Fill geometry correction factor (see Figures C-11 and C-12).

(4) All soil above water table.



Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	INITIAL DESIGN CRITERIA 1/2 g HOUSNER AND RE-EVALUATION CRITERIA 2/3 g HOUSNER	Fig. C-1

WOODWARD–CLYDE CONSULTANTS



LEGEND

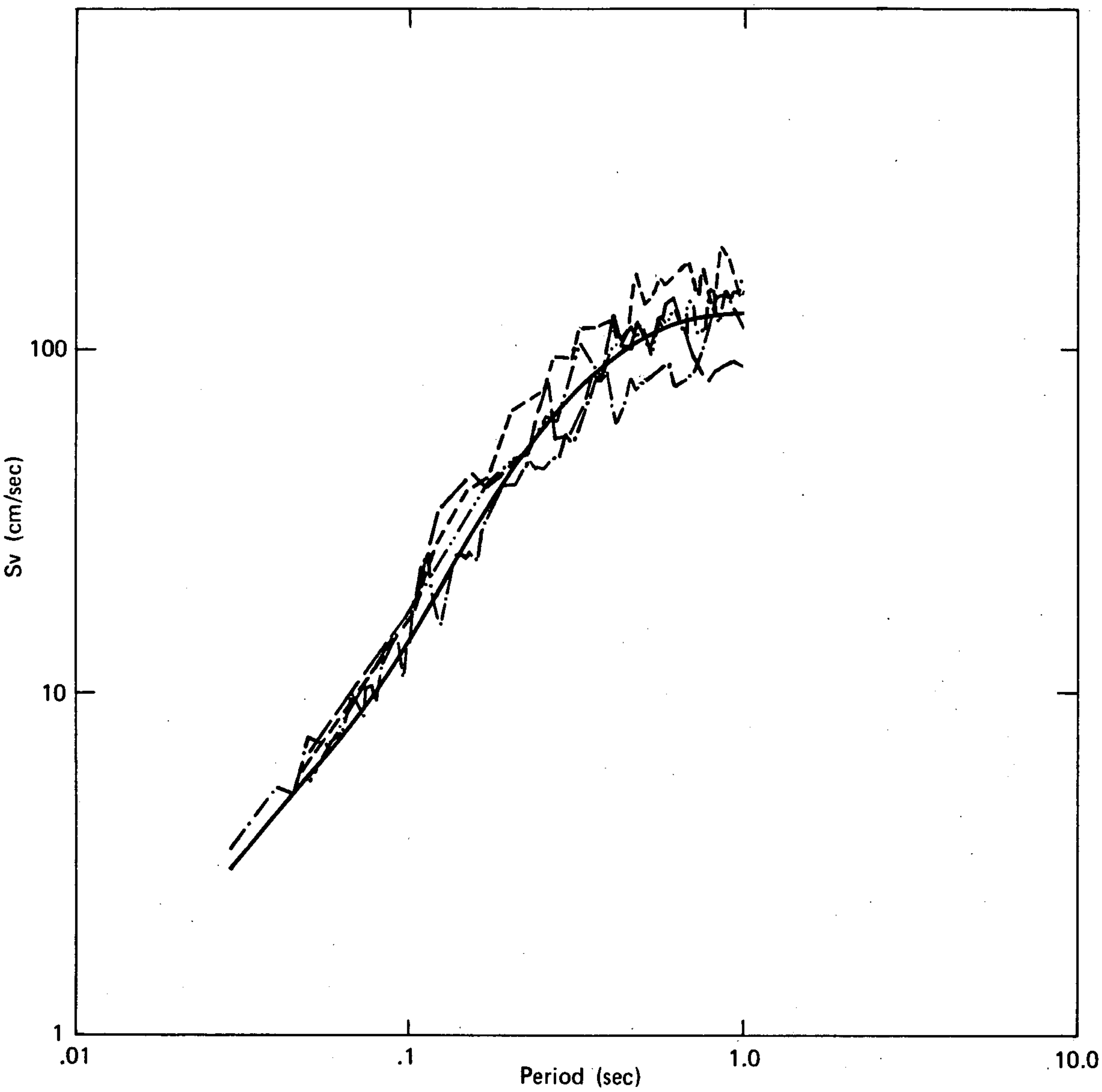
——— Housner (2/3 g)

— — — — WCC & TERA 84th Percentile Instrumental

2% Damping

Project: Project No.	SONGS UNIT 1 SEISMIC RE-EVALUATION 413521	HOUSNER (2/3 g) VS. WCC AND TERA INSTRUMENTAL	Fig. C-2

WOODWARD—CLYDE CONSULTANTS



LEGEND

2/3 g Housner

Trace A } Synthetic accelerograms, April 1977 report

Trace B } "Seismic Re-evaluation and Modification"

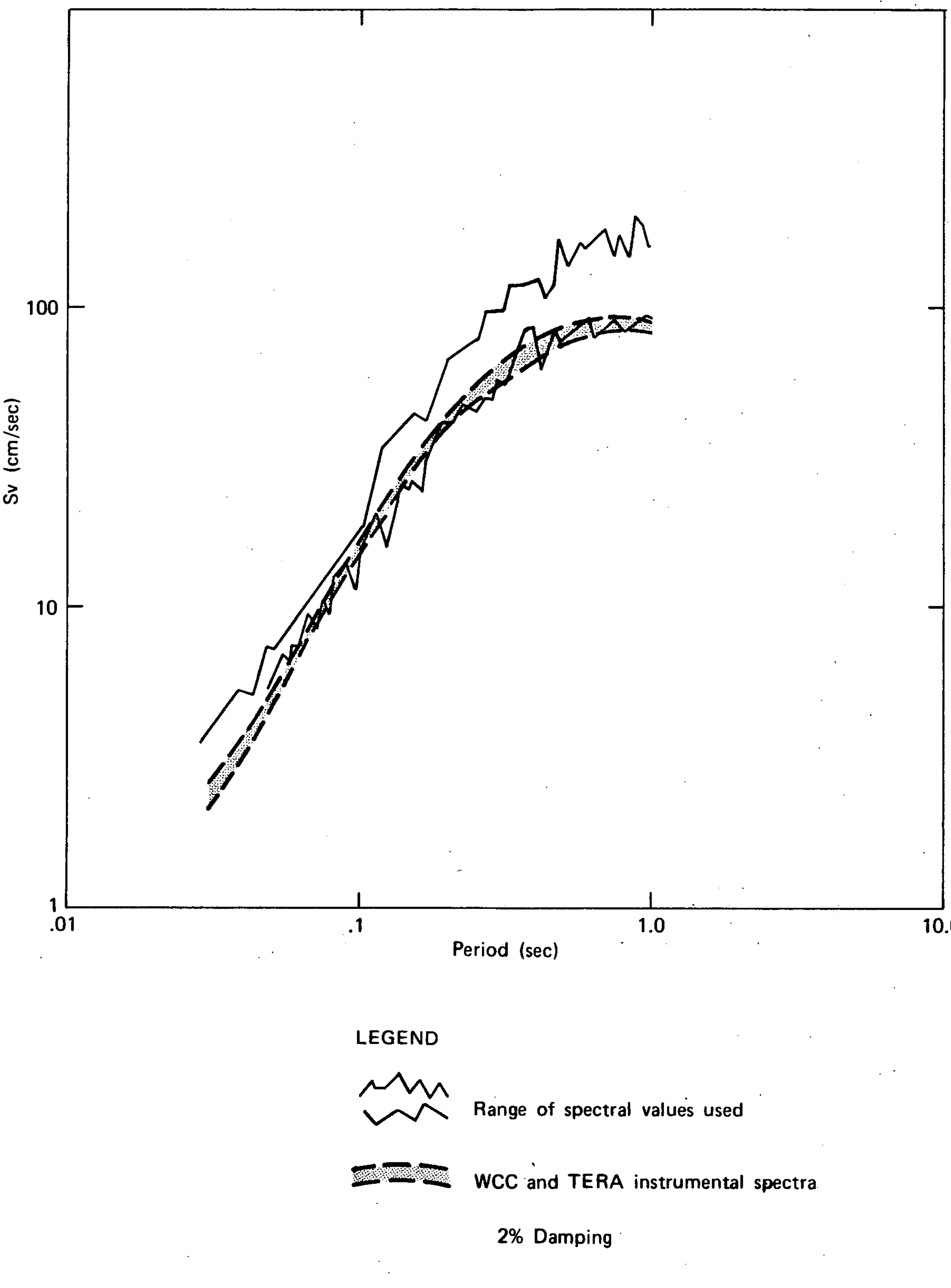
IV-79 Station 955, 2/3 g

IV-79 Station 5165, 2/3 g

2% Damping

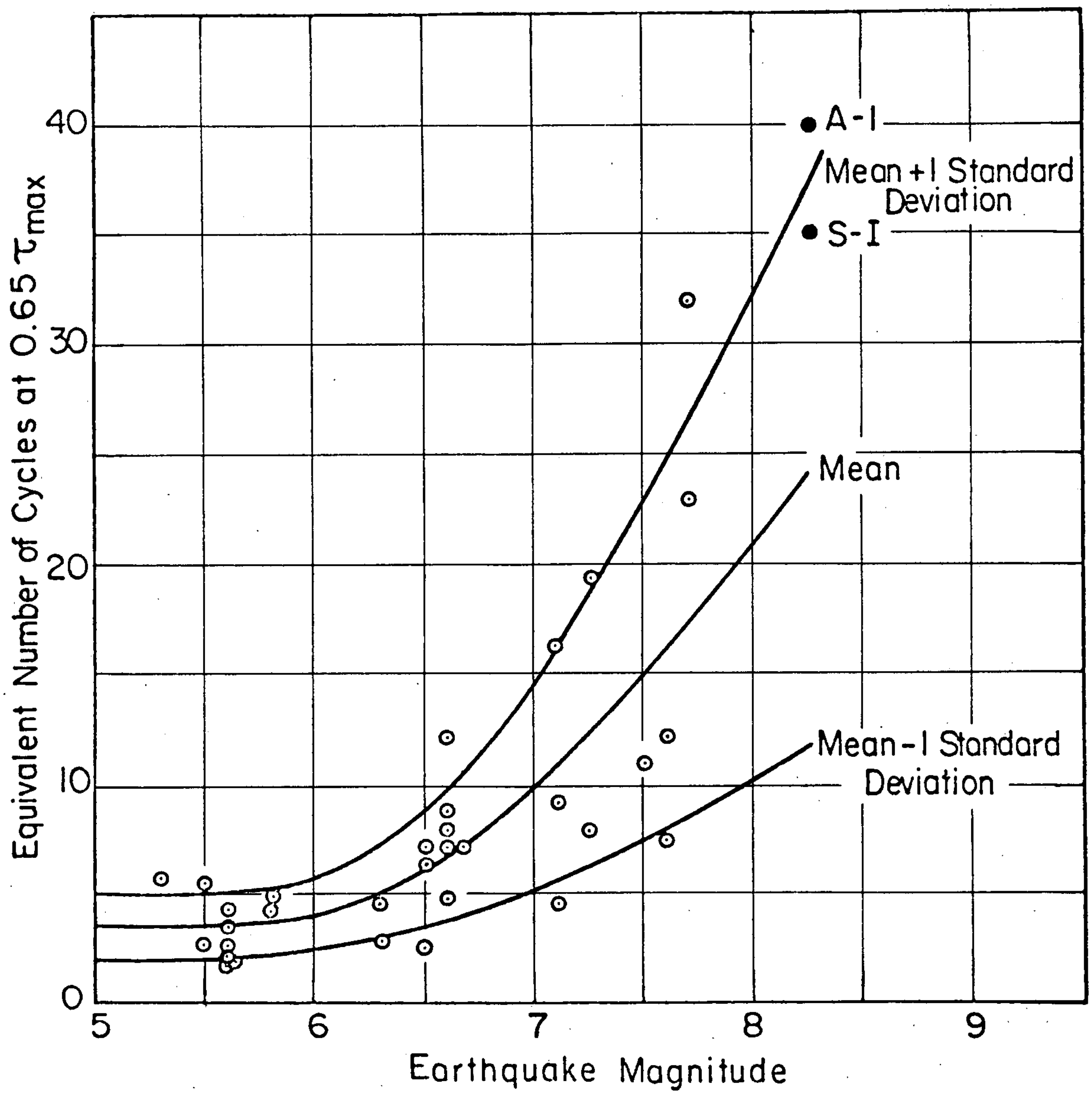
Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	COMPARISON OF SPECTRA USED IN ANALYSIS WITH 2/3 g HOUSNER	Fig. C-3

WOODWARD–CLYDE CONSULTANTS



Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	COMPARISON OF RANGE OF SPECTRA USED WITH INSTRUMENTAL SPECTRA	Fig. C-4

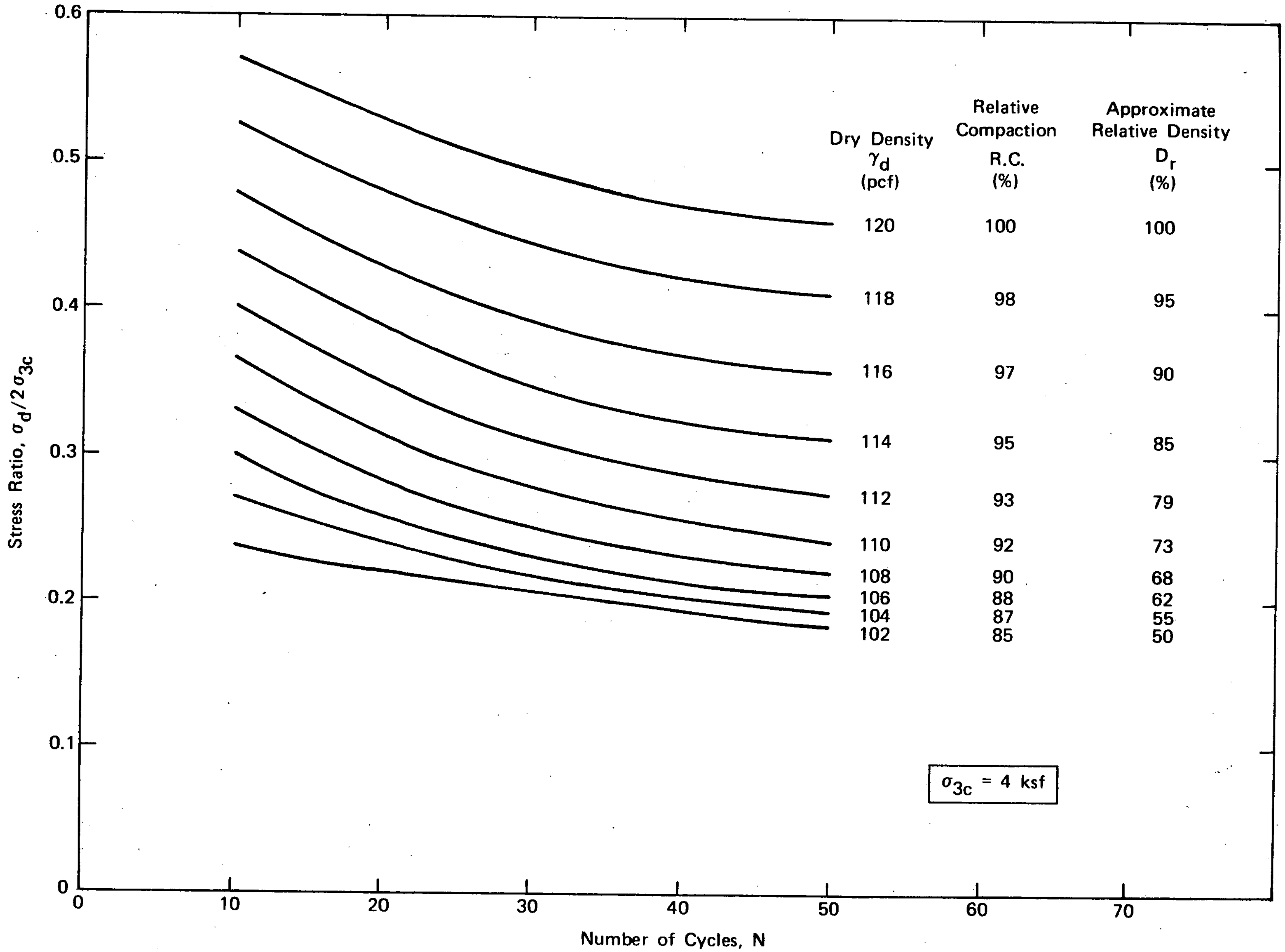
WOODWARD–CLYDE CONSULTANTS



After Seed and others, 1975 (Ref. C-8).

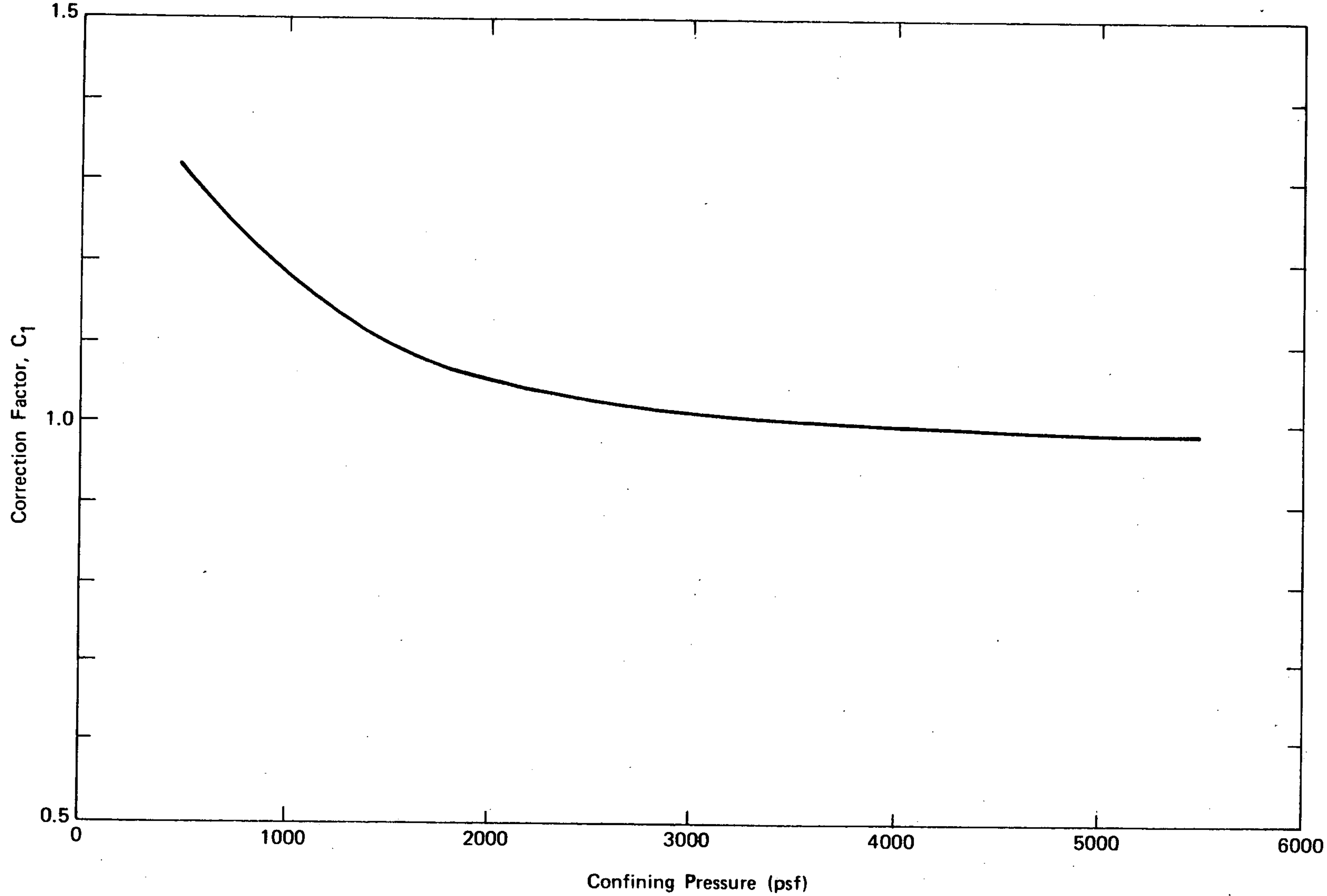
Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	EQUIVALENT NUMBERS OF UNIFORM STRESS CYCLES BASED ON STRONGEST COMPONENTS OF GROUND MOTION	Fig. C-5

WOODWARD–CLYDE CONSULTANTS



Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	STRESS RATIO, $\sigma_d/2\sigma_{3c}$ VS. NUMBER OF CYCLES, N_l	Fig. C-6

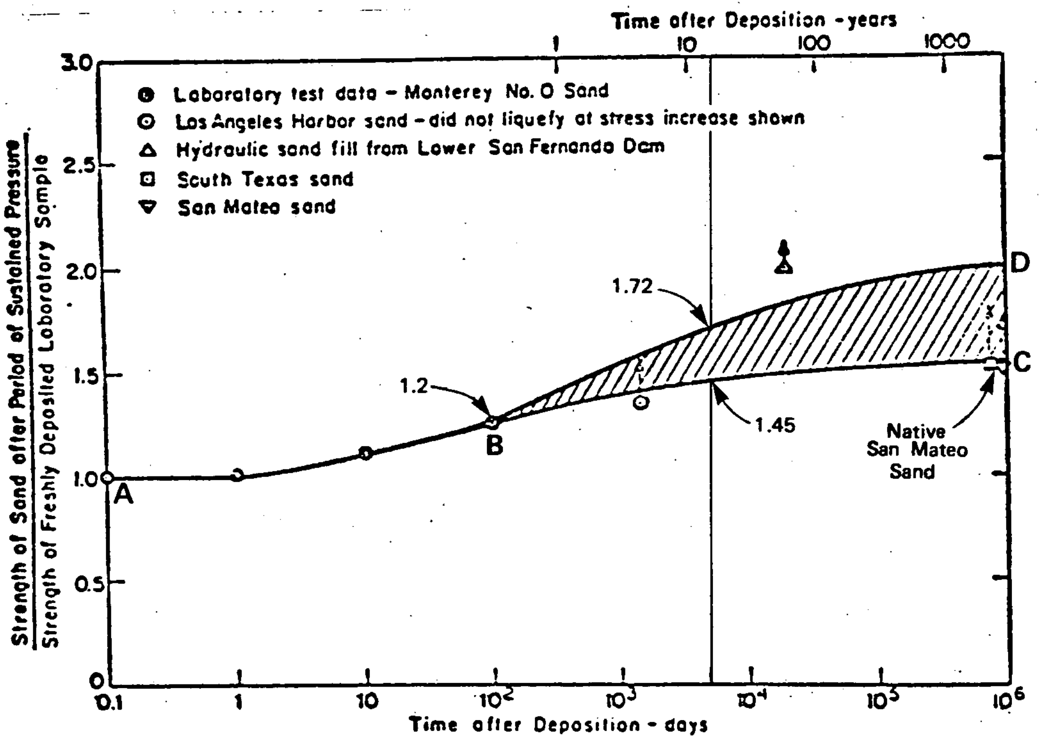
WOODWARD–CLYDE CONSULTANTS



NOTE: These data are based on test results on remolded San Mateo Sand. See Songs 2 and 3 FSAR, Appendix 2.5A.

Project: SONGS UNIT 1 SEISMIC RE-EVALUATION **Project No.** 413521	EFFECT OF CONFINING PRESSURE ON LABORATORY STRENGTH	**Fig.** C-7

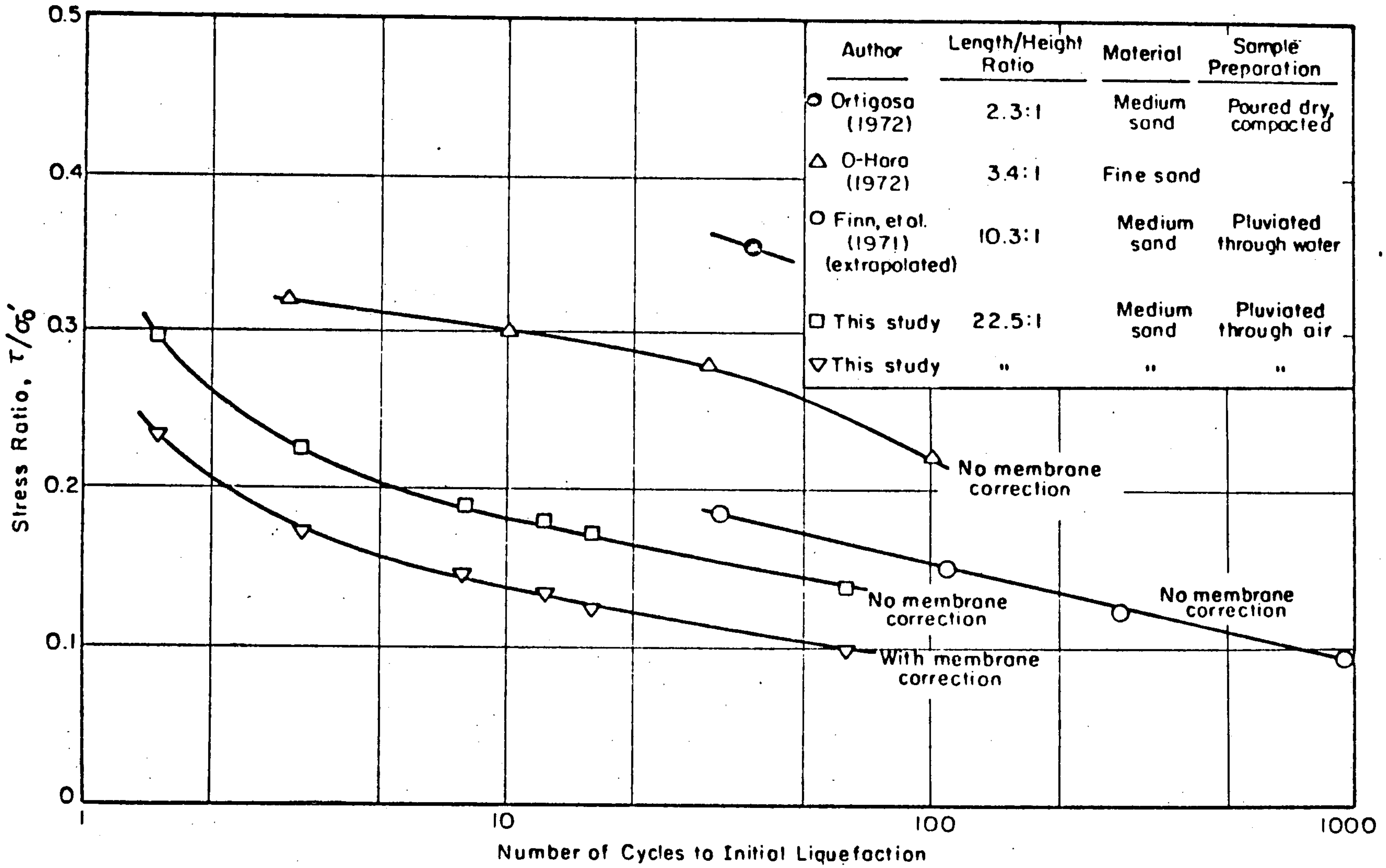
WOODWARD–CLYDE CONSULTANTS



After Seed, 1976 (Ref. C-6).

Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	INFLUENCE OF PERIOD OF SUSTAINED PRESSURE ON STRESS RATIO CAUSING INITIAL LIQUEFACTION	Fig. C-8

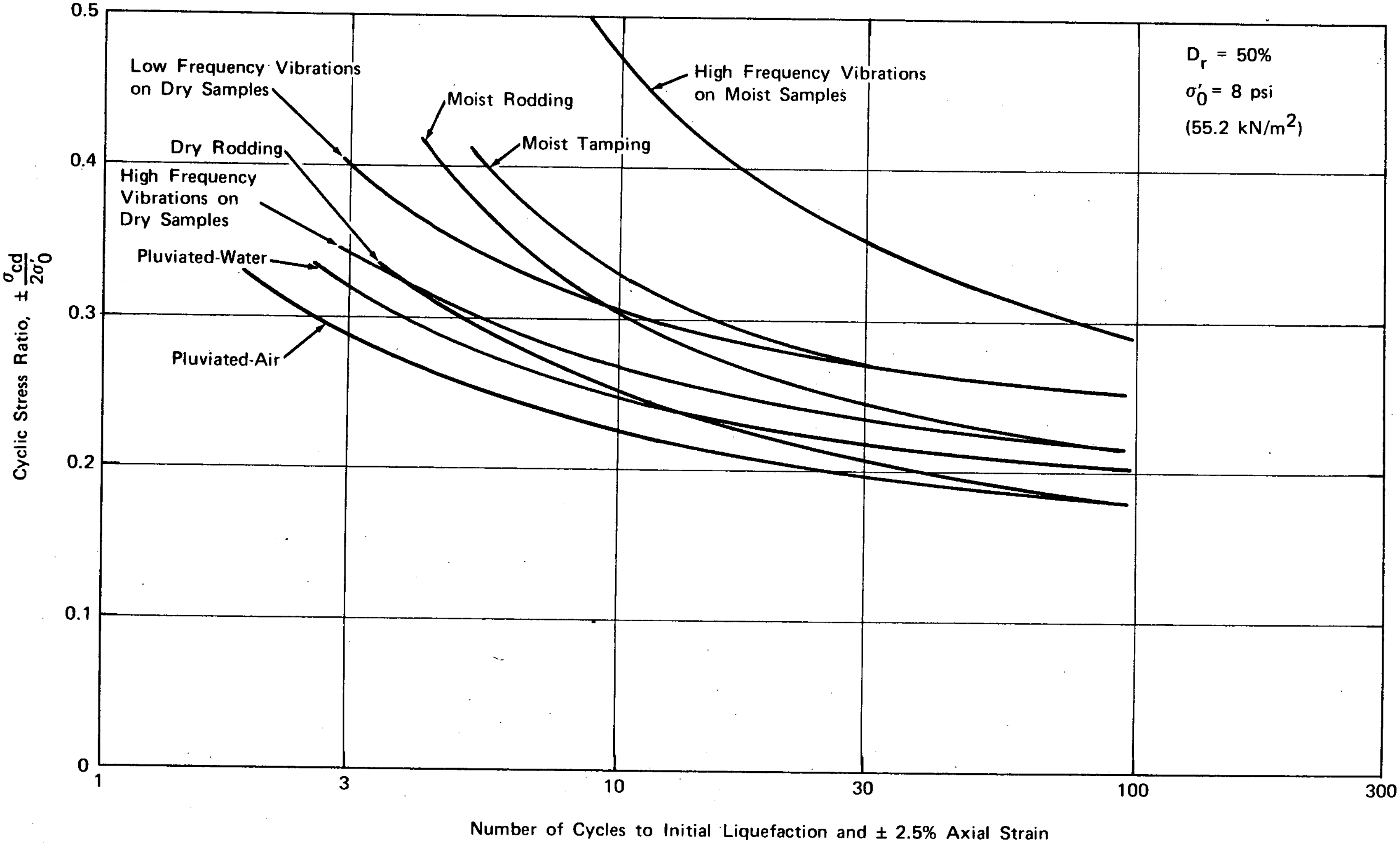
WOODWARD–CLYDE CONSULTANTS



After DeAlba, Seed and Chan, 1976 (Ref. C-1).

Project: Project No.	SONGS UNIT 1 SEISMIC RE-EVALUATION 413521	COMPARISON OF SHAKING TABLE TEST RESULTS FOR D_r = 50%	Fig. C-9

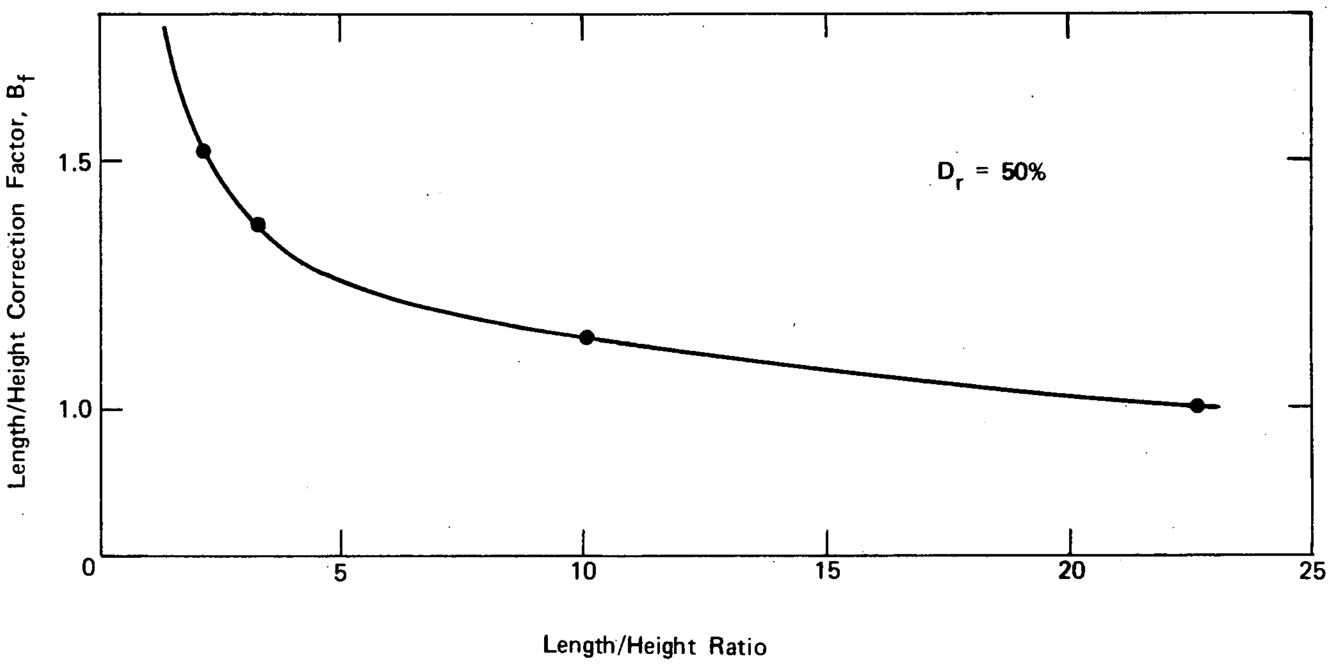
WOODWARD–CLYDE CONSULTANTS



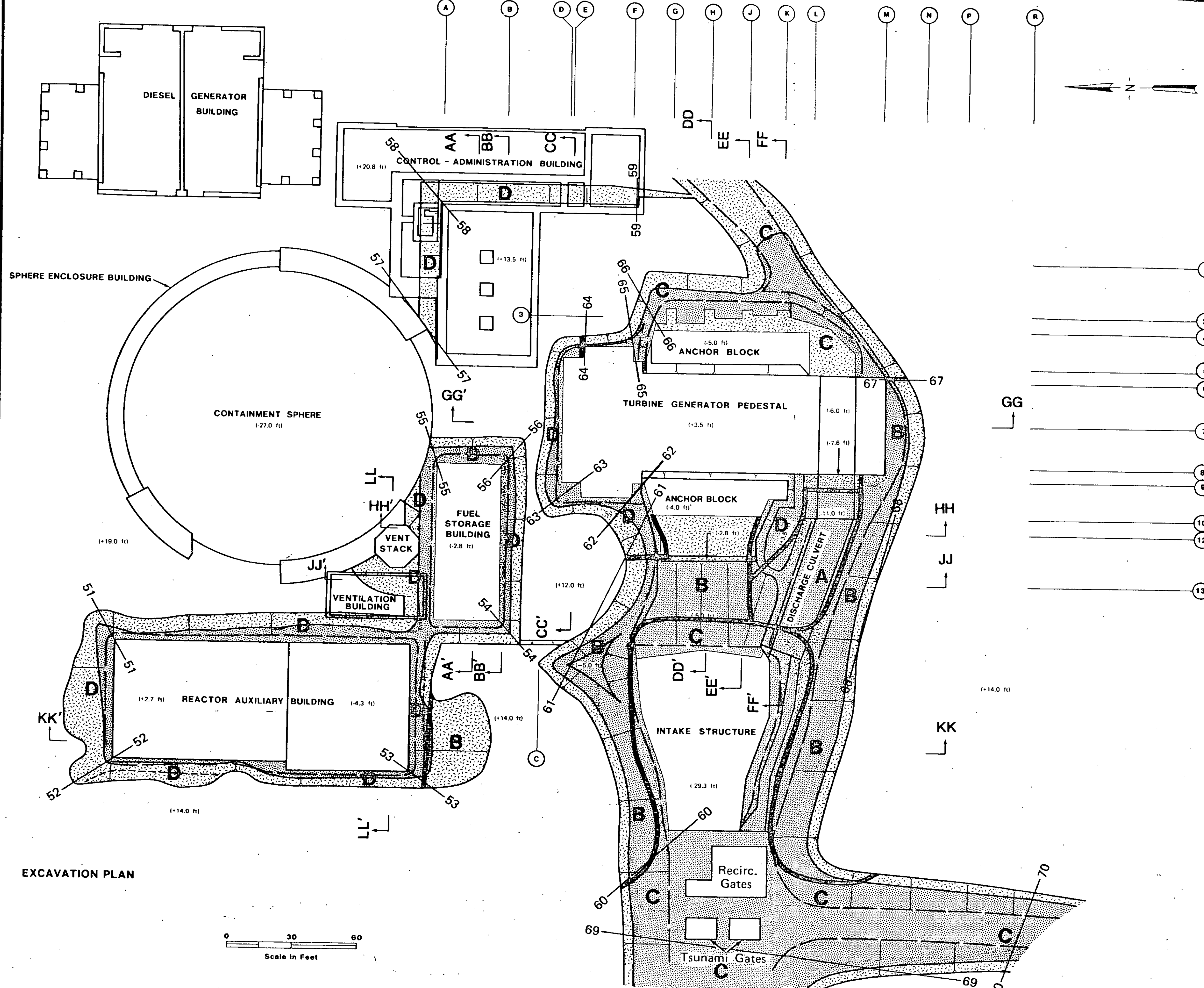
After Mulilis, Chan and Seed, 1975 (Ref. C-2).

Project: SONGS UNIT 1 SEISMIC RE-EVALUATION **Project No.** 413521	CYCLIC STRESS RATIO VS. NUMBER OF CYCLES FOR DIFFERENT COMPACTION PROCEDURES	**Fig.** C-10

WOODWARD–CLYDE CONSULTANTS



Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	CORRECTION FACTOR B_f VS. LENGTH AND HEIGHT RATIO	Fig. C-11



EXCAVATION PLAN

LEGEND

Top of Cut for Construction
Water Table
Estimated Toe of Cut Slope
Edge of Structure

Fill above water table

Fill below water table

HH——HH' Cross section locations (Figures 2-23 to 2-26, Section 2.0).

A Well compacted, ~95%.

B Moderately to well compacted, ~90 to 95%. Deep narrow fills with width <6 to 10 feet, ~85%.

C Moderately compacted, ~85 to 90%. Deep narrow fills with width <6 to 10 feet, ~85%.

D 85% relative compaction.

53———53 Identification line for backfill areas.

(+3.5 ft) Elevation of native San Mateo Sand at the bottom of the foundation.

— — Estimated toe of cut slope

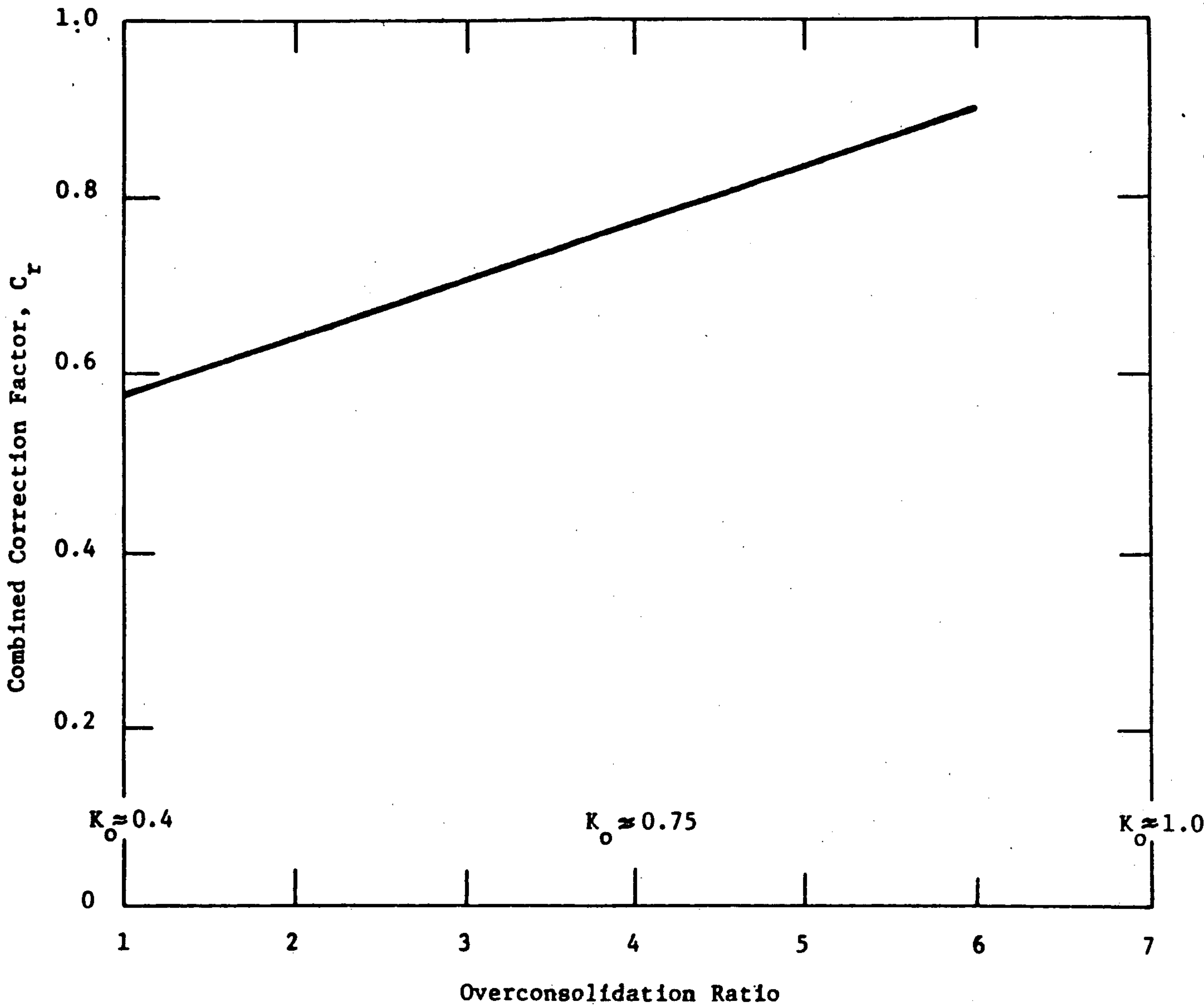
- - - - - Indicates break between flatter upper slope and steeper lower slope

Woodward-Clyde Consultants

SOIL BACKFILL CHARACTERIZATION AND IDENTIFICATION

Project No. 413521
SONGS UNIT 1
SEISMIC RE-EVALUATION

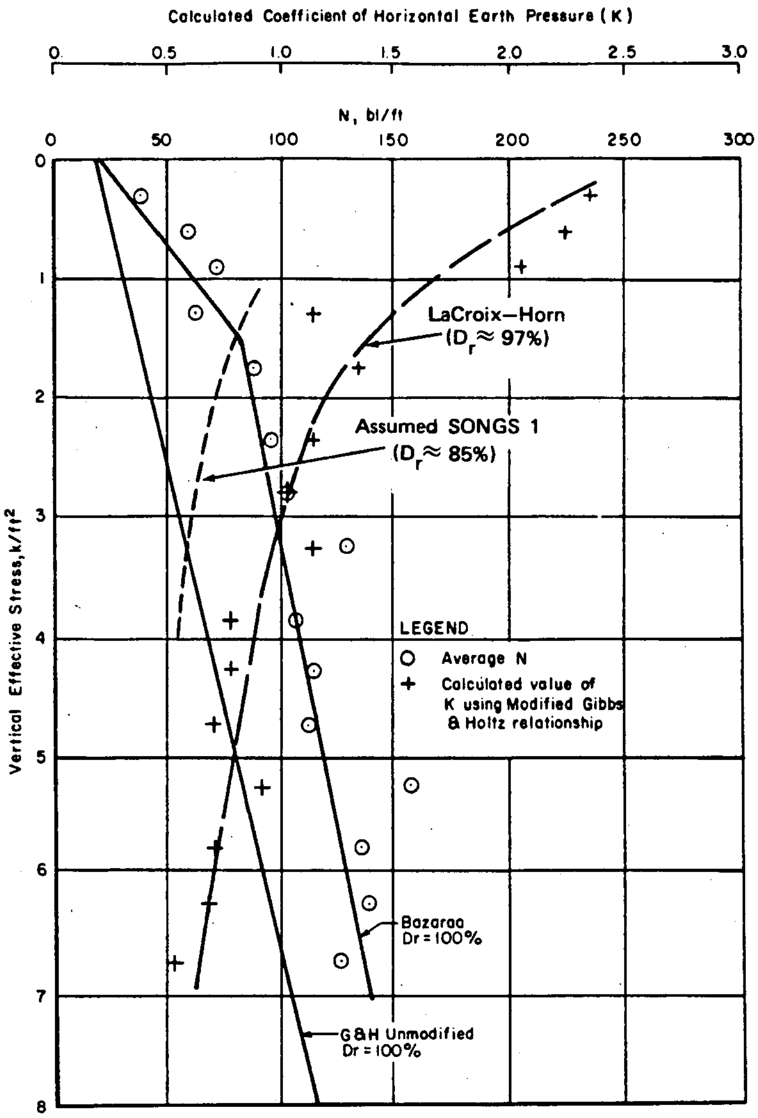
Fig. C-12



*After Seed, Arango and Chan, 1975 (Ref. C-8).

Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	COMBINED CORRECTION FACTOR FOR CYCLIC TRIAXIAL COMPRESSION TESTS ACCOUNTING FOR MULTIDIRECTIONAL SHAKING AND OVERCONSOLIDATION*	Fig. C-13

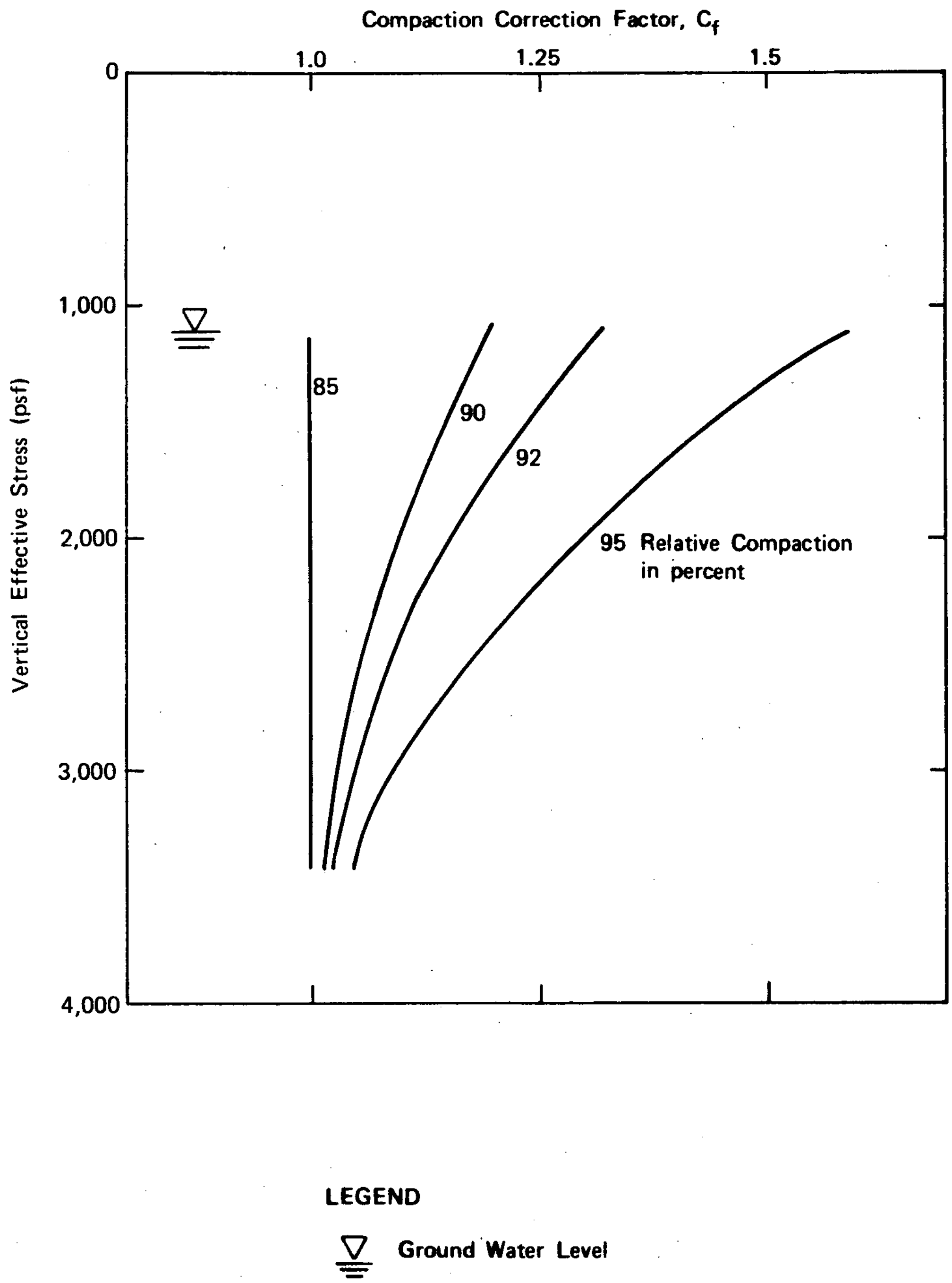
WOODWARD–CLYDE CONSULTANTS



After LaCroix and Horn, 1973 (Ref. C-10).

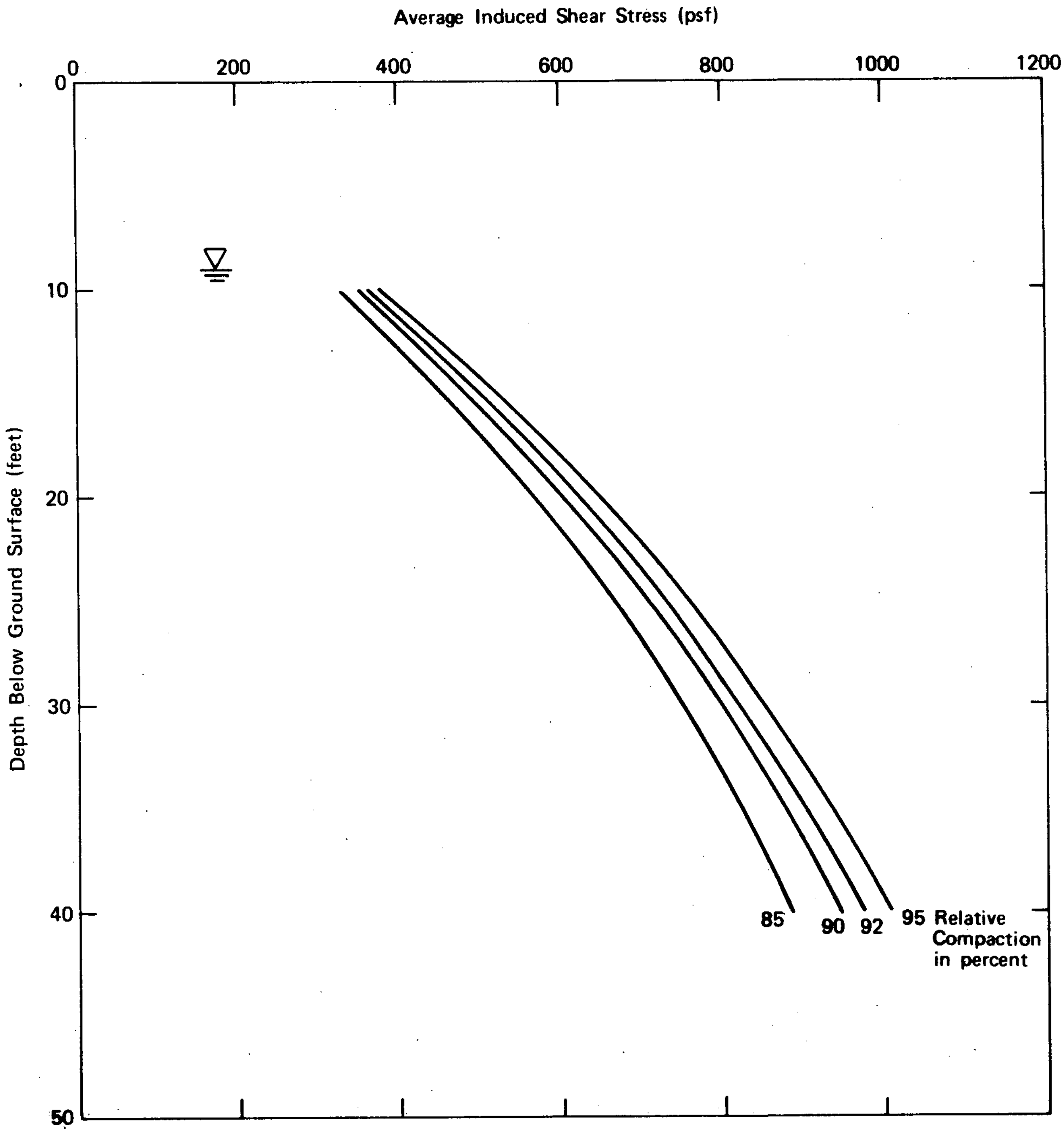
Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	VARIATION OF K_o WITH VERTICAL EFFECTIVE STRESS	Fig. C-14

WOODWARD—CLYDE CONSULTANTS



Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	ESTIMATED VARIATION OF COMPACTION CORRECTION FACTOR C_f WITH VERTICAL EFFECTIVE STRESS AS A FUNCTION OF RELATIVE COMPACTION	Fig. C-15

WOODWARD–CLYDE CONSULTANTS

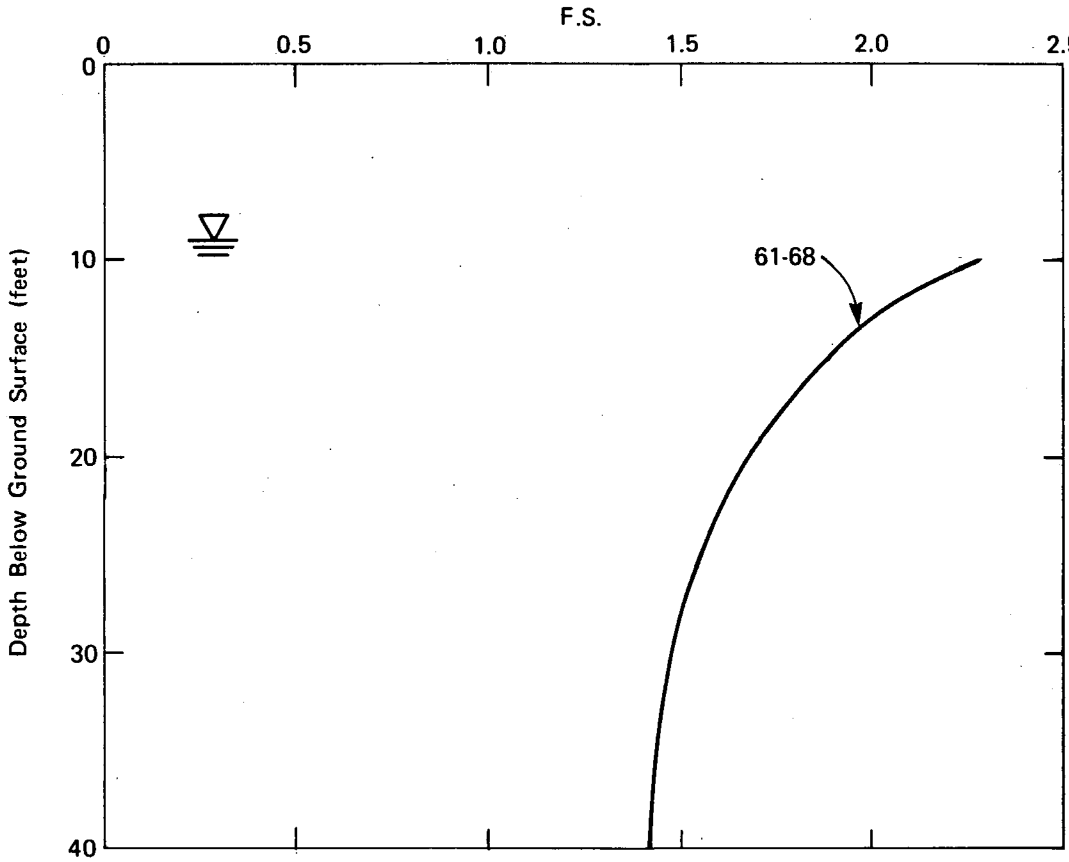


LEGEND

▽ Ground Water Level

Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	INDUCED SHEAR STRESS VS. DEPTH AS A FUNCTION OF RELATIVE COMPACTION	Fig. C-16

WOODWARD–CLYDE CONSULTANTS

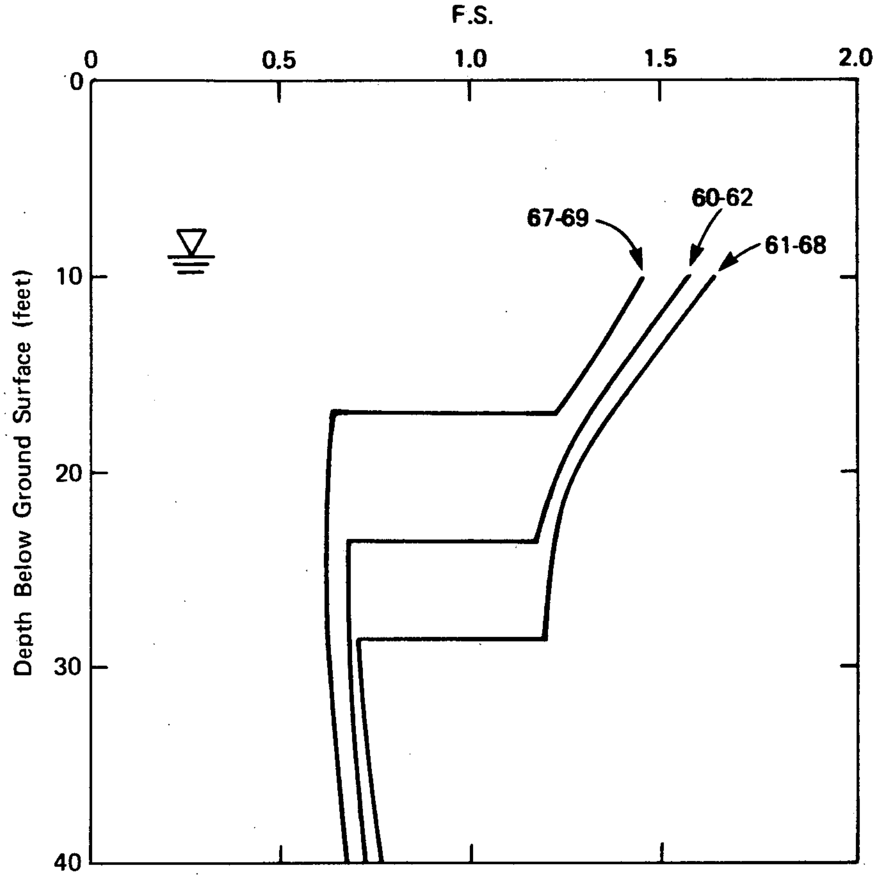


LEGEND

Ground Water Level

NOTE: The 61-68 backfill area is defined in Figure C-12.

Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	F.S. VS. DEPTH FOR CATEGORY A SOIL FILL	Fig. C-17



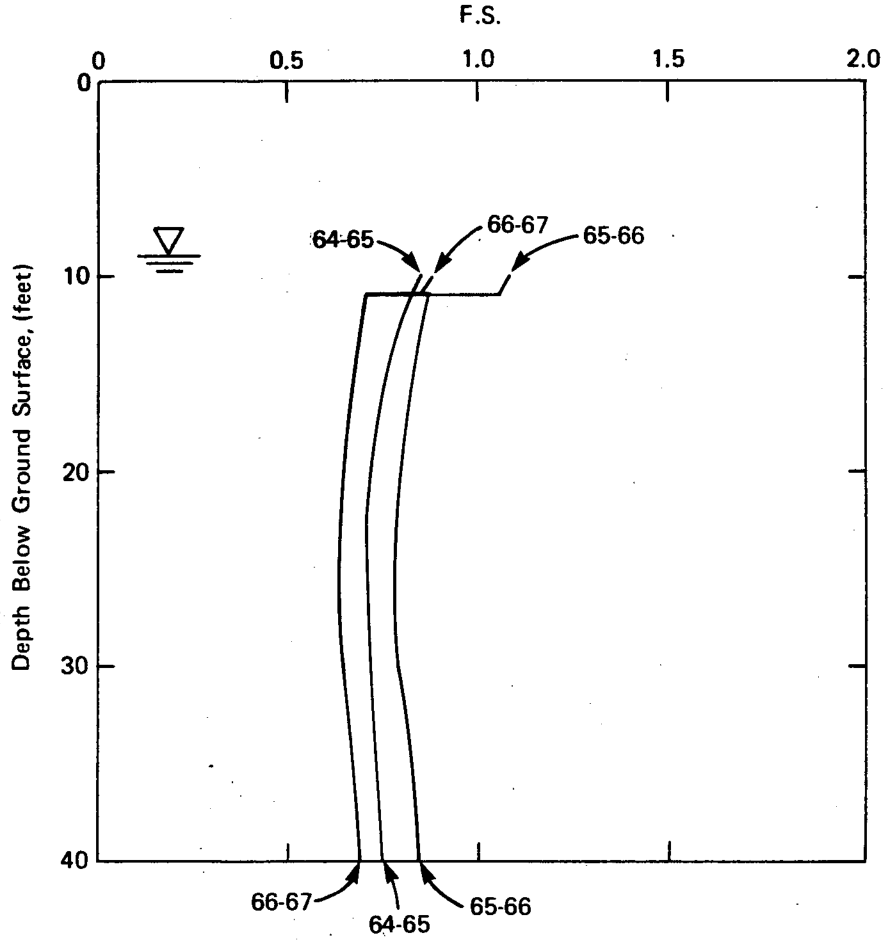
LEGEND

∇ Ground Water Level

NOTE: The backfill areas (e.g. 61- 68) are defined in Figure C-12.

Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	F.S. VS. DEPTH FOR CATEGORY B SOIL FILL	Fig. C-18

WOODWARD–CLYDE CONSULTANTS



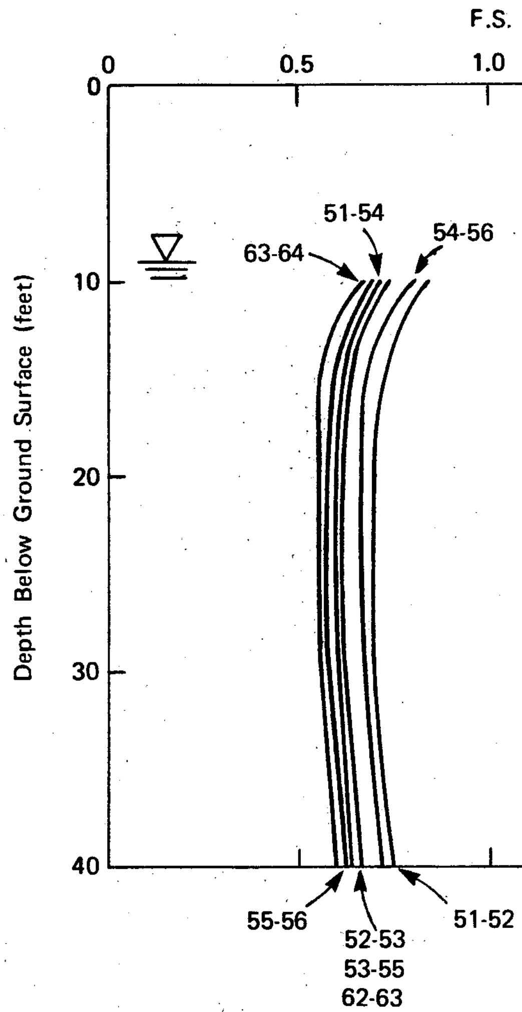
LEGEND

▽ Ground Water Level

NOTE: The backfill areas (e.g. 64-65) are defined in Figure C-12.

Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	F.S. VS. DEPTH FOR CATEGORY–C SOIL FILL	Fig. C-19

WOODWARD–CLYDE CONSULTANTS



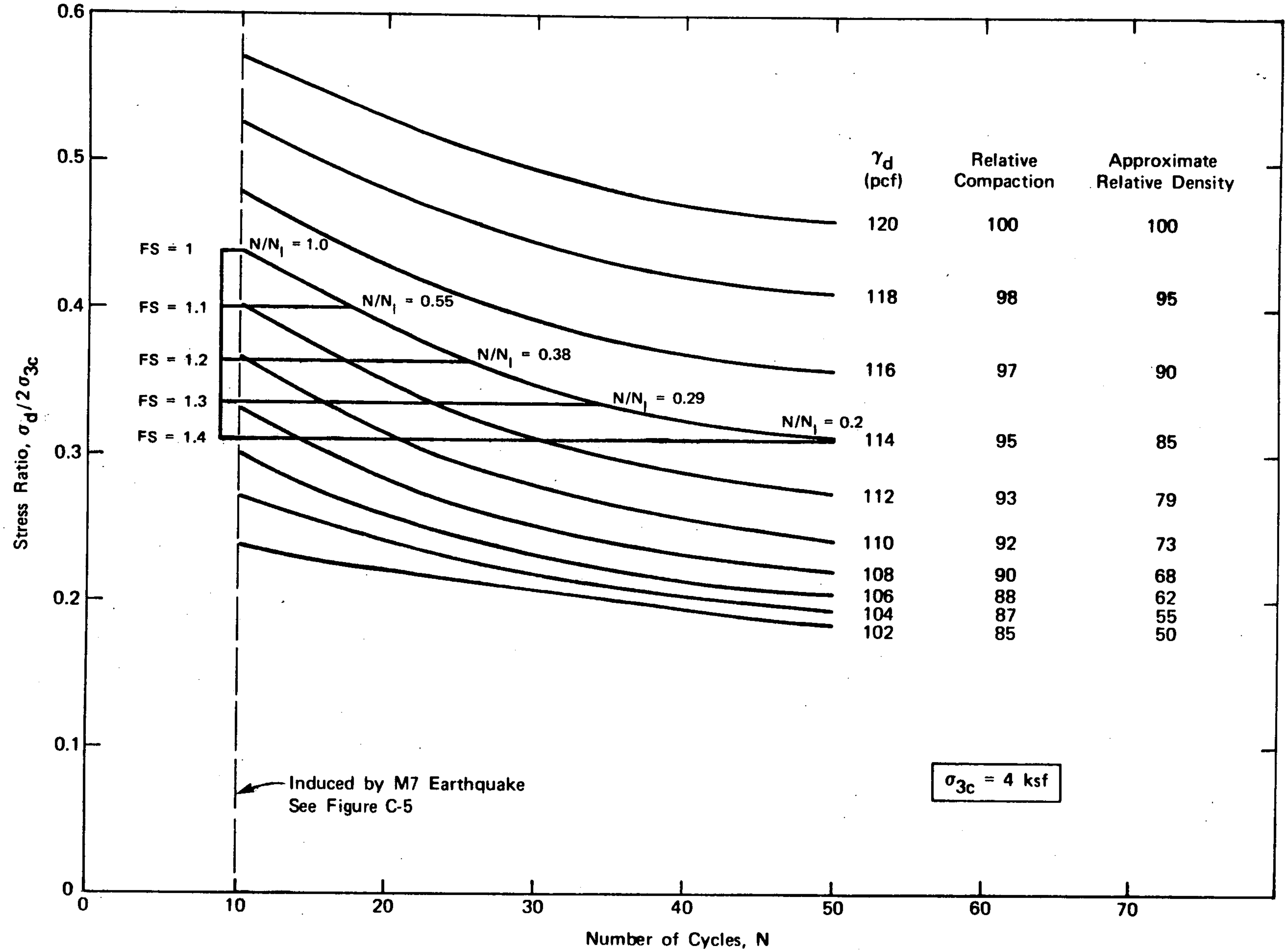
LEGEND

▽ Ground Water Level

NOTE: The backfill areas (e.g. 51-52) are defined in Figure C-12.

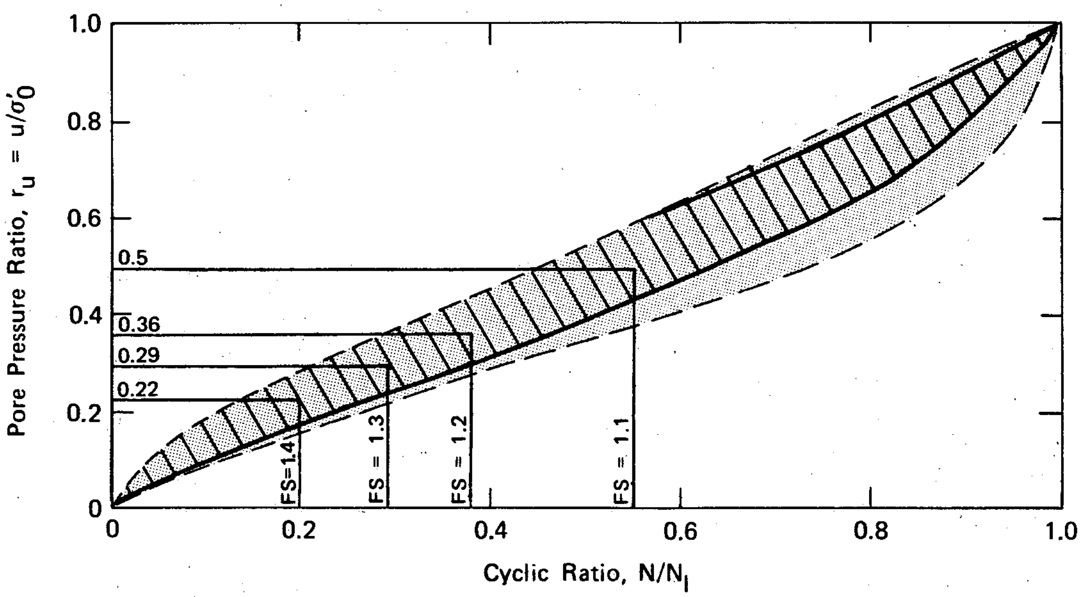
Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	F.S. VS. DEPTH FOR CATEGORY D SOIL FILL	Fig. C-20

WOODWARD–CLYDE CONSULTANTS



Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	STRESS RATIO, $\sigma_d/2\sigma_{3c}$ VS. NUMBER OF CYCLES, N	Fig. C-21

WOODWARD–CLYDE CONSULTANTS



LEGEND

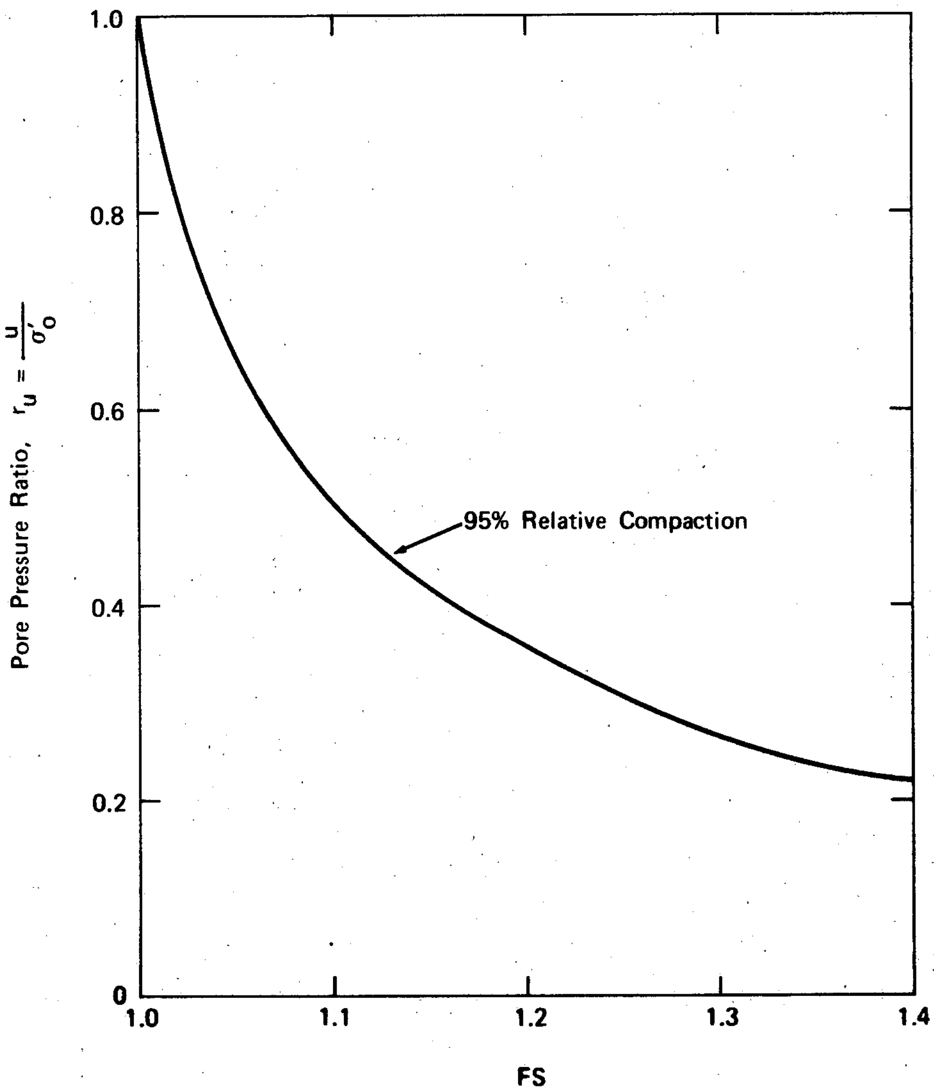
Range of laboratory test results (Ref. C-8). reported by Seed et al (1975)

From laboratory test results on San Mateo Sand

NOTE: Factors of safety and corresponding N/N_l are from Figure C-21 for relative compaction of 95%.

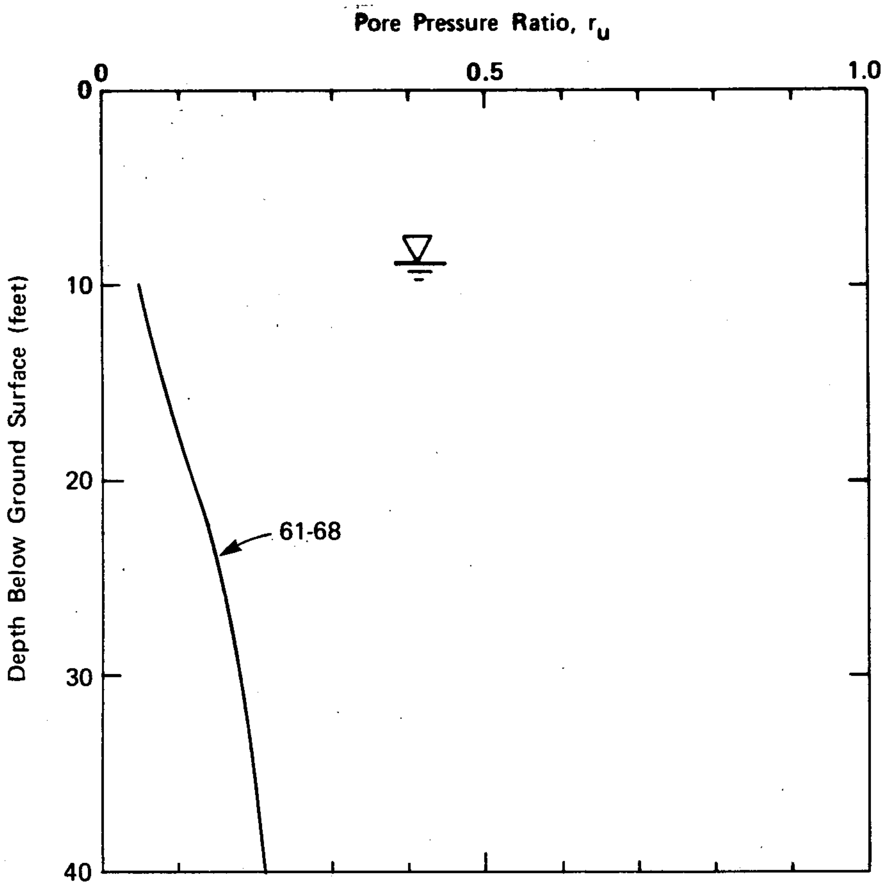
Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	PORE PRESSURE RATIO, $r_u = u/\sigma_o'$ VS. CYCLIC RATIO, $N/N_{liq.}$	Fig. C-22

WOODWARD–CLYDE CONSULTANTS



Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	R_u VS. FS	Fig. C-23

WOODWARD—CLYDE CONSULTANTS

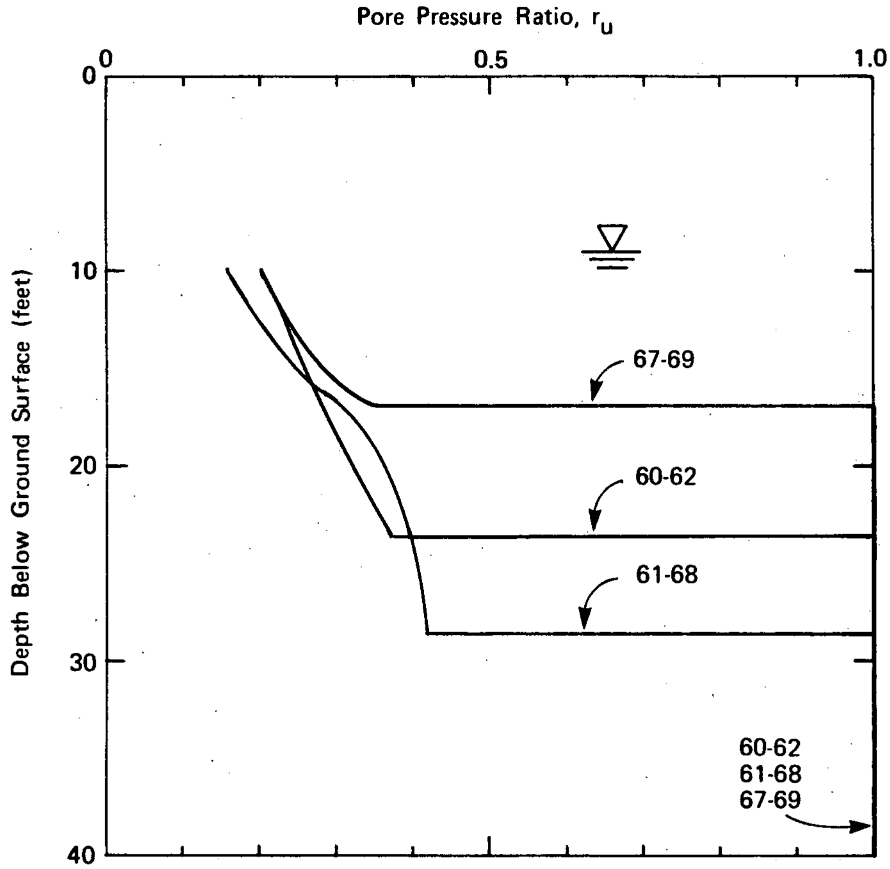


LEGEND

▽ Ground Water Level

NOTE: The 61-68 backfill area is defined in Figure C-12.

Project: Project No.	SONGS UNIT 1 SEISMIC RE-EVALUATION 413521	r_u VS. DEPTH FOR CATEGORY-A SOIL FILL	Fig. C-24

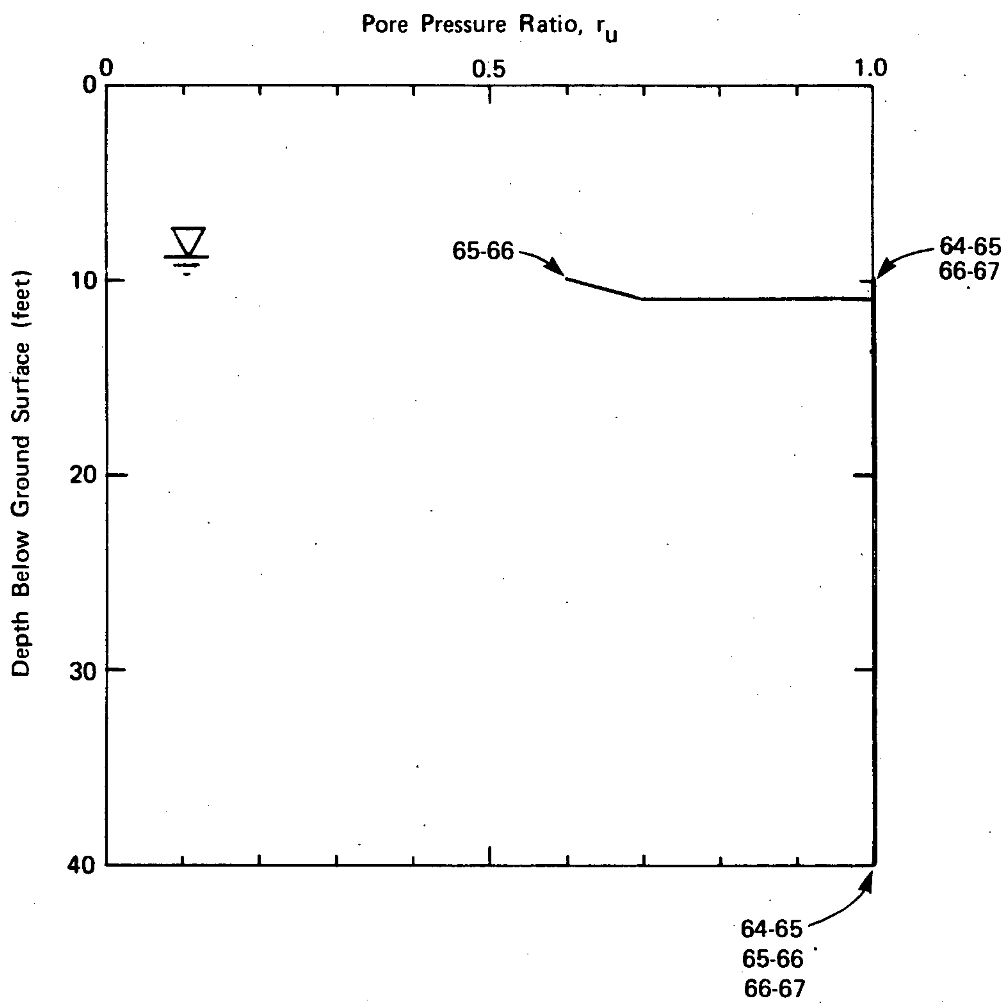


LEGEND

▽ Ground Water Level

NOTE: The backfill areas (e.g. 61-68) are defined in Figure C-12.

Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	r_u VS. DEPTH FOR CATEGORY-B SOIL FILL	Fig. C-25



LEGEND

▽ Ground Water Level

NOTE: The backfill areas (e.g. 65-66) are defined in Figure C-12.

Project: Project No. SONGS UNIT 1 SEISMIC RE-EVALUATION 413521	r_u VS. DEPTH FOR CATEGORY-C SOIL FILL	Fig. C-26

APPENDIX D

EXAMPLE: SEISMICALLY INDUCED SETTLEMENT CALCULATION

SONGS UNIT 1

D-1 INTRODUCTION

The calculation of settlement of the turbine plant cooler footing, as reported in the text of this report of soil backfill conditions at SONGS Unit 1, is presented here as an example of how seismically induced settlements were estimated. The general subsoil conditions for the turbine plant cooler footing located in Figure D-1 are shown in Figure D-2. The water table is at elevation +5 feet. The footing rests on backfill which has an average slope of 1/2:1 (horizontal to vertical) and extends to the base of the intake structure at elevation -29 feet. Locally, at the north end of the foundation, above elevation +6 ft to elevation +14 ft the slope flattens to a slope of about 2:1 as shown in the cross section in Figure D-2. The backfill has been conservatively characterized to have a relative compaction of 85 percent below the heavy dashed line in Figure D-2 where the width of backfill is 10 ft, i.e., at approximately elevation -14 ft, and to have an average relative compaction of 92 percent above elevation -14 feet. For computational purposes, to incorporate uncertainty in density, two cases have been identified: Case 1 accommodates the characterization of 92 percent relative compaction above elevation -14 ft and 85 percent relative compaction below elevation -14 ft; and Case 2 provides for 85 percent relative compaction for the entire soil profile. The evaluation of seismically induced settlement for this footing, as described below, provides for an upper bound settlement from Case 2 and a best estimate settlement from

Case 1. The example calculations are described in Sections D-2 and D-3 below. Specifically, Section D-2 provides example calculations for the turbine plant cooler foundation for below and above the water table. A discussion of how the final settlements presented in the text of this report were developed based on discussions with the consulting review board using the example calculations of settlement herein is presented in Section D-3.

D-2 EXAMPLE CALCULATION OF SETTLEMENT

Below the water table the seismically induced settlement is the consequence of the combined effects of soil compaction due to ground shaking and soil consolidation due to the dissipation of excess pore pressure generated under the seismic loading condition. Above the water table, settlement results from the soil compaction due to ground shaking. It is assumed that moist or partially saturated sands behave similarly to dry sands above the water table. The test data regarding the vertical and volumetric strain changes due to cyclic loading presented for dry sands by Silver and Seed (Reference D-3), and for saturated sands by Lee and Albaisa (Reference D-1), are used to calculate the settlements in this study.

The settlement calculation has been separated into five steps as follows: Step 1, calculation of the factor of safety for initial liquefaction for saturated sand; Step 2, determination of the induced pore pressure ratio; Step 3, determination of average seismically induced shear strain; Step 4, calculation of the vertical or volumetric strains; and, Step 5, calculation of the resulting settlement for the layer. For simplicity, the detailed discussion is focused at the 4-1/2 and 14-ft deep soil layers for Case 2.

<u>Step 1.</u> The factor of safety against liquefaction for saturated sands is calculated using the following equation:

$$F.S. = \frac{\dfrac{\sigma_d}{2\sigma_{3c}} \quad C_1 \; C_r \; A_f \; B_f \; C_f \; \sigma_v'}{\sigma_i}$$

where

$\dfrac{\sigma_d}{2\sigma_{3c}}$ = laboratory shear stress ratio to cause ± 5 percent strain for 10 uniform stress cycles from cyclic triaxial tests normalized to a confining pressure of 4 ksf (Figure D-3).

C_1 = correction factor for confining pressures other than 4 ksf (Figure D-4).

C_r = correction factor between cyclic traixial and simple shear test results related to K_o and the relative compaction (=0.57).

A_f = aging factor on strength (=1.2)

B_f = fill geometry corrective factor from Figures D-1 and D-5 and Table D-1.

C_f = compaction correction factor from Figure D-6.

σ_v' = effective overburden pressure.

τ_i = average induced shear stress from an M7 earthquake at 8 km (Figure D-7).

σ_d = cyclic deviator stress

σ_{3c} = initial consolidation pressure

For Case 2, the backfills have a relative compaction of 85 percent. The maximum dry density of backfill is 120 pcf.

Thus,

γ_d (dry unit weight) 120 x 85 percent = 102 pcf.

Based on experience at the site during foundation excavation, the moisture content of the soils above water table is about 8 percent. For a fully saturated soil with γ_d = 102 pcf and a specific gravity of 2.65 the moisture content (w_c) is found to be 23.5 percent as illustrated in Figure D-8.

Thus,

γ_T (total unit weight) = 102 x (1+0.08) = 110 pcf -- above the water table.

γ_T (total unit weight) 102 x (1+0.235) = 126 pcf -- below the water table.

γ_B (buoyant unit weight) = 126 - 64 (unit weight of sea water) = 62 pcf.

Therefore, at the depth of 14 ft

σ'_v (effective overburden presure) = 110 x 9 + 5 x 62 = 1300 pcf.

From Figure D-3 the stress ratio $\sigma_d/2\sigma_{3c}$ is 0.24 for 85 percent relative compaction at N = 10 cycles. Other parameters are determined as follows:

C_r = 0.57, for K_o = 0.4

C_l = 1.12, at σ_v' = 1300 psf from Figure D-4.

A_f = 1.2, a constant (Appendix C of this report).

B_f = 1.3 (based on the location of footing from Figure D-1, the length to height ratio from Table D-1, and the curve presented in Figure D-5).

C_f = 1.0 as indicated in Figure D-6 for σ_v' = 1300 psf and relative compaction = 85 percent.

τ_i = 435 psf as shown in Figure D-7 for depth = 14 ft and relative compaction = 85 percent.

The factor of safety (F.S.) is determined using these parameters and the above equation as follows:

$$F.S. = \frac{.24 \times 1.12 \times .57 \times 1.2 \times 1.3 \times 1.0 \times 1300}{435}$$

$$F.S. = 0.71$$

The same procedures are applied to various depths in the soil profile for Cases 1 and 2 and the results are summarized in Table D-2.

Step 2. Pore Pressure Ratio (r_u):

From Table D-2 at the depth of 14 ft the γ_d = 102 pcf and F.S. = 0.71 $\leq$1.0, Figure D-9 yields N_ℓ < 10 cycles. Therefore N/N_ℓ > 1.0, and from Figure D-10, r_u = 1.0. Other values of N_{ℓ} are developed in Figures D-9 and D-11 from which r_u values summarized in Table D-3 are developed from Figure D-10.

Step 3. Average Induced Shear Strain

From Appendix F, the average induced major principal strain in near-surface soils was found to be 0.2 percent for a relative compaction of 95 percent or greater. This corresponds to an average induced shear strain of 0.27 percent for a Poisson's Ratio of 0.35. For lower densities (to 85 percent relative compaction) the induced shear strain is calculated to be up to 50 percent higher (0.4 percent) than that for 95 percent or greater relative compaction because the differences in moduli are not completely offset by the differences in induced stress for the various densities (Appendix F). By linear interpolation 0.27, 0.36, and 0.4 percent induced shear strain are used for the settlement calculations for soils compacted to 95, 92, and 85 percent average relative compaction, respectively.

Step 4. Volumetric Strain

The volumetric strain is evaluated considering the reconsolidation resulting from seismically induced pore-water pressure and compaction due to disturbance of the grain structure. For a relative compaction of 85 percent and r_u = 1.0 from Step 2, Figure D-12 is used to obtain the volumetric strain. As shown in Figure D-12, the volumetric strain, ε_v, is found to be 1.5 percent. For cases of r_u < 1, the graphs in Figure D-13 should be used to develop the volumetric strain. For r_u = 1.0, the graphs in Figure D-13 yield lower volumetric strains than those in Figure D-12, because Figure D-12 provides for the effects of prolonged cycling after initial liquefaction while Figure D-13 does not. Because of the uncertainty in analysis and because high values of r_u above 0.6 to 0.8 may cause large changes in volume compressibility (Reference D-2), values of r_u above 0.6 are conservatively treated as if $r_u = 1.0$, and the curves in Figure D-12 are used to develop volumetric strain. For $r_u < 0.6$ the values of volumetric strain from Figure D-13 are used. Also, for dry sands or for sands where $r_u < 0.6$ the volumetric strain developed using the procedure suggested by Silver and Seed (1972) discussed below was used. For saturated sands with $r_u < 0.6$ the volumetric strain is calculated from Figure D-13 and from the Silver and Seed (Reference D-3) procedure, and whichever volumetric strain is the greater is used in analysis of settlement.

The vertical volumetric strain has been related to the shear strain by Silver and Seed (Reference D-3). For a shear strain of 0.27 percent from Step 3 and N = 10 corresponding to earthquake magnitude of 7 from Figure D-14, the vertical strains at relative densities of 80, 60, and 45 percent are 0.2, 0.55, and 0.95 percent, respectively, as shown in Figure D-15. These results together with those for shear strains of 0.36 and 0.4 percent are plotted in Figure D-16 to facilitate interpolation at other relative densities. As indicated in Figure D-16 $\varepsilon_v = 0.36$ percent at $D_r = 73$ percent (i.e., at R.C. = 92 percent) and $\varepsilon_v = 1.02$ percent at $D_r = 50$ percent (ie., at R.C. = 85 percent).

<u>Step 5.</u> Estimate Settlement:

The settlement, Δh, is found from:

$$\Delta h = \varepsilon_v \times h$$

where

ε_v = volumetric or vertical strain calculated by the procedures described in Step 4

and

h = the thickness of layer.

Therefore, for the soil layer at 4-1/2 ft for Case 2 (Figure D-2):

ε_v = 1.02 percent from Figure D-16

$\Delta h = 1.02 \times 9 \times 12/100$

$= 1.10$ inches

and for the soil layer at 14 ft for Case 2 (Figure D-2):

ε_v = 1.5 percent from Figure D-12 and 1.02 percent from Figure D-16; using 1.5 percent

$\Delta h = 1.5 \times 10 \times 12/100$

$= 1.8$ inches

The same general procedures in Steps 2 to 5 are applied to various depths for Cases 1 and 2 and the resulting summary of settlements calculated for the soil profile is shown in Table D-3.

D-3 DEVELOPMENT OF SETTLEMENT ESTIMATE

The settlement estimate developed in Table D-3 for the Turbine Cooler Footing ranges from a best estimate of 3.9 inches to an extreme value of 7.2 inches. Based on discussions with the review board comprised of Drs. I. M. Idriss, R. L. McNeill and H. B. Seed, a range of settlements of 3 to 5 inches was considered conservative. This was based on the conclusion that the Case 1 characterization was considered more credible taking into account the following observations: 1) there have been no obvious surface settlements observed during the life of the project (settlement would be expected if the average relative compaction was 85 percent); 2) it is unlikely that a fill having the configuration indicated on Figure D-1 should have an average relative density as low as 50 percent (85 percent relative compaction) especially considering the construction access in the wider portion of the fill above elevation -14 ft; and 3) the fact that two field density tests (having 92 and 96 percent relative compaction) were taken in this general area at elevation -10 ft indicate an attempt to attain some level of compaction at an elevation where access of compaction equipment was not restricted. The range of 3 to 5 inches also provides for uncertainty on the factors used as well as some uncertainty in the relative compaction.

REFERENCES

D-1 Lee, K. L., and Albaisa, A., 1974, "Earthquake Induced Settlements in Saturated Sands", Journal of the Geotechnical Engineering Division, ASCE, v. 100, no. GT4, April.

D-2 Seed, H. B., Martin, P. M., and Lysmer, J., 1975, "The Generation and Dissipation of Pore Water Pressures During Soil Liquefaction", Report No. EERC 75-26, Earthquake Engineering Research Center, August.

D-3 Silver, M. L., and Seed, H. B., 1972, "Settlement of Dry Sands During Earthquakes", Journal of the Soil Machanics and Foundation Division, ASCE, v. 98, no. SM4, April.

D-4 Seed, H. B., Idriss, I. M., Makdisi, F., and Banerjee, N., "Repression of Irregular Stress Time Histories by Equivalent Uniform Stress Series in Liquefaction Analysis", Report No. EERC 75-29, Earthquake Engineering Research Center, University of California; Berkeley, California; 1975.

TABLE D-1

LENGTH/HEIGHT RATIOS OF BACKFILL AREAS

Area	Location	Soil Category(1)	L/H(2)	B_f(3)
51-52	North of Auxiliary Bldg.	D	3-3/4	1.32
52-53	West of Auxiliary Bldg.	D	8	1.18
51-54	East of Auxiliary Bldg.	D	10	1.15
53-55	South of Auxiliary Bldg. and North of Fuel Storage	D	9	1.17
55-56	East of Fuel Storage	D	12-1/2	1.10
54-56	South of Fuel Storage	D	5	1.27
57-58		D	-------------	(4)
58-59		D	-------------	(4)
60-62	North of Pump Well and West Footing (example case)	B	4	1.30
62-63	West of Turbine Generator and North of West Anchor Block	D	8	1.18
63-64	North of Turbine Generator	D	17	1.06
64-65	East of Turbine Generator and North of East Anchor Block	C	8	1.18
65-66	North of East Anchor Block	C	1.6	1.50
66-67	East of East Anchor Block	C	6	1.22
67-69	South of Anchor Blocks, Turbine Generator and Pump Well	B, C	7	1.20
61-68	East of Pump Well and West of West Anchor Block	A, B, C	3-1/3	1.35
60-70	West of Pump Well	C	wide open area	1.0

(1) See Figure D-1.

(2) Approximate length of fill to depth of fill ratio.

(3) Fill geometry correction factor.

(4) All soil above water table.

TABLE D-2

SUMMARY OF LIQUEFACTION F.S. CALCULATIONS

Layer Depth*	Relative Compaction (%)	γ_d	w_c	γ_T	γ_B	σ_v'	$\sigma_d/2\sigma_{3c}$	C_r	C_1	A_f	B_f	C_f	τ_i	F.S.
Case 1														
14	92	110.4	19.2	131.6	67.6	1411.1	.37	.57	1.11	1.2	1.3	1.26	480.	1.35
24	92	110.4	19.2	131.6	67.6	2515.1	.37	.57	1.03	1.2	1.3	1.09	710.	1.16
34	85	102.0	23.5	126.0	62.0	3157.5	.24	.57	1.02	1.2	1.3	1.00	900.	0.76
Case 2														
14	85	102.0	23.5	126.0	62.0	1301.3	.24	.57	1.12	1.2	1.3	1.0	435.	0.71
24	85	102.0	23.5	126.0	62.0	1921.0	.24	.57	1.06	1.2	1.3	1.0	645.	0.67
34	85	102.0	23.5	126.0	62.0	2540.7	.24	.57	1.03	1.2	1.3	1.0	805.	0.69

* Depth to center of layer, See Figure D-2.

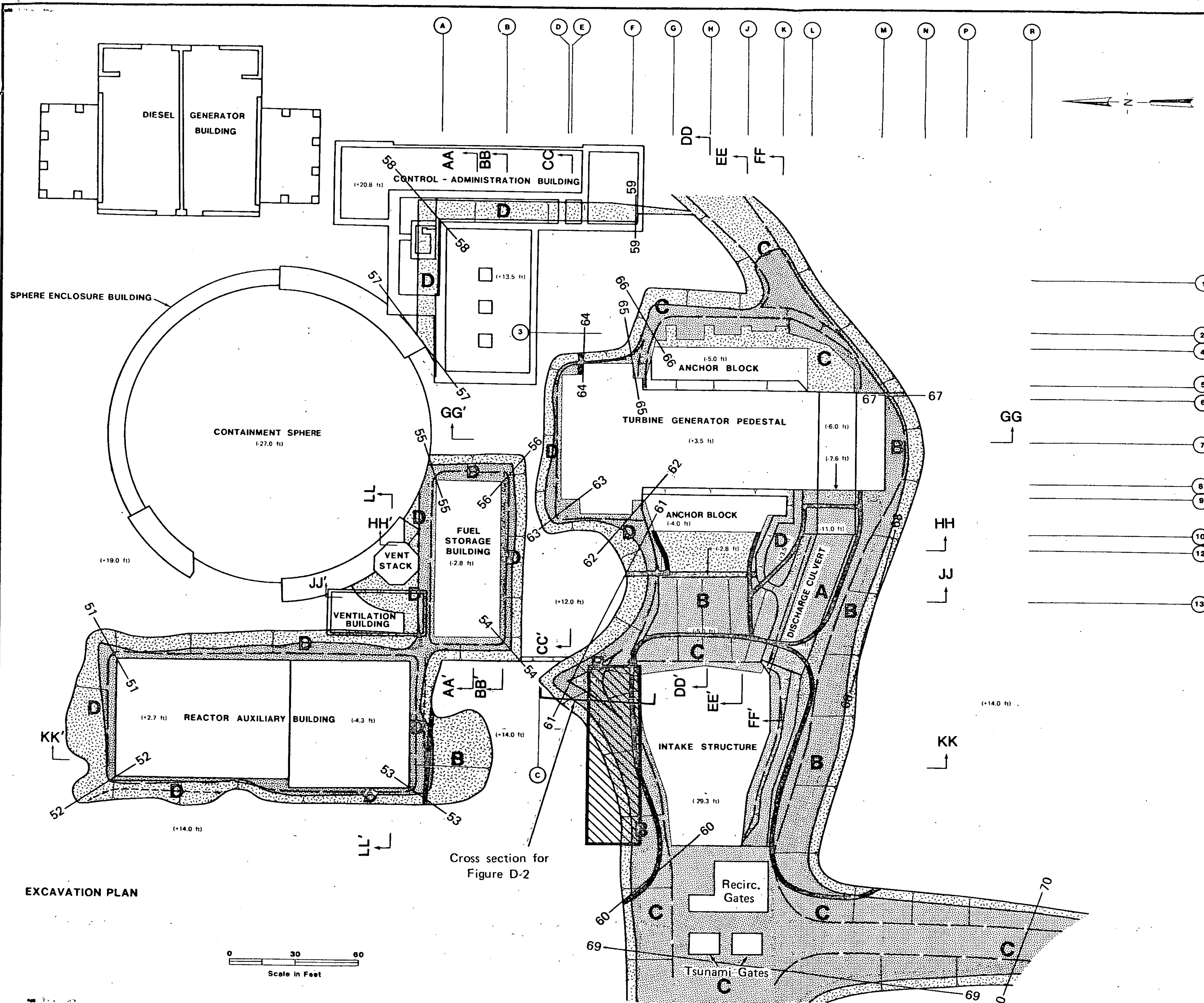
TABLE D-3

SUMMARY OF SETTLEMENT RESULTS FOR SOIL BELOW WATER TABLE

Layer Depth (ft)	F.S.	N_ℓ*	$N(=10)/N_\ell$	r_u**	ϵ_v, % Using Figure D-12 or D-13	ϵ_v, % Using Figure D-16	h (settlement) = ϵ_v x h (in.)
Case 1							
4-1/2	--------above water table---------					.36	.36x9x12/100=0.39 in.
14	1.35	35	0.29	0.29	0.10	.36	.36x10x12/100=.43 in.
24	1.16	20	0.50	0.44	0.15	.36	.36x9x12/100=.39 in.
34	0.76	10	>1.00	1.00	1.50	1.02	1.5x15x12/100=2.70 in.
							h = 3.9 in.
Case 2							
4-1/2	--------above water table---------					1.02	1.02x9x12/100=1.10
14	0.71	10	>1.00	1.00	1.50	1.02	1.5x10x12/100=1.8 in.
24	0.67	10	>1.00	1.00	1.50	1.02	1.5x9x12/100=1.62 in.
34	.69	10	>1.00	1.00	1.50	1.02	1.5x15x12/100=2.70 in.
							h = 7.2 in.

* From Figures D-9 and D-11 with respect to corresponding γ_d.

** From Figure D-10.



LEGEND

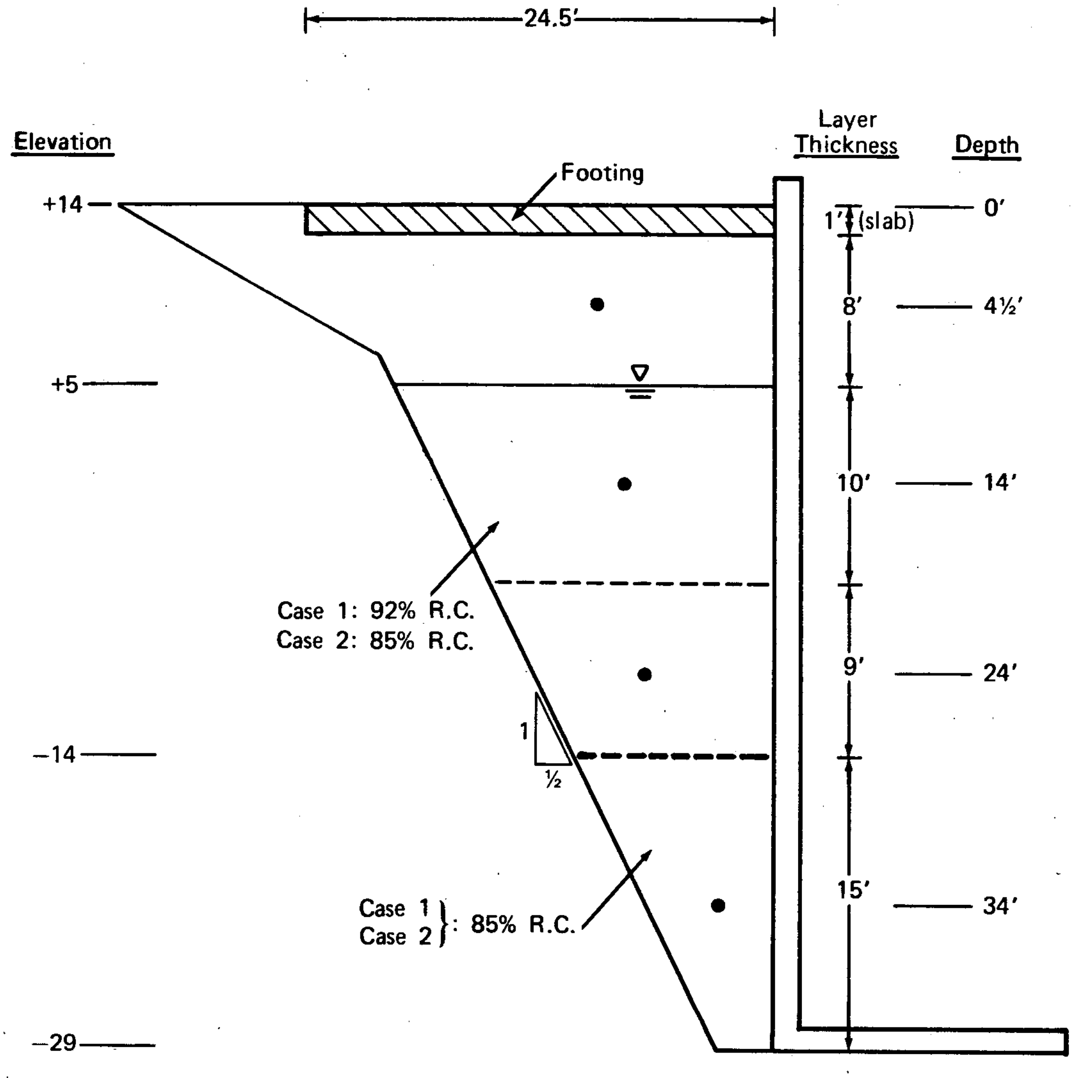
Top of Cut for Construction / Water Table	Fill above water table
Estimated Toe of Cut Slope / Edge of Structure	Fill below water table
HH——HH'	Cross section locations (Figures 2-23 to 2-26, Section 2.0).
A	Well compacted, ~95%.
B	Moderately to well compacted, ~90 to 95%. Deep narrow fills with width <6 to 10 feet, ~85%.
C	Moderately compacted, ~85 to 90%. Deep narrow fills with width <6 to 10 feet, ~85%.
D	85% relative compaction.
53——53	Identification line for backfill areas.
	Turbine plant cooler footing
(+3.5 ft.)	Elevation of native San Mateo Sand at the bottom of the foundation.
—— ——	Estimated toe of cut slope
– – – – – –	Indicates break between flatter upper slope and steeper lower slope

Woodward-Clyde Consultants

SOIL BACKFILL CHARACTERIZATION AND IDENTIFICATION

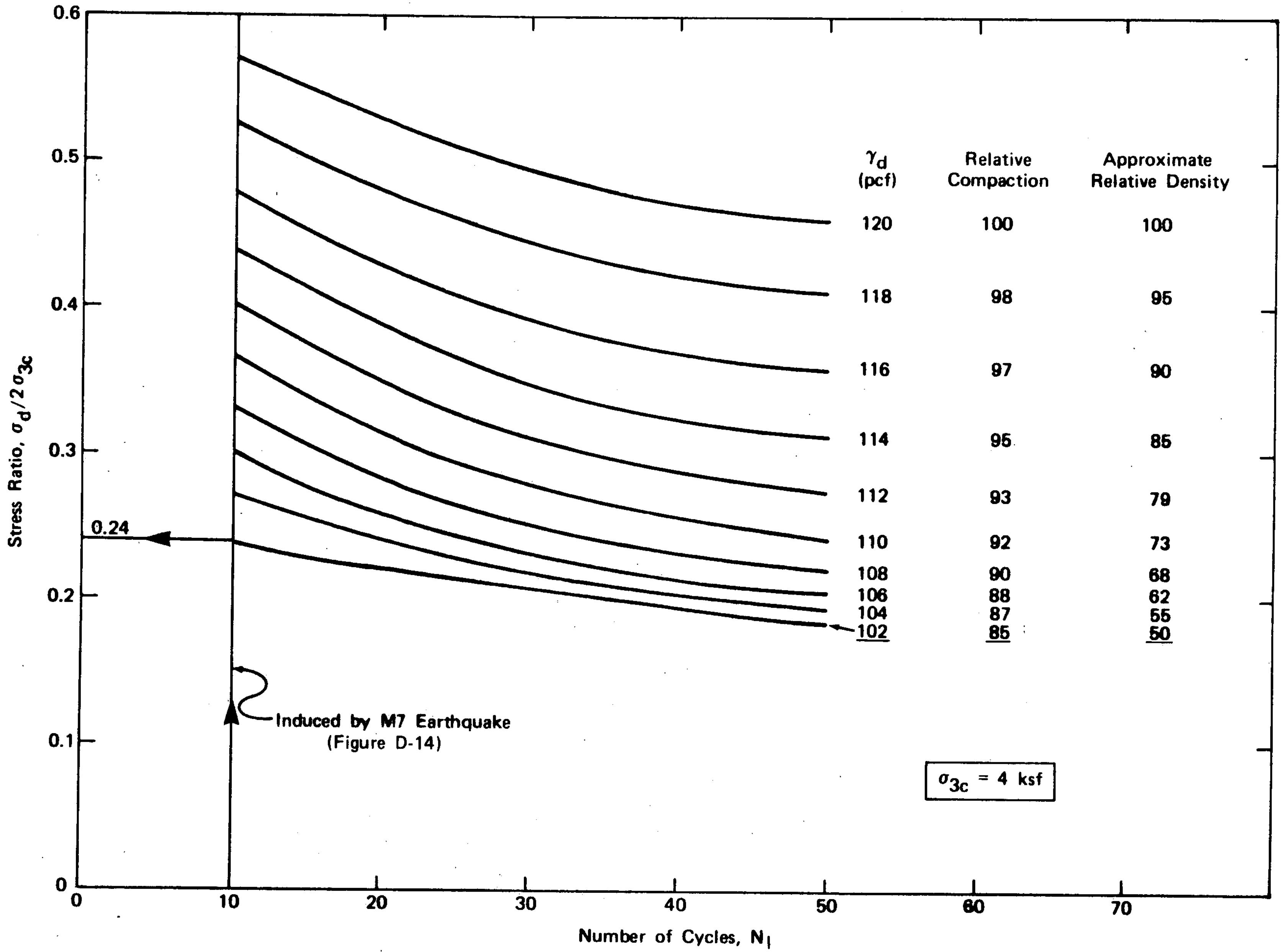
Project No. 413521
SONGS UNIT 1
SEISMIC RE-EVALUATION

Fig. D-1



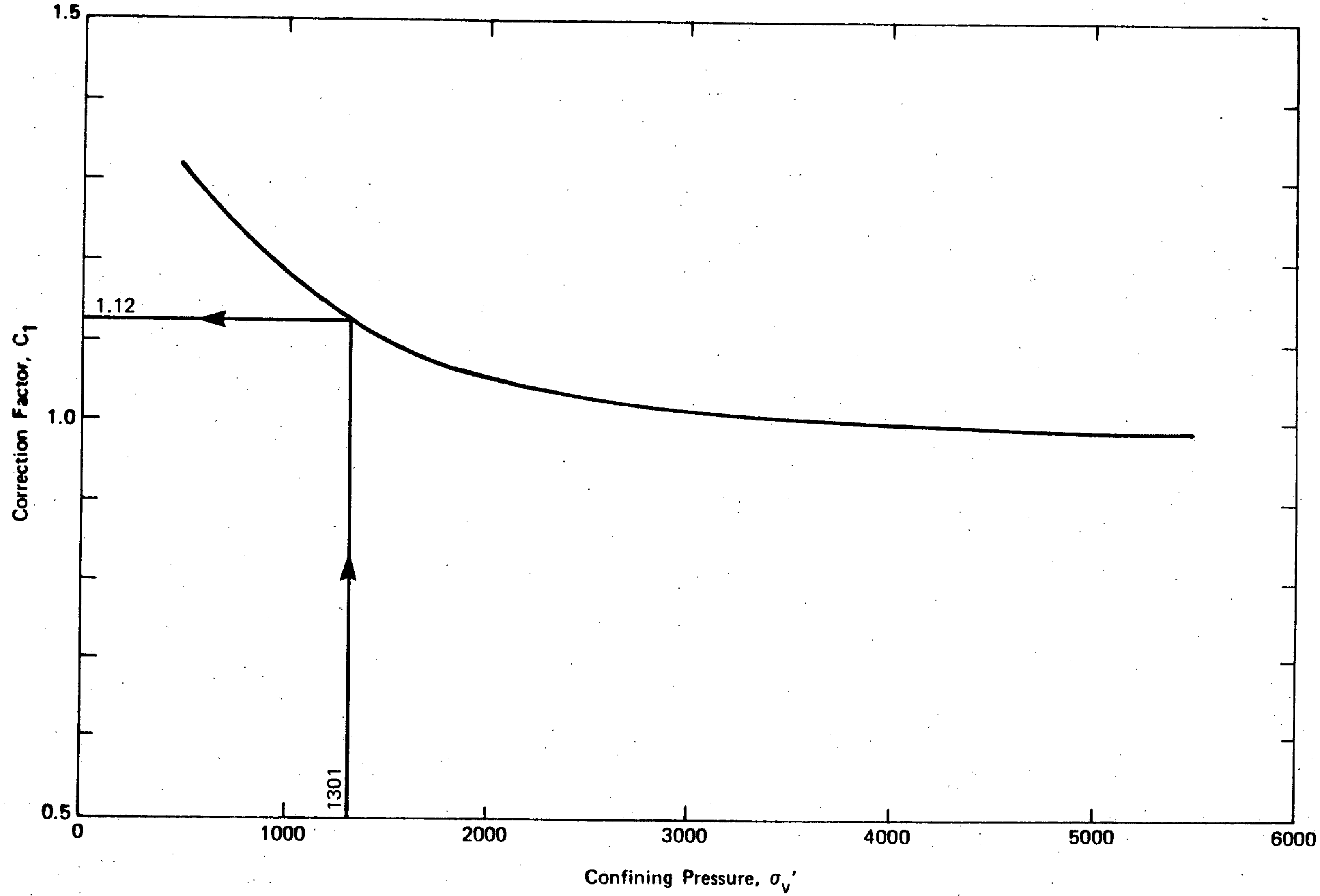
Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	TURBINE PLANT COOLER FOOTING	Fig. D-2

WOODWARD–CLYDE CONSULTANTS



Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	STRESS RATIO, $\sigma_d/2\sigma_{3c}$ VS. NUMBER OF CYCLES, N_l	Fig. D-3

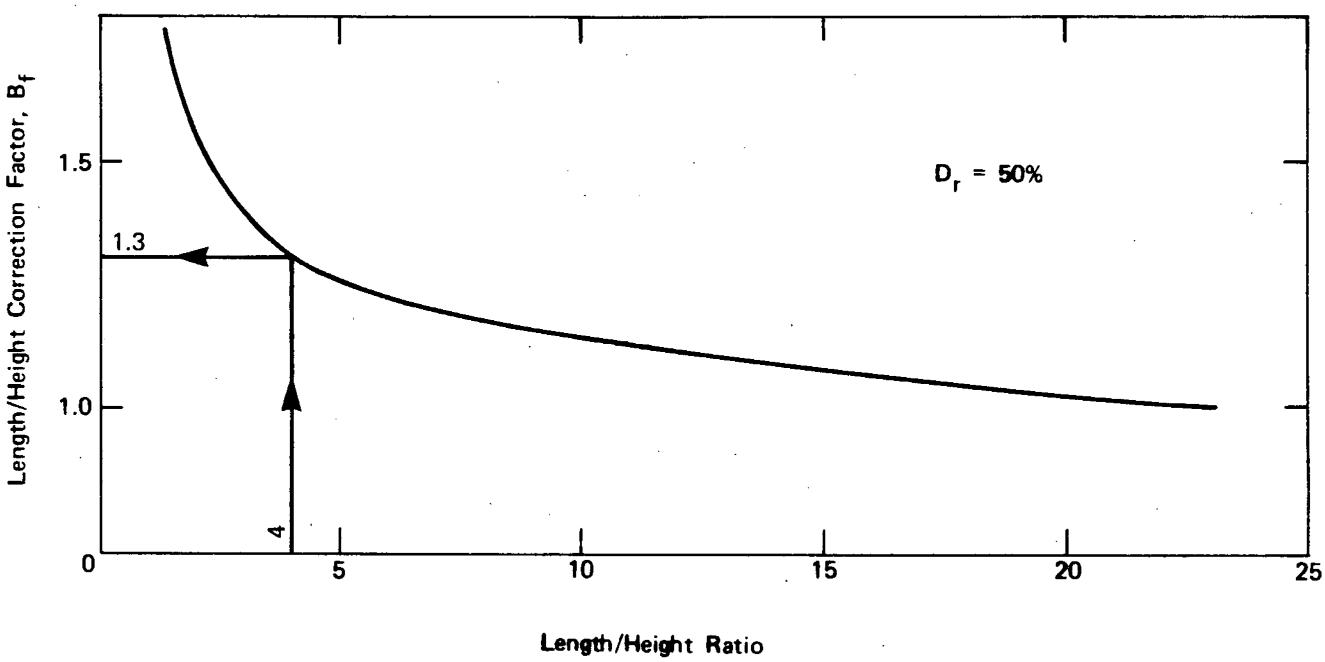
WOODWARD–CLYDE CONSULTANTS



NOTE: These data are based on test results on remolded San Mateo Sand.
See Songs 2 and 3 FSAR, Appendix 2.5A.

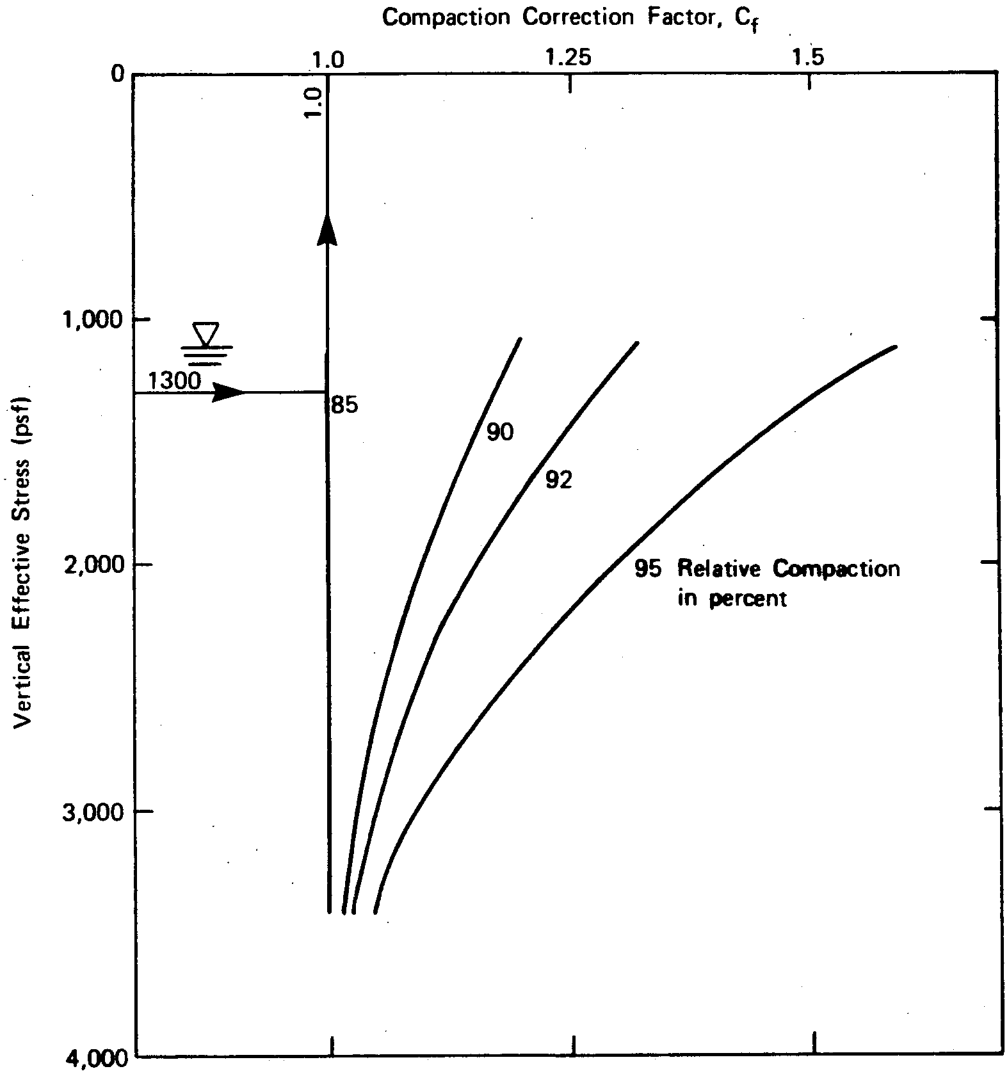
Project: SONGS UNIT 1 SEISMIC RE-EVALUATION **Project No.** 413521	EFFECT OF CONFINING PRESSURE ON LABORATORY STRENGTH	**Fig.** D-4

WOODWARD–CLYDE CONSULTANTS



Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	CORRECTION FACTOR B_f VS. LENGTH AND HEIGHT RATIO	Fig. D-5

WOODWARD–CLYDE CONSULTANTS

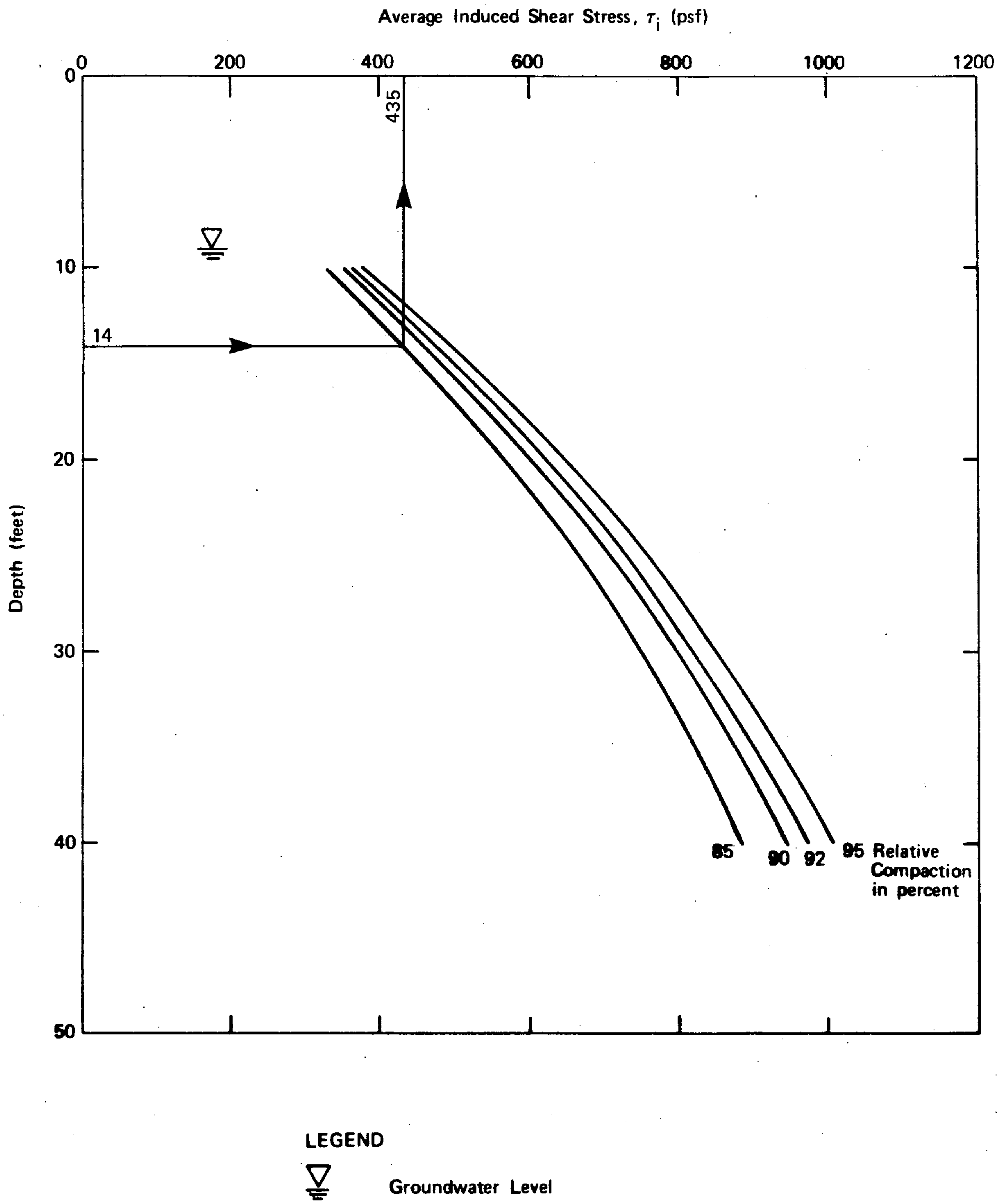


LEGEND

Groundwater Level

Project: Project No.	SONGS UNIT 1 SEISMIC RE-EVALUATION 413521	ESTIMATED VARIATION OF COMPACTION CORRECTION FACTOR C_f WITH VERTICAL EFFECTIVE STRESS AS A FUNCTION OF RELATIVE COMPACTION	Fig. D-6

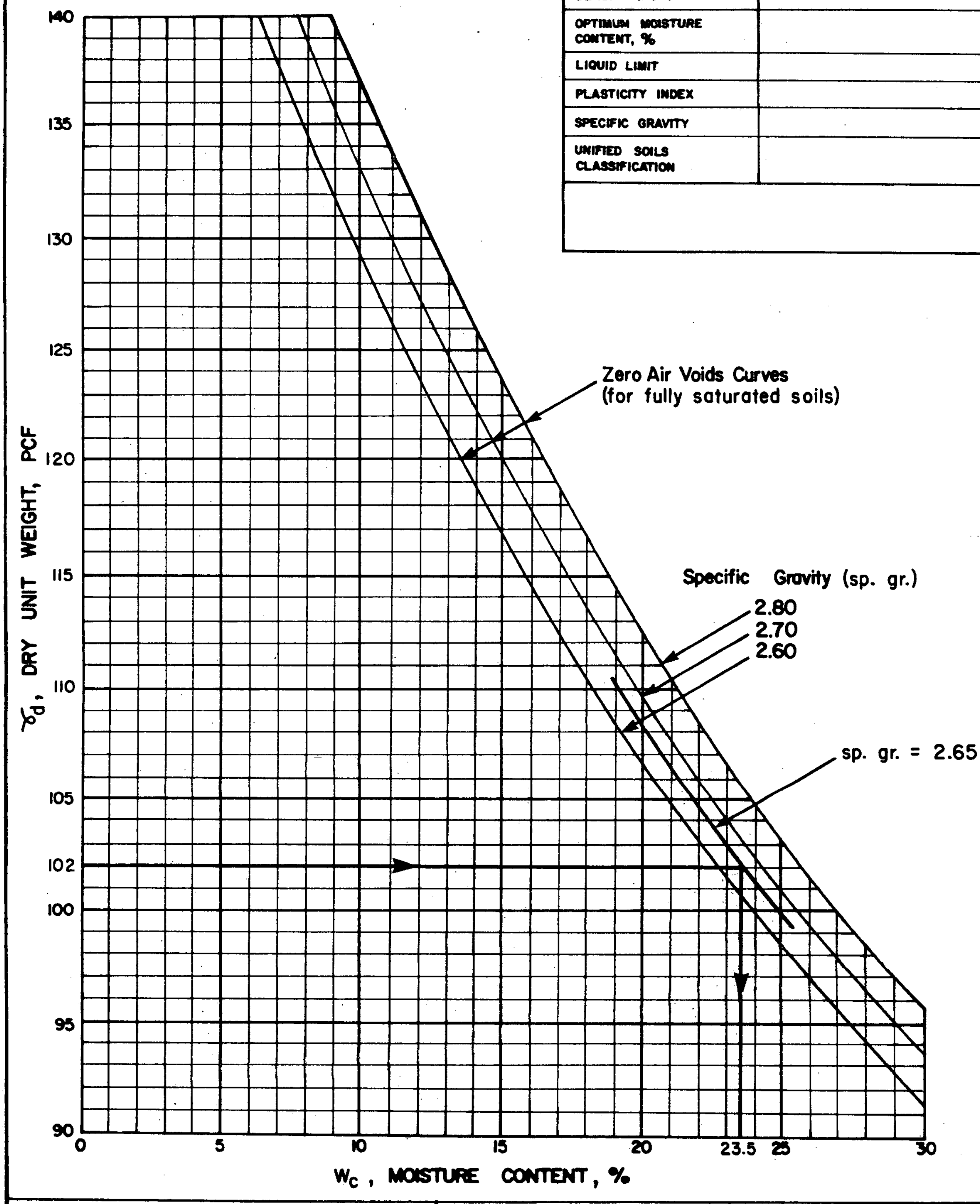
WOODWARD–CLYDE CONSULTANTS



Project: Project No.	SONGS UNIT 1 SEISMIC RE-EVALUATION 413521	INDUCED SHEAR STRESS VS. DEPTH AS A FUNCTION OF RELATIVE COMPACTION	Fig. D-7

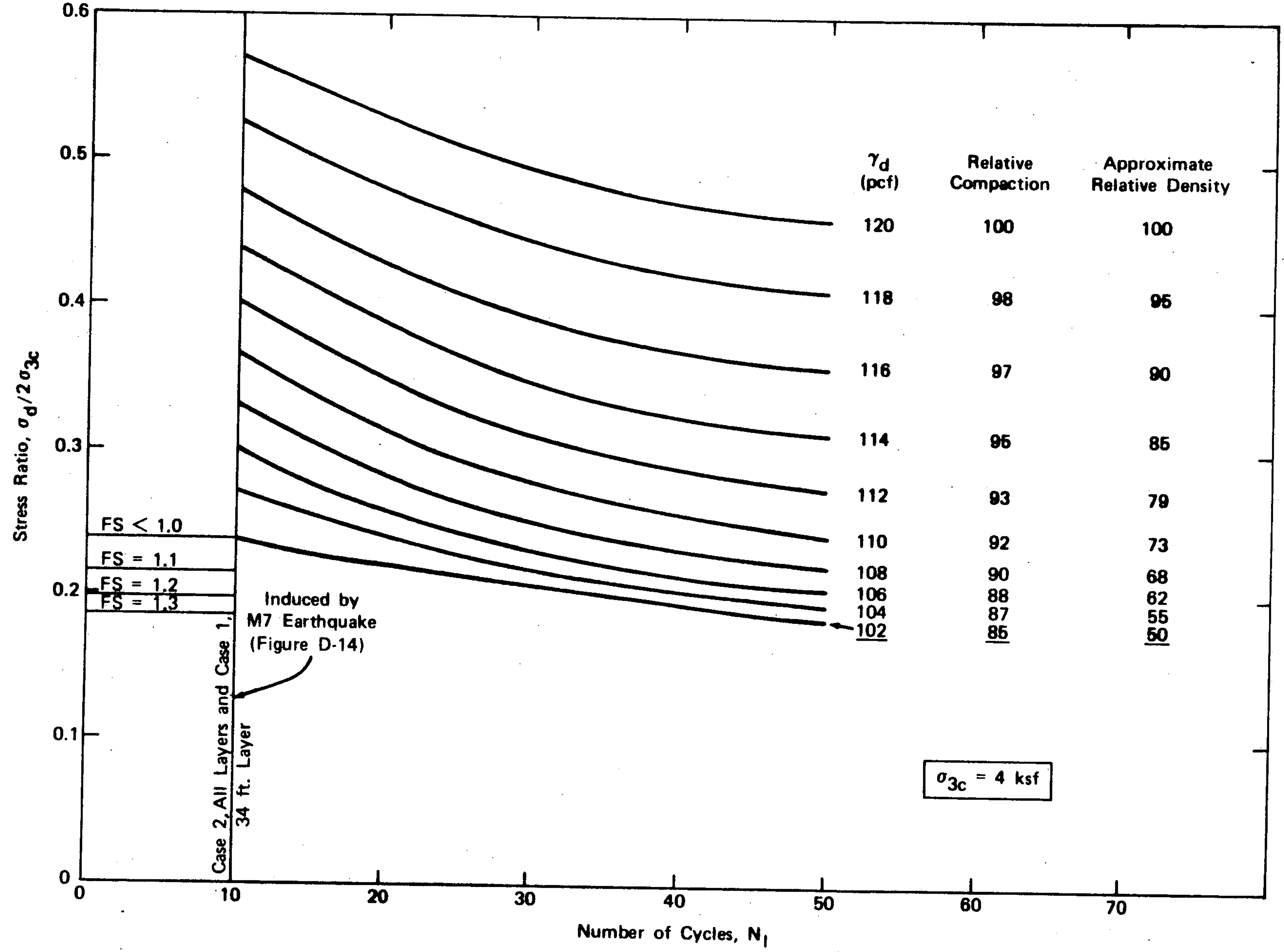
WOODWARD–CLYDE CONSULTANTS

SUMMARY OF TEST RESULTS	
SAMPLE NO.	
TEST METHOD	
MAXIMUM DRY DENSITY (PCF)	
OPTIMUM MOISTURE CONTENT, %	
LIQUID LIMIT	
PLASTICITY INDEX	
SPECIFIC GRAVITY	
UNIFIED SOILS CLASSIFICATION	



Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	COMPACTION TEST CURVE	Fig. D-8

WOODWARD-CLYDE CONSULTANTS

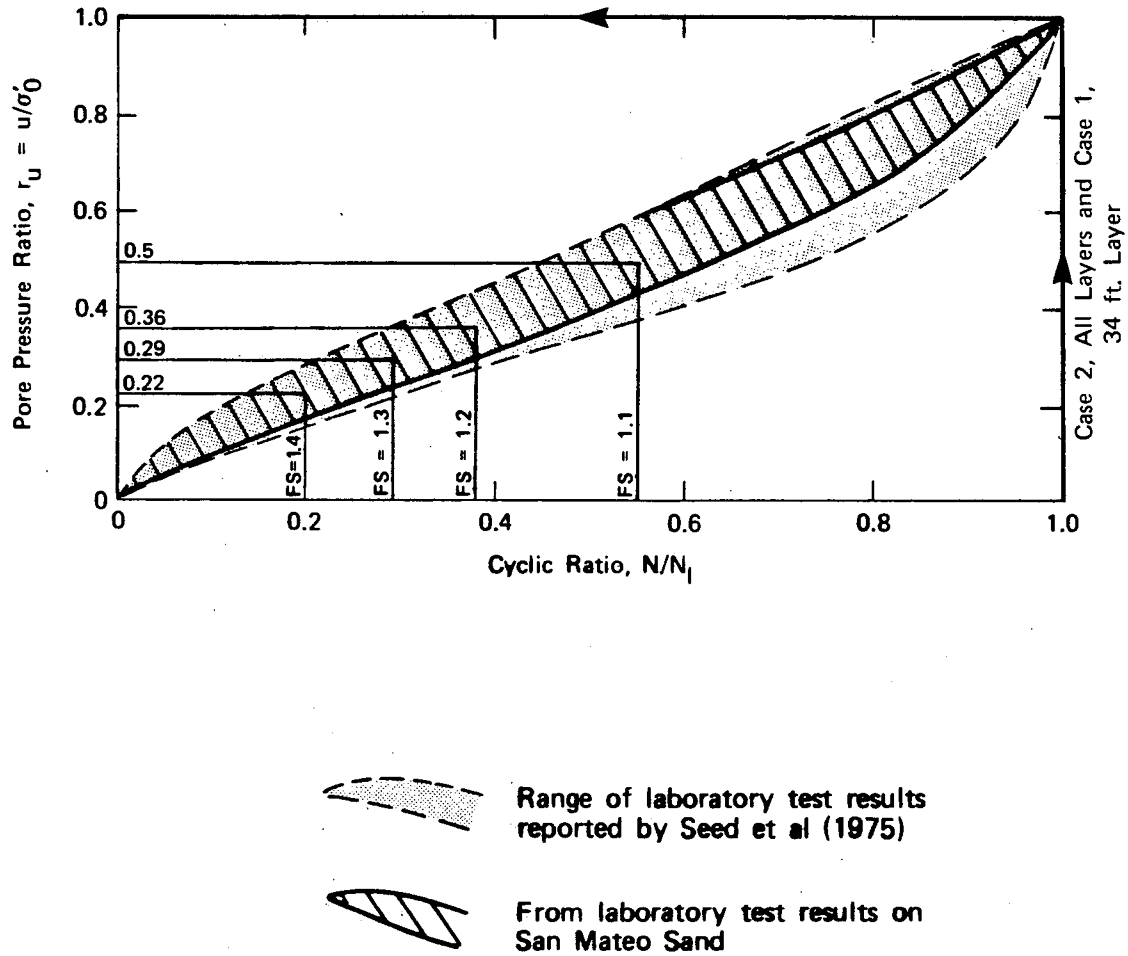


Project: SONGS UNIT 1 SEISMIC RE-EVALUATION
Project No. 413521

STRESS RATIO, $\sigma_d/2\sigma_{3c}$ VS. NUMBER OF CYCLES, N_l

Fig. D-9

WOODWARD–CLYDE CONSULTANTS

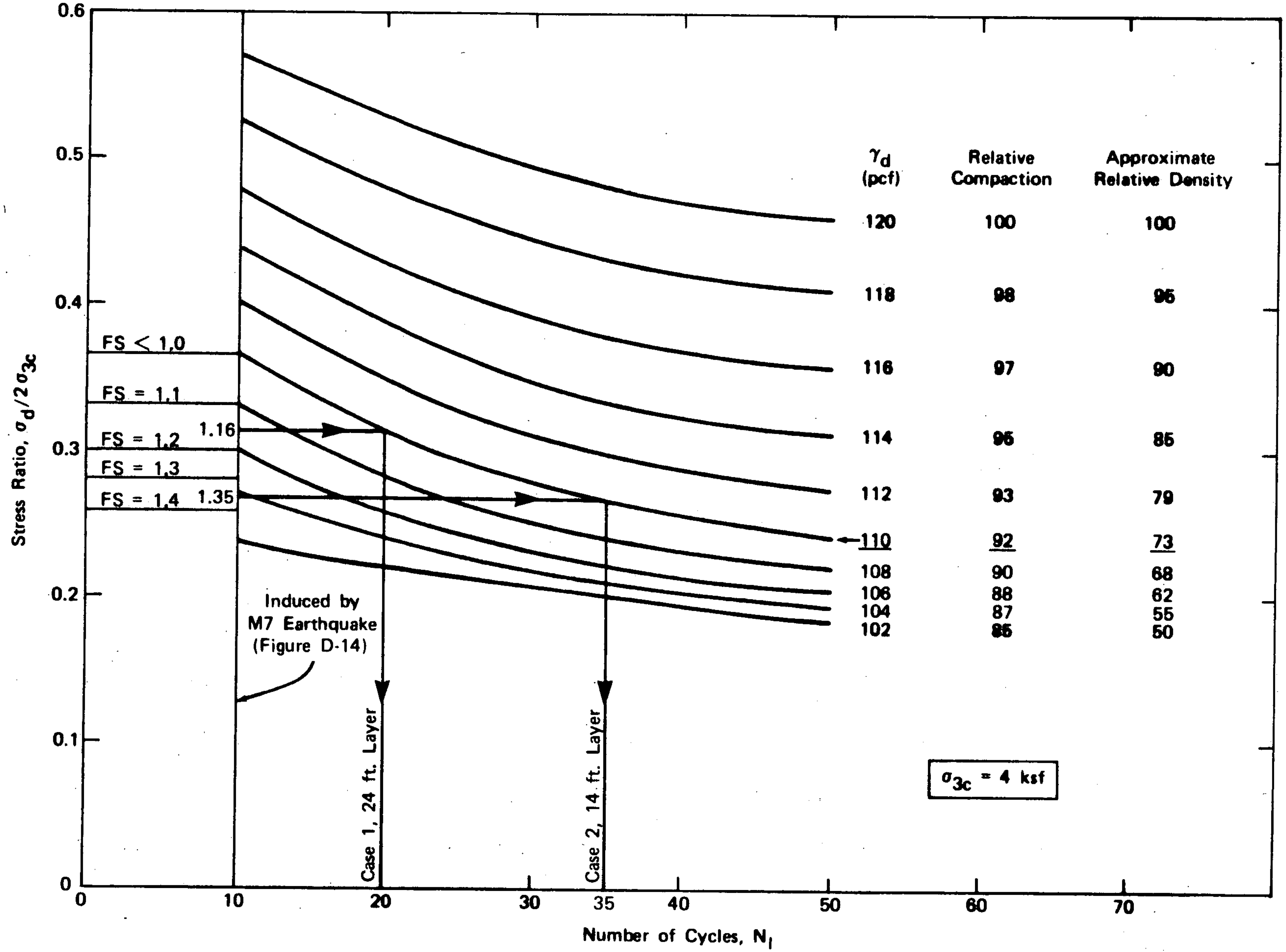


Note: **Factors of safety and corresponding N/N_l are from Figure C-21 for relative compaction of 95%.**

Ref. "The Generation and Dissipation of Pore Water Pressures During Soil Liquefaction", Report No. EERC 75-26, Earthquake Engineering Research Center, University of California; Berkeley, California; August 1975. (Ref. D-2)

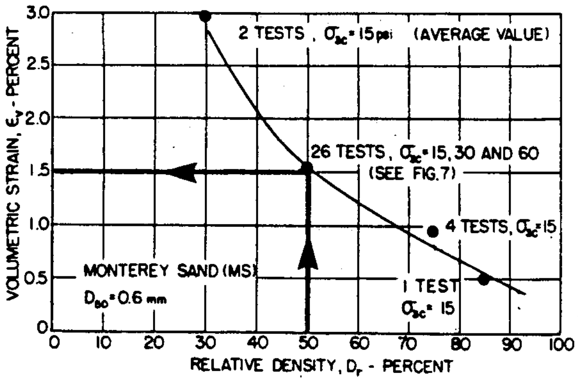
Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	PORE PRESSURE RATIO, $r_u = u/\sigma'_o$ VS. CYCLIC RATIO, $N/N_{liq.}$	Fig. D-10

WOODWARD—CLYDE CONSULTANTS



Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 41352I	STRESS RATIO, $\sigma_d/2\sigma_{3c}$ VS. NUMBER OF CYCLES, N_l	Fig. D-11

WOODWARD–CLYDE CONSULTANTS



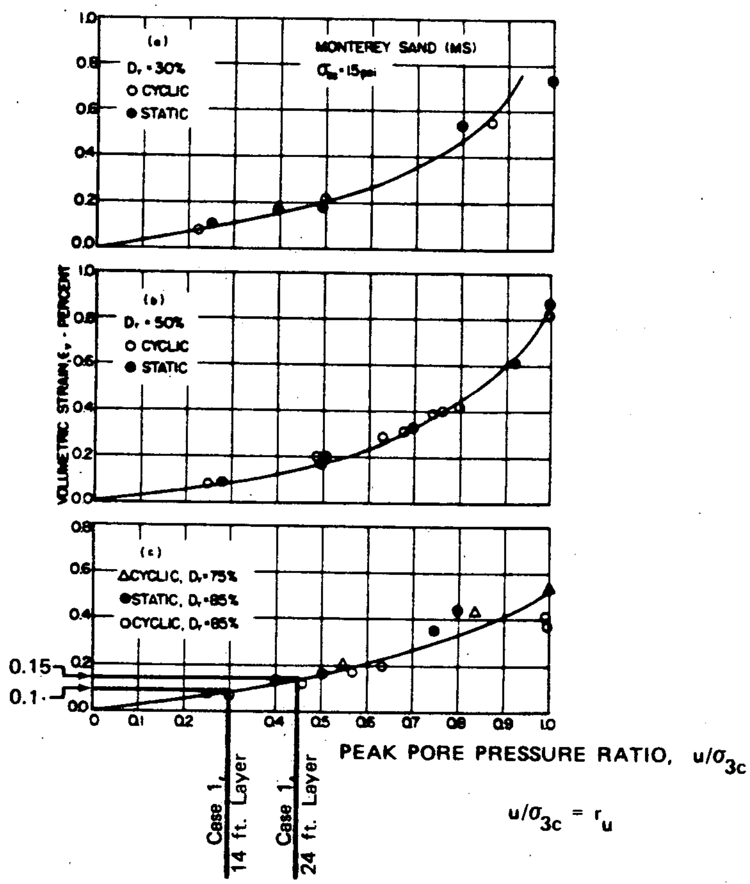
Note $D_r = 50 \approx$ R.C. = 85%

$r_u = 1.0$

From Lee, K.L. and Albaisa, A.; "Earthquake Induced Settlements in Saturated Sands", p. 395, Journal of Geotechnical Engineering, American Society of Civil Engineers, April 1975. (Ref. D-1)

Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	EFFECT OF RELATIVE DENSITY ON VOLUMETRIC STRAIN FOLLOWING COMPLETE LIQUEFACTION, i.e. $r_u = 1$	Fig. D-12

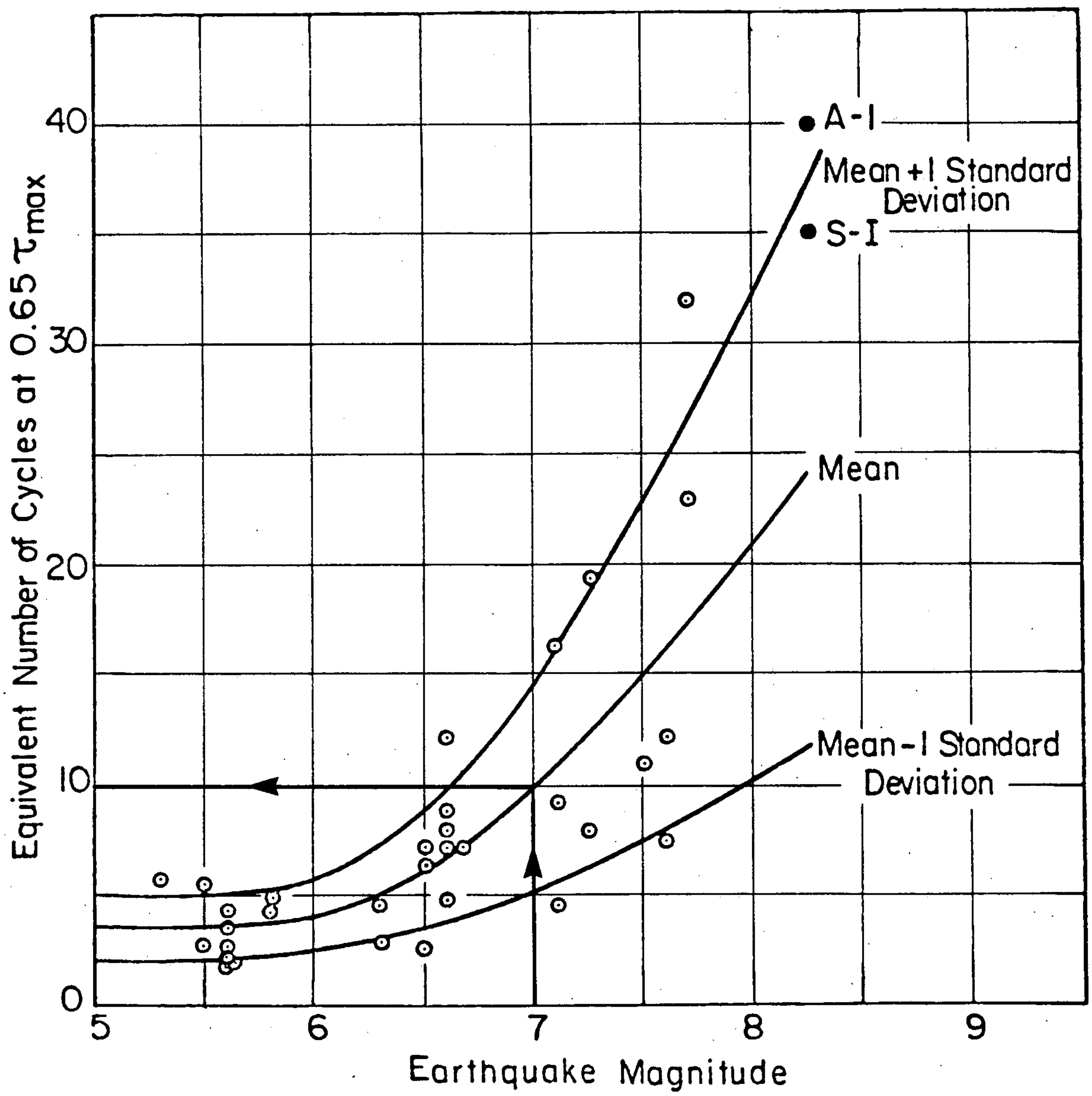
WOODWARD–CLYDE CONSULTANTS



From Lee, K.L. and Albaisa, A.; "Earthquake Induced Settlements in Saturated Sands", p. 397, Journal of Geotechnical Engineering, American Society of Civil Engineers, April 1975. (Ref. D-1)

Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	RECONSOLIDATION VOLUMETRIC STRAIN FOLLOWING STATIC AND CYCLIC PORE PRESSURE INCREASES	Fig. D-13

WOODWARD–CLYDE CONSULTANTS

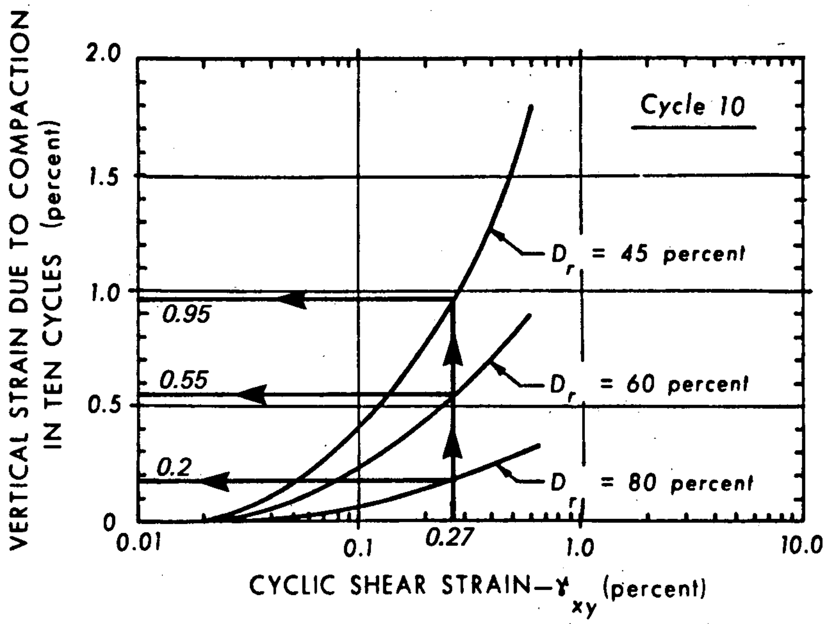


After Seed and others, 1975

Ref. Seed, H.B., Idriss, I.M., Makdisi, F., and Banerjee, N.; "Representation of Irregular Stress Time Histories by Equivalent Uniform Stress Series in Liquefaction Analysis", Report No. EERC 75-29, Earthquake Engineering Research Center, University of California; Berkeley, California; 1975. (Ref. D-4)

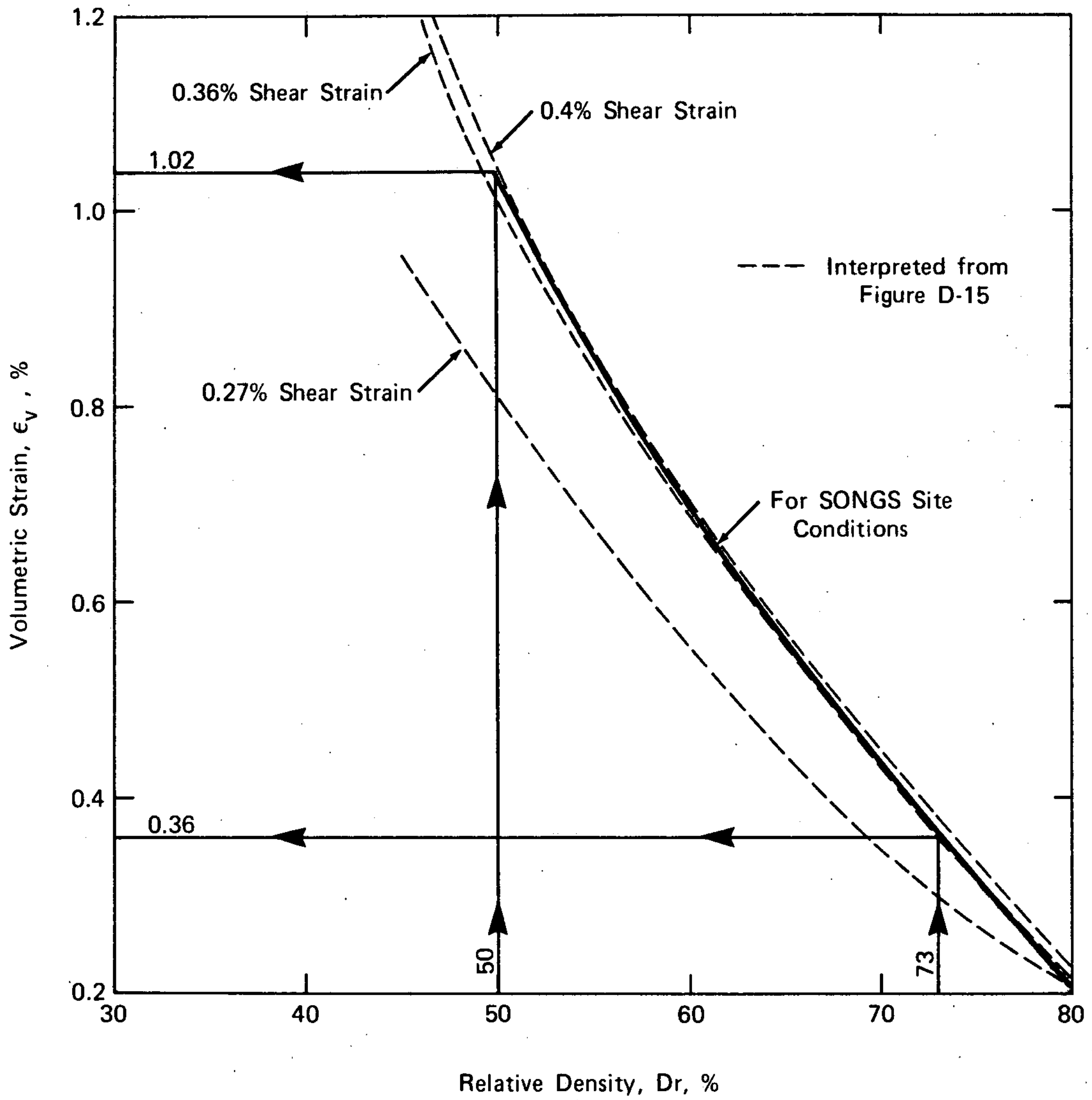
Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	EQUIVALENT NUMBERS OF UNIFORM STRESS CYCLES BASED ON STRONGEST COMPONENTS OF GROUND MOTION	Fig. D-14

WOODWARD–CLYDE CONSULTANTS



Ref. Silver, M.L. and Seed, H.B.; "Settlement of Dry Sands During Earthquakes", p. 384, Journal of the Soil Mechanics and Foundation Division, American Society of Civil Engineers, April 1972. (Ref. D-3)

Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	EFFECT OF RELATIVE DENSITY ON SETTLEMENT IN 10 CYCLES	Fig. D-15



Note Dr = 50 ≈ R.C. = 85%

Dr = 73 ≈ R.C. = 92%

Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	VOLUMETRIC STRAIN VS. RELATIVE DENSITY	Fig. D-16

WOODWARD—CLYDE CONSULTANTS

APPENDIX E
PERMEABILITY AND CLOGGING POTENTIAL OF SONGS UNIT 1 GRAVEL DRAINS

E-1 INTRODUCTION

On the sea side of the SONGS 1 seawall 29 vertical gravel drains, 24 inches in diameter and at 10 ft center to center spacing were constructed to mitigate the effects of liquefaction. Plan and cross-sections showing the distribution and depths of the gravel drains and site soils, are shown in Figure E-1, and a summary of construction notes is included as Table E-1.

For these gravel drains to be effective in dissipating excess pore water pressures, the gravel used in the vertical drains must satisfy two requirements:

1) The permeability must be great enough to allow the dissipation of pore-water pressure due to seismic loading to reduce the liquefaction potential of the sand deposit.

2) The gradation of the gravel must be such that sand grains from the surrounding soils are prevented from entering the filter and clogging it.

The extent to which these criteria are met by the gravel used in the drains adjacent to the seawall is discussed in the sections that follow. Specifically, Section E-2 discusses the permeability requirement, and Section E-3 discusses the gradation requirement.

E-2 PERMEABILITY REQUIREMENT

In a 1976 paper, Seed, et al. (Reference E-1), suggested that the permeability of the filter material in the vertical drains should be at least 50 to 200 times greater than that of the surrounding soil to ensure proper dissipation of excess pore water pressure generated in the cyclic loading condition.

The field measuremements data from laboratory and field permeability tests for the SONGS Units 2 and 3 project indicated that the average horizontal permeability for the native San Mateo sand is about 1.5×10^{-2} cm/sec. Also laboratory permeability tests performed on remolded samples of San Mateo sand yielded a range of permeabilities of from 0.6 to 1.5×10^{-2} cm/sec.

As shown in Figure E-2 the gravel used in the vertical drains has a maximum size of approximately 3/4 inch. The permeability (k) of this material may be estimated from the grain size distribution based on empirical methods as follows.

a) Justin's Formula (Reference E-1):

k (cm/sec) $= 77\,(D_{20})^{2.2}$ where D_{20} is about 0.55 cm from Figure E-2.

$= 77 \times (0.55)^{2.2}$

$= 20$ cm/sec

b) Hazen's Formula (Reference E-1):

k (cm/sec) $= 100$ to $150\,(D_{10})^2$

$D_{10} = 0.45$ cm from Figure E-2

$= 100$ to $150\,(0.45)^2$

$= 20$ to 30 cm/sec

Based on these estimates, the permeability of the gravel should be greater than 1000 times that of the in-situ soils, easily meeting the factor of 50 to 200 suggested by Seed et al. (Reference E-2).

E-3 GRADATION REQUIREMENT

The gravel material must conform to filter criteria so that the finer surrounding material is not carried into the voids of the gravel to clog the drain. Procedures to design filters, as recommended by the Bureau of Reclamation (Reference E-3) based on considerable experience, are listed as follows:

1) $$\frac{D_{15} \text{ of filter material}}{D_{15} \text{ of base material}} = 5 \text{ to } 40$$

   provided that the filter does not contain more than 5 percent of material finer than 0.074 mm (No. 200 sieve).

2) $$\frac{D_{15} \text{ of filter material}}{D_{85} \text{ of base material}} = 5 \text{ or less}$$

3) The grain-size curve of the filter should be roughly parallel to that of the base material.

   where:

   - the D_{15} is the size at which 15 percent of the total soil particles by weight are smaller,

   - the D_{85} is the size at which 85 percent of the total soil particle by weight are smaller, and

- the base material as used here refers to the in-situ soil.

The range of gradation characteristics of the in-situ soils in this vicinity of the seawall and that of the gravel used in the drains is shown in Figure E-2. Based on this range and the filter criteria, the range of grain size distributions acceptable for an effective filter material was also plotted in Figure E-2. Based on Figure E-2, the grain size distribution curve of the gravel used in the drains falls within the overall bounds of acceptable filter materials corresponding to an average gradation of in-situ soil. As can be noted however, the range of extreme coarse-grained bounds of the D_{15} of acceptable filter material corresponding to the fine-grained bound of the in-situ soil does not envelope the gravel used in the drains. Therefore the results of soil intrusion tests performed on similar gravel samples using San Mateo sand as the base material were reviewed as described below.

To evaluate the effectiveness of the gravel pack against soil intrusion for the SONGS Units 2 and 3 dewatering wells, laboratory soil intrusion tests were performed. In these tests, San Mateo sand (loose, compacted, and intact-carved) was placed above typical filter material and water was pumped through the sand at gradients varying from 5 to 500. The first group of tests was run in a specially fabricated cylindrical apparatus (3.25-in. diameter and 6-in. long) in which gravel material was overlain by loose San Mateo sand. The base of the cylinder consisted of a No. 20 sieve over a plastic slab drilled with 1/4-inch holes. Water was introduced into the apparatus from the top through a 1/4-inch opening and collected at the base after filtering through a No. 30 sieve first and then a No. 200 sieve. The material from which the filter was graded was a gravel meeting the gradation range of Class 2 permeable material, Caltrans Standard Specifications, Section 68-1.025. This material was selectively graded for use in the tests as follows:

Test 1: Fines passing the No. 200 sieve were removed from the filter gravel.

Test 2: Materials retained on the 3/8-inch sieve and passing the No. 10 sieve were removed from the filter gravel.

Test 3: Filter gravel consisted of material retained on the No. 4 sieve.

The gradation curves for each test are shown in Figure E-3 together with that of the gravel used in the drains for the Unit 1 seawall.

The test procedure was to increase the water pressure from 5 to 50 psi to complete the test in 10 minutes. In Test 3, the pressure was increased to 90 psi and left overnight (16 hours).

The observations from these tests conducted at high gradients (50 psi is a gradient of about 500) indicate essentially no loss of filter material and San Mateo sand. Even though the filter and the sand were placed loosely, no collapse of the sand or soil transport through the filter was visible.

The second group of tests was run in a triaxial cell on specimens of San Mateo sand (4-inch diameter and 5-1/4 to 5-1/2 inch long) carved from intact block samples. The general procedure of these tests was to first saturate the sand with slow water penetration from bottom to top of the specimen. Then the specimen was subjected to water flow from top to bottom in gradually increasing gradients up to gradients of 25 to 120. The filter at the base of the specimen was varied as follows:

- A No. 20 Sieve
- Test 1 gravel material in Figure E-3
- Gravel material with about the same gradation as the gravel used in the drains.

In each case, no significant soil transport through the filter was noted. Some colloidal material came out at first but the flow cleared as the test progressed. The rate of flow did not change appreciably.

As indicated in Figure E-3, the gravel materials used in the various tests are similar to those used in the drains; and in one case, the Test 3 sample was more coarse-grained. Based on the results of these tests together with the fact that the gradation of the gravel used in the drains falls within the range of gradations acceptable for a filter material, it is concluded that the gravel drains will not experience clogging due to intrusion of the in-situ native or fill soil.

APPENDIX E
REFERENCES

E-1 Seed, H. B., Martin, P. P., and Lysmer, J., "The Generation and Dissipation of Pore Water Pressure During Soil Liquefaction", EERC 75-76 Earthquake Engineering Research Center, University of California, Berkeley, California, 1975.

E-2 Seed, H. B., and Booker, J. R., "Stabilization of Potentially Liquefiable Sand Deposits using Gravel Drain Systems," EERC 76-10 Earthquake Engineering Research Center, University of California, Berkeley, California, 1976.

E-3 Bureau of Reclamation, "Design of Small Dams," Water Resources Technical Publication, 1977.

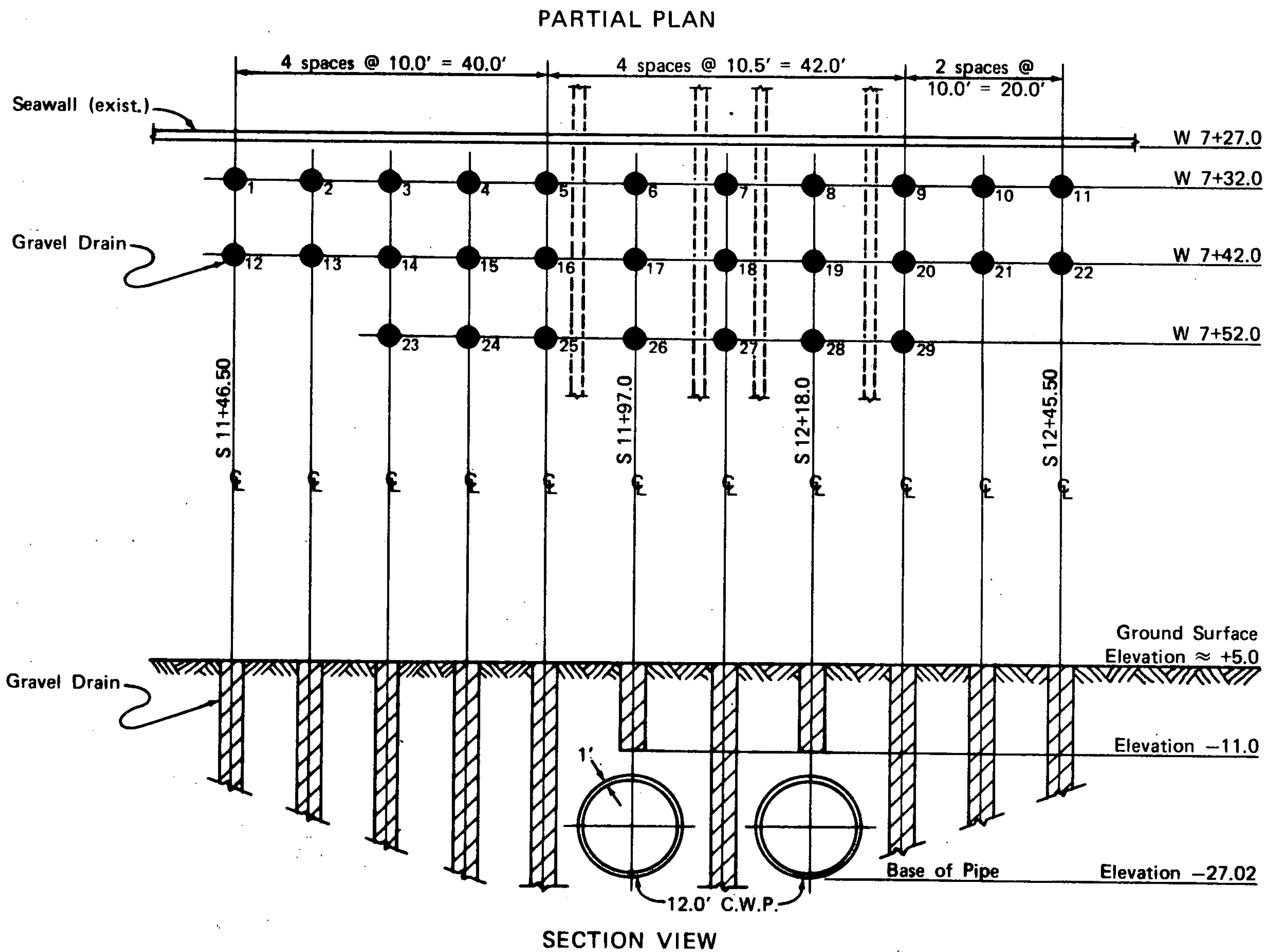
TABLE E-1

Summary of Construction Notes

Vertical gravel drains, west side of seawall in area of beach walkway - subgrade elevation approximately +5 feet

Date	Drain No.	Depth Drilled (ft)	Diameter (inches)	Depth to Undisturbed San Mateo* (ft)	Drilling Mud Used	3/8" Gravel Placed	Notes
20 Aug 81	1	10	24	3	No	Yes	
20,21 Aug 81	2	5	24	3	No	Yes	Redrilled additional 1 ft
20,21 Aug 81	3	5	24	3	No	Yes	Redrilled additional 1 ft
20,22 Aug 81	4	22	24	17	Yes	Yes	Caved to 7 ft depth redrilled with mud to 22 ft
22 Aug 81	5	30	30	25	Yes	Yes	
22 Aug 81	6	16	30	No contact	Yes	Yes	Drain over conc. pipe
21 Aug 81	7	35	30	33	Yes	Yes	Construction debris 31' to 33'
21 Aug 81	8	15.5	24	No contact	Yes	Yes	Drain over conc. pipe
21 Aug 81	9	33	30	29	Yes	Yes	Siltstone 29' to 33'
2[illegible]2 Aug 81	10	22	24	17	Yes	Yes	Caved to 8 ft, redrilled with mud to 22 ft
21 Aug 81	11	4	24	2	No	Yes	
	12						Not drilled
	13						Not drilled
21 Aug 81	14	6	24	3	No	Yes	
24 Aug 81	15	20	30	17	Yes	Yes	
24 Aug 81	16	32	30	29	Yes	Yes	
24 Aug 81	17	15.5	30	No contact	Yes	Yes	Drain over conc. pipe
24 Aug 81	18	34	30	32	Yes	Yes	
24 Aug 81	19	16	30	No contact	Yes	Yes	Drain over conc. pipe
24 Aug 81	20	33	30	30	Yes	Yes	
24 Aug 81	21	5	30	3	No	Yes	
24 Aug 81	22	5	30	3	No	Yes	
21 Aug 81	23	7	24	2	No	Yes	
26 Aug 81	24	22	30	17	Yes	Yes	
25 Aug 81	25	32	30	30	Yes	Yes	
25,25 Aug 81	26	14	30	No contact	Yes	Yes	Caved, redrilled to 14 ft
25 Aug 81	27	35	30	33	Yes	Yes	
25 Aug 81	28	15.5	30	No contact	Yes	Yes	Drain over conc. pipe
25 Aug 81	29	33	30	30	Yes	Yes	

* or siltstone

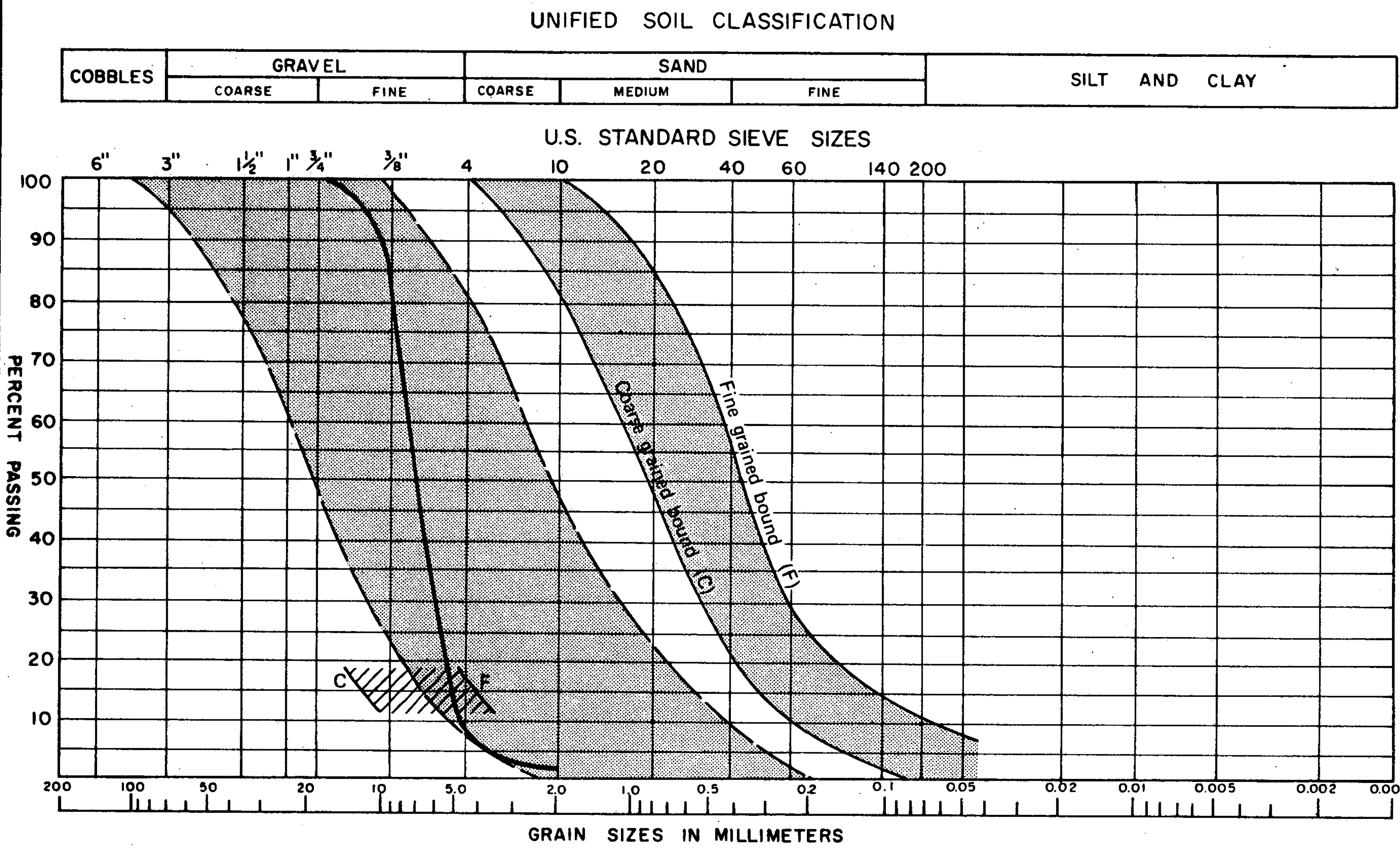


Project: SONGS UNIT 1 SEISMIC RE-EVALUATION
Project No. 413521

PLAN AND SECTION VIEWS OF GRAVEL DRAINS

Fig. E-1

WOODWARD–CLYDE CONSULTANTS



LEGEND

Range for in-situ soil

Range of acceptable filters for the average gradation of in-situ soil

Gravel used in drain

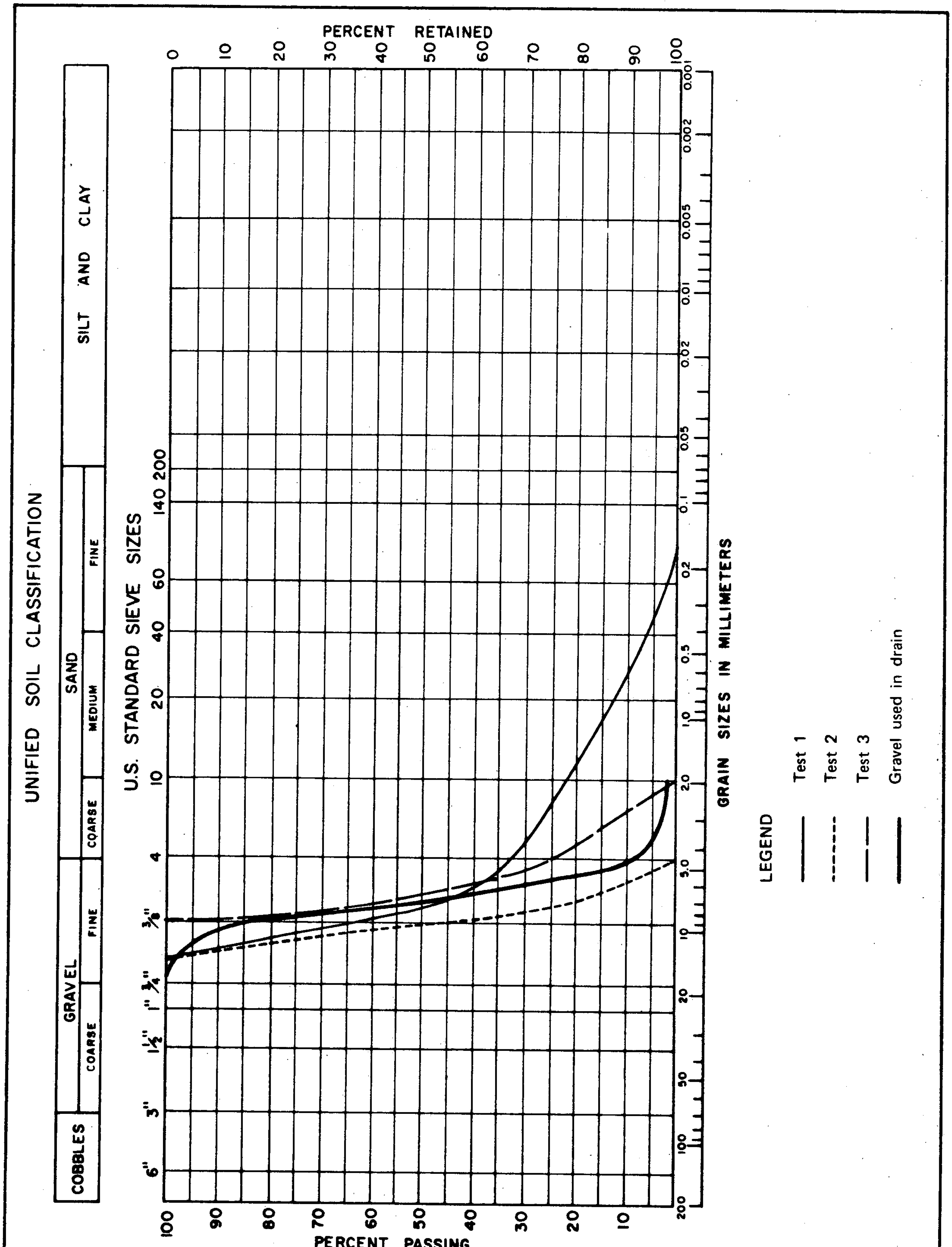
F
C
Range of the extreme coarse grained bounds for the D_{15} with "C" corresponding to the coarse grained bound of the in-situ soil and "F" corresponding to the fine grained bound of the in-situ soil

Project: SONGS UNIT 1 SEISMIC RE-EVALUATION
Project No. 413521

GRAIN SIZE DISTRIBUTION CURVES

Fig. E-2

WOODWARD-CLYDE CONSULTANTS



Project: SONGS UNIT 1
SEISMIC RE-EVALUATION
Project No. 413521

GRAIN SIZE DISTRIBUTION CURVES

Fig. E-3

WOODWARD-CLYDE CONSULTANTS

APPENDIX F

VARIATION OF MODULUS AND DAMPING WITH STRAIN, DENSITY, AND PEAK GROUND ACCELERATION

F-1 INTRODUCTION

This appendix describes the development of shear modulus and damping relationships as a function of the induced strain and the density of the soil deposit, and provides an indication of how modulus is affected by peak ground acceleration. Specifically, Section F-2 describes the development of the variation of the shear modulus with density and strain and data variations. The affect of peak ground acceleration on the shear modulus is discussed in Section F-3. Section F-4 discusses hysteretic damping as a function of these parameters.

F-2 DEVELOPMENT OF MODULUS RELATIONSHIPS AND DATA VARIATION

The shear modulus-strain relationship for the San Mateo sand at the SONGS site was developed based upon carefully performed laboratory cyclic triaxial tests, and upon cross-hole and downhole geophysical measurements as reported in the SONGS 2 and 3 PSAR. These results were later verified based on foundation response tests from which excellent correlation was found between the actual and theoretical transient response of large foundations constructed at the SONGS site as reported in the SONGS 2 and 3 FSAR, Appendix 3.7C. The shear modulus-strain relationship developed for use from these studies is as shown in Figure F-1. These results indicated no differences between modulus values for native San Mateo sand and San Mateo sand compacted to 95 percent relative compaction or higher (as determined by ASTM

Test Procedure 1557) in the strain range greater than about 7 x 10^{-3} percent major principal strain (10^{-2} percent shear strain).

The shear modulus to be used in the spring constants for the response analyses of structures is therefore developed based on the relationship:

$$G \text{ (psf)} = 100\ K_m\ (\sigma_m)^{2/3}$$

where K_m is a parameter dependent on shear strain and σ_m is the mean confining pressure equal to 2/3 of the overburden pressure σ_o. A single value of K_m = 50 is used for DBE level seismic loading (i.e., 2/3 g peak ground acceleration) corresponding to an average major principal strain of 0.2 percent from Figure F-1 (shear strain of 0.27 percent). This strain was calculated based on 2 dimensional finite element response analyses completed for the Units 2 and 3 site and supplemented by parametric one dimensional wave propagation (SHAKE) response analyses in the Unit 1 analyses. The value of σ_o (from which σ_m is calculated) is based on the confining pressure due to overburden plus bearing pressure at a depth of 1/2 equivalent foundation radius below the foundation.

The soil conditions at the SONGS site are extremely uniform, resulting in very small areal and depth non-uniformities in the soil properties. The soils at the site are uniformly dense and extend to about 1,000 ft below site grade with the absence of significant layering, thereby precluding impedance mismatches. Based on over 100 field density tests at the site, the dry density of the native San Mateo sand within 70 ft of plant grade covering the SONGS Units 1, 2, and 3 areas varies up to 3 percent about a median value. Considering this variation in density and the possible variation in Poisson's ratio and shear modulus due to the small areal non-uniformities in the soil, a ± 5 percent variation in the soil properties is judged conservative. Also, a variation in calculated DBE-level seismically induced strain of ± 30 percent is considered appropriate because it accommodates most of the seismically induced strain peaks dominant in the site response as shown in Figure F-2. Specifically, the top of Figure F-2 shows the seismically induced strain-time history for the SONGS 1 DBE as calculated from one-dimensional response analysis of the site. The absolute peaks of the dominant strain pulses are plotted with time at the bottom of Figure F-2. The maximum peak strain was calculated to be 0.3 percent. An average strain of 2/3 the maximum peak value, or 0.2 percent strain, was selected for use in analysis. As shown in Figure F-2, ± 30 percent of the selected average strain accommodates all peak values between about 45 and 90 percent of the calculated maximum strain. Also 90 percent of the dominant strain peaks occurring during seismic shaking are accommodated by this range. Therefore, this range of strain dominates site response during DBE level shaking. The ± 30 percent variation in strain results in a variation no greater than ± 10 percent in the shear modulus.

The shear modulus values appropriate for other densities less than 95 percent relative compaction were developed based on the shape of the strain curves published by Seed and Idriss (1972) as shown in Figure F-3. Specifically, the ratio of the shear modulus values at 50 percent relative density (85 percent relative compaction) and 30 percent relative density (80 percent relative compaction) to 85 to 90 percent relative density (95 percent relative compaction) is 0.79 and 0.74, respectively, at high induced shear strain and 0.66 and 0.49, respectively, at low induced shear strain as identified in Figure F-3. By taking similar ratios at strains ranging between 10^{-2} and 1 percent shear strain the 85 percent relative compaction curve was developed in this strain range as indicated in Figure F-4. At shear strain lower than 10^{-2} percent, the shape of the D_r = 90 percent curve from Figure F-3 was used to extrapolate that portion of the 95 percent relative compaction curve in this strain range. The 95 percent relative compaction curve is considerably lower than the curve for the native soil as might be expected considering that strains below 10^{-2} percent would not likely affect the response of the native structure of the soil. The remainder of the 85 percent relative compaction shear modulus curve (km versus shear strain) in Figure F-4 below 10^{-2} percent shear strain was determined based on the ratio between the D_r = 50 percent to D_r = 90 percent curves in Figure F-3 and applied to the 95 percent and higher relative compaction curve in Figure F-4.

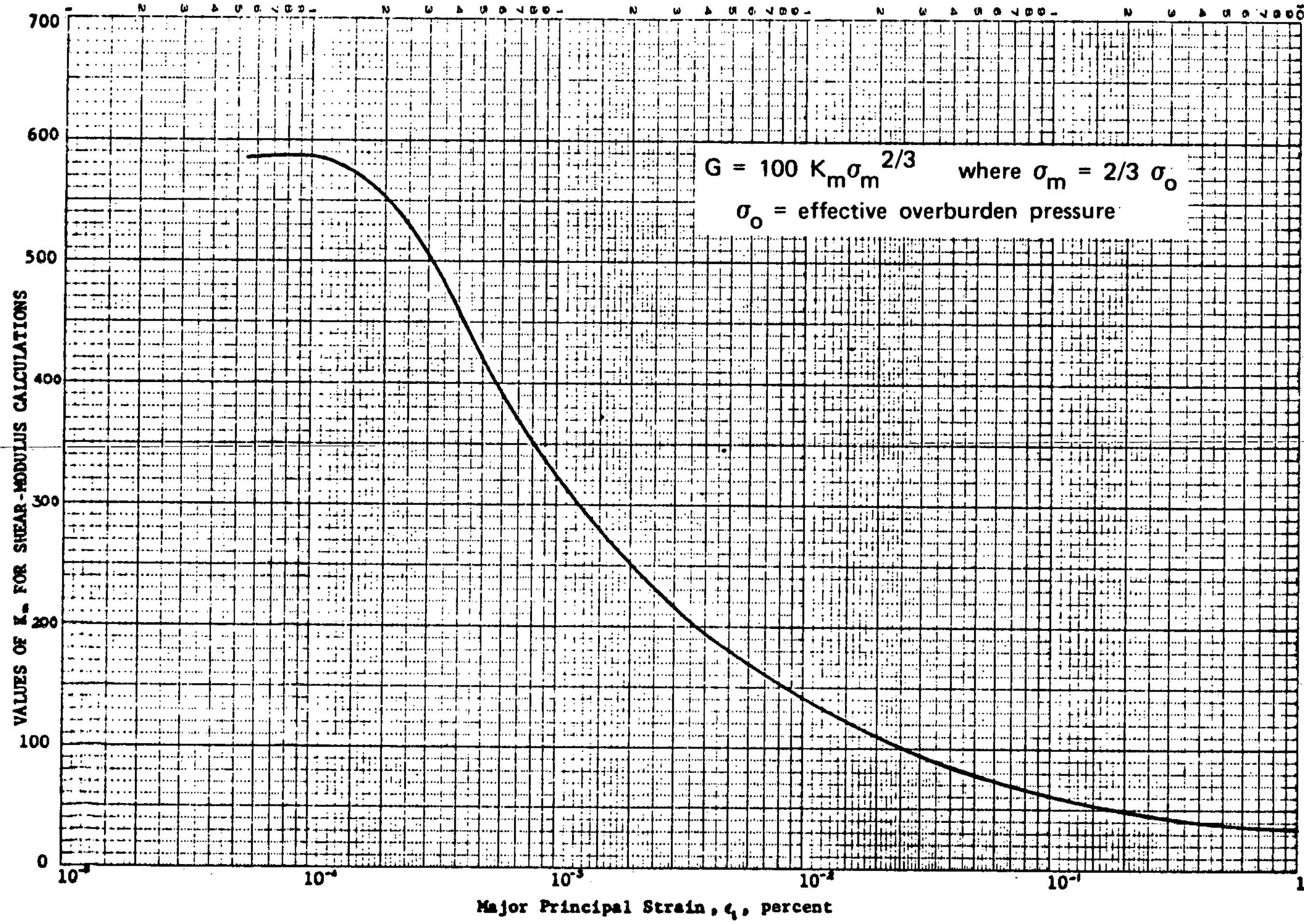
F-3 EFFECT OF PEAK GROUND ACCELERATION ON SHEAR MODULUS

A set of 24 response analyses using the program SHAKE were completed to evaluate the affect of peak ground acceleration on shear modulus. The results of these analyses are shown in Figure F-4 for earthquakes having peak ground accelerations ranging between 1/100 g to 2/3 g. The shear modulus has been plotted on this figure as a function of induced seismic strain for given values of peak ground acceleration and density condition. As can be seen in Figure F-4, the variation of shear modulus with soil density is quite small for DBE level loading, showing a 20 percent reduction between the native or 95 percent relative compaction curves and the 85 percent relative compaction. As indicated in Section 3.3 of the text of this report, the reduction in spring constant may be considerably larger considering a reduction in efficiency of embedment and effects of potential liquefaction or high pore water pressures. The corresponding reduction remains relatively small (increasing to 30 percent or less) for earthquakes extending down to 1/10 g. For smaller earthquakes however, the range between the various soil conditions is considerably larger, with the reduction between native and soil at 85 percent relative compaction increasing to 50 to 65 percent for very small earthquakes. Also, whereas there is no difference in modulus values between the native soil and soil compacted to 95 percent and greater for earthquakes with a peak ground acceleration of 1/10 g to 2/3 g, there is a difference for earthquakes with a peak ground acceleration smaller than 1/10 g.

F-4 DEVELOPMENT OF DAMPING RELATIONSHIPS

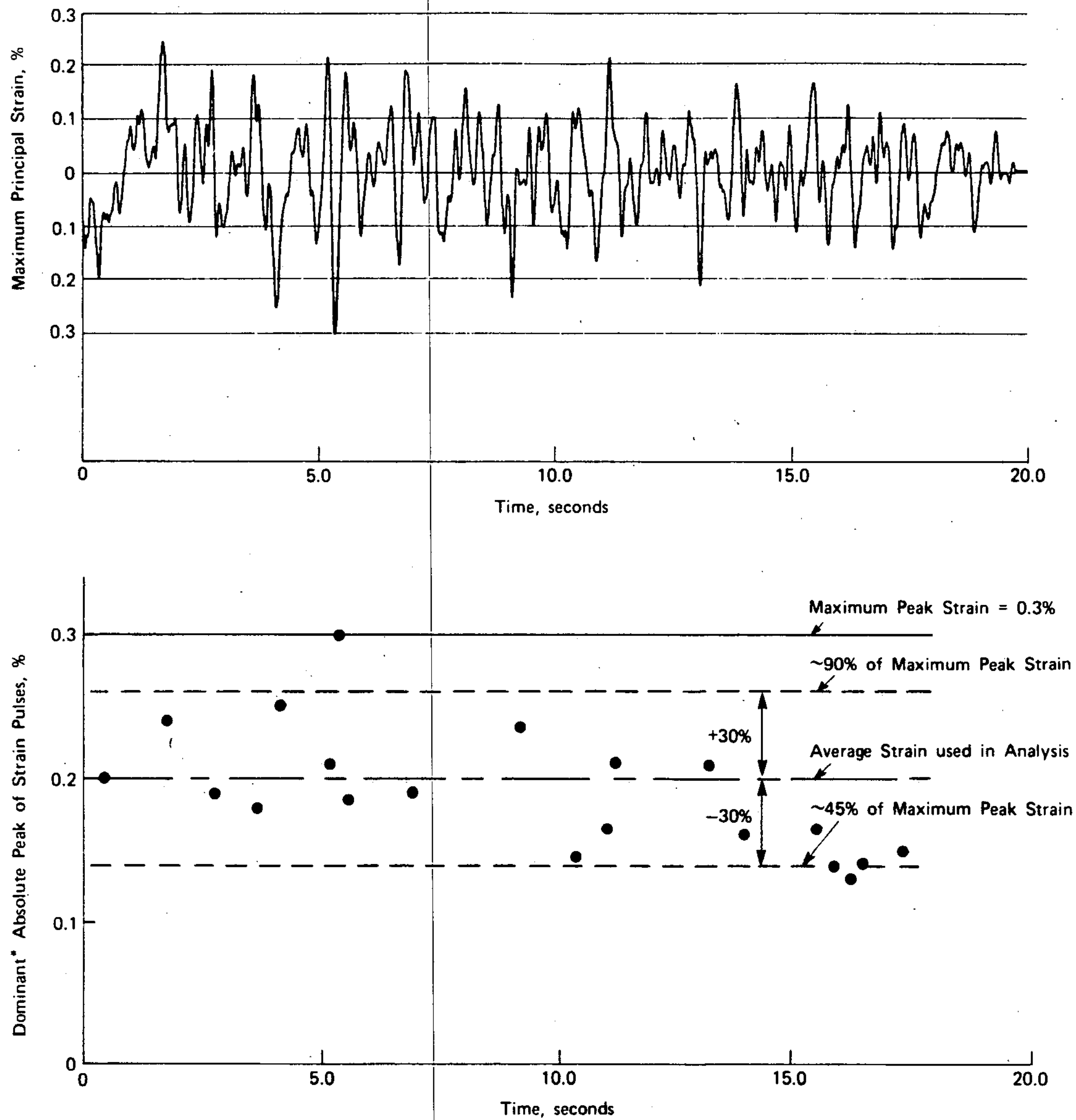
The damping-strain relationship for the San Mateo sand at the SONGS site shown in Figure F-5 was developed in parallel with the shear modulus-strain relationship discussed in Section F-2. The hysteretic damping was measured by cyclic triaxial testing in the laboratory and by field attenuation tests at the site as documented in the SONGS Units 2 and 3 PSAR and FSAR. The measurements were made on intact native samples and samples recompacted to 95 percent relative compaction and greater in the laboratory and on the exposed native San Mateo at plant grade at the site. The foundation response tests set up the procedure for combining hysteretic and geometric damping to develop conservative damping parameters for soil-structure interaction analyses as documented in Appendix 3.7C of the SONGS 2 and 3 FSAR.

To develop the damping strain relationship for the hysteretic damping component for soils compacted to densities lower than 95 percent relative compaction, the damping-strain relationship from Figure F-5 was overplotted onto the damping-strain curves and data from Seed and Idriss, 1972 shown in Figure F-6. As shown in Figure F-6, the SONGS damping curve is at the lower bound of damping for shear strains greater than about 10^{-2} percent, and covers the range of data at strains smaller than about 10^{-2} percent. This curve was used for the San Mateo sand independent of compaction density. Considering that the damping ratios are at the low bound of those measured by others as indicated in Figure F-6 the use of these damping ratios is considered to be conservative for backfill soils.



Project: SONGS UNIT 1 SEISMIC RE-EVALUATION **Project No.** 413521	VARIATION OF SHEAR MODULUS WITH STRAIN FOR SAN MATEO SAND	**Fig.** F-1

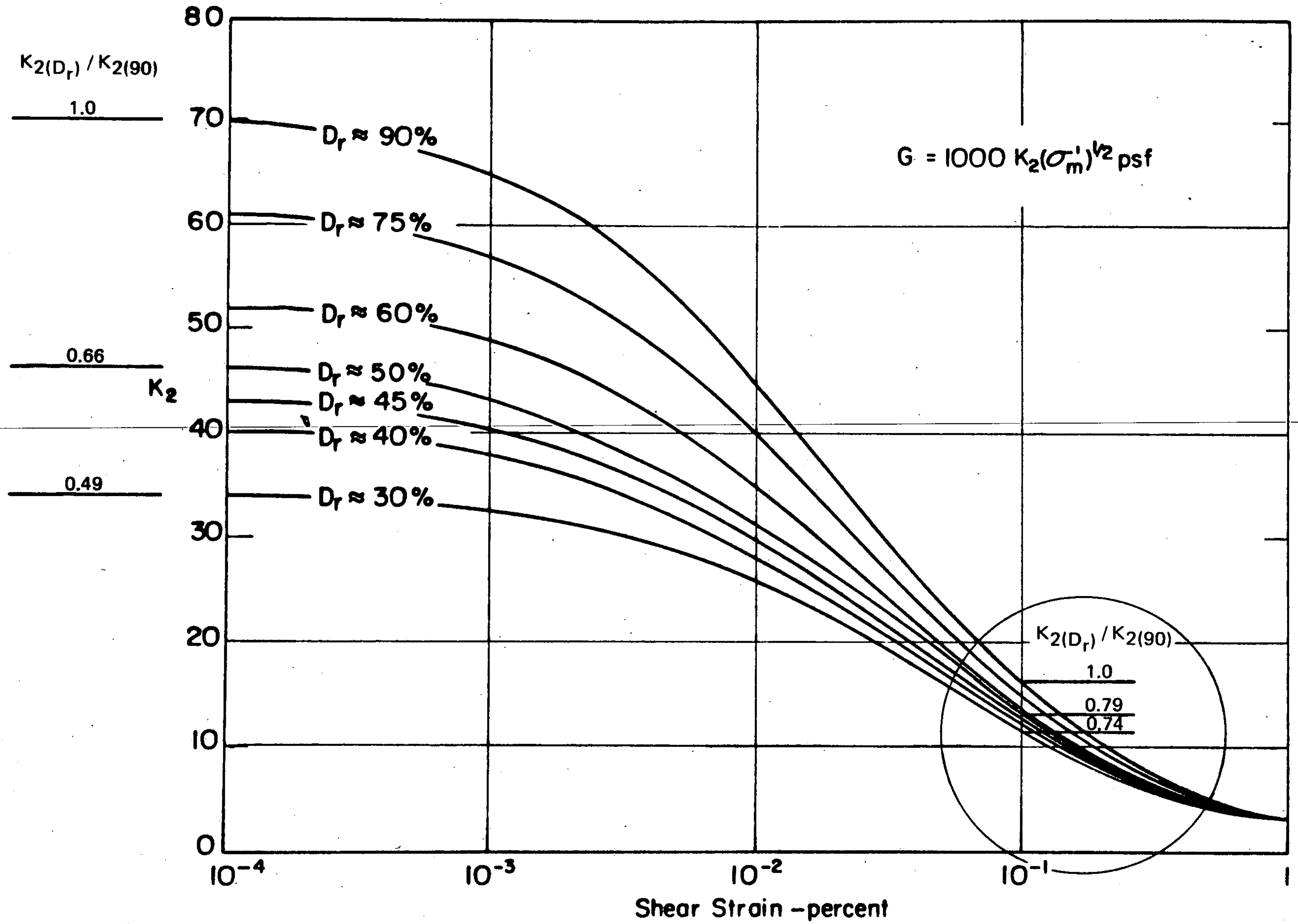
WOODWARD–CLYDE CONSULTANTS



*Judged to be the 5 highest strain pulses in each 5 seconds of strain — time history (consistent with 10 equivalent cycles of loading for a M7 earthquake).

Project: SONGS UNIT 1 SEISMIC RE-EVALUATION **Project No.** 413521	SUMMARY OF DOMINANT SEISMIC INDUCED STRAIN	**Fig.** F-2

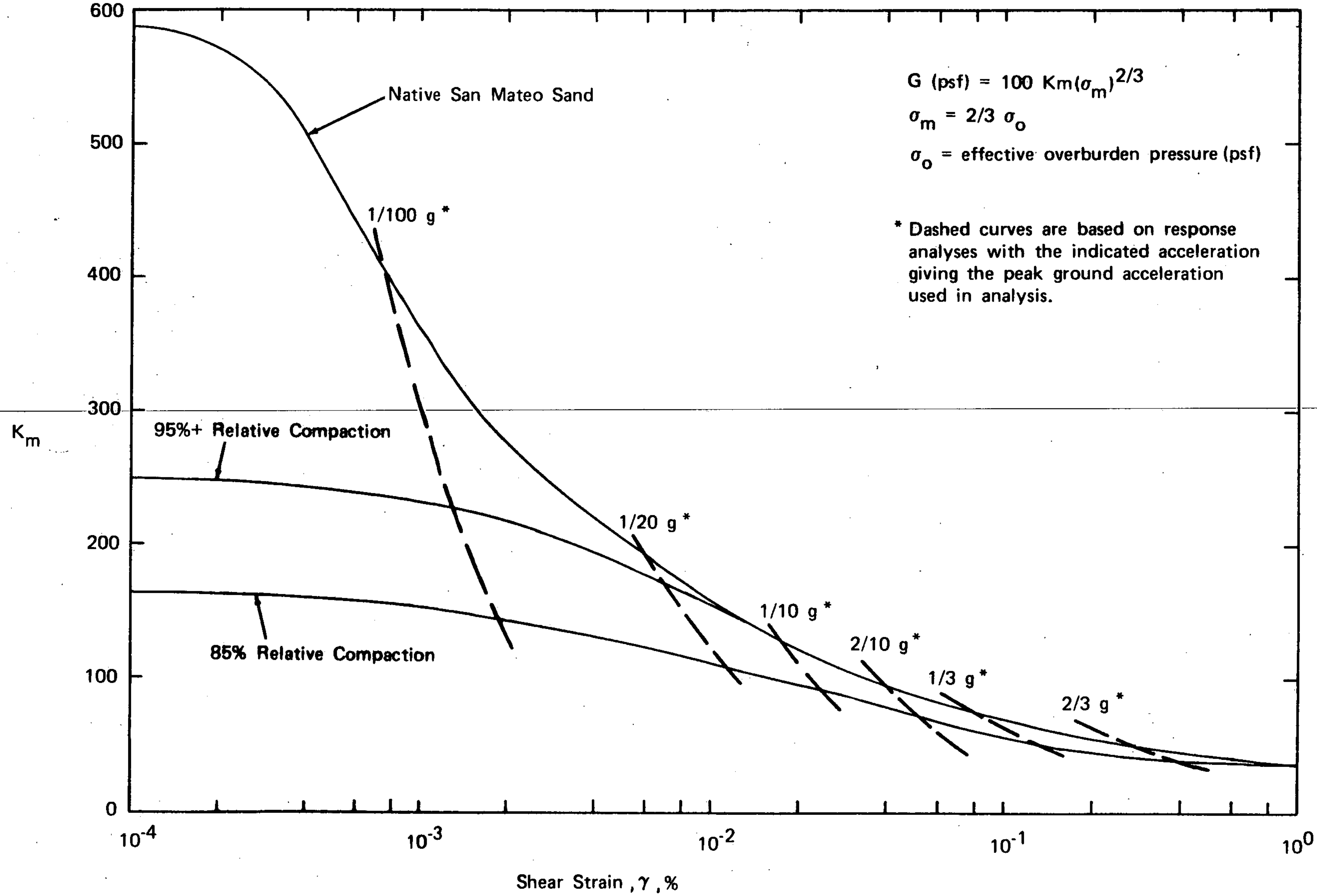
WOODWARD—CLYDE CONSULTANTS



Ref. "Seed, H.B. and Idriss, I.M.; "Soil Moduli and Damping Factors for Dynamic Response Analysis", Report No. EERC 70-10, Earthquake Engineering Research Center, University of California; Berkeley, California; 1970.

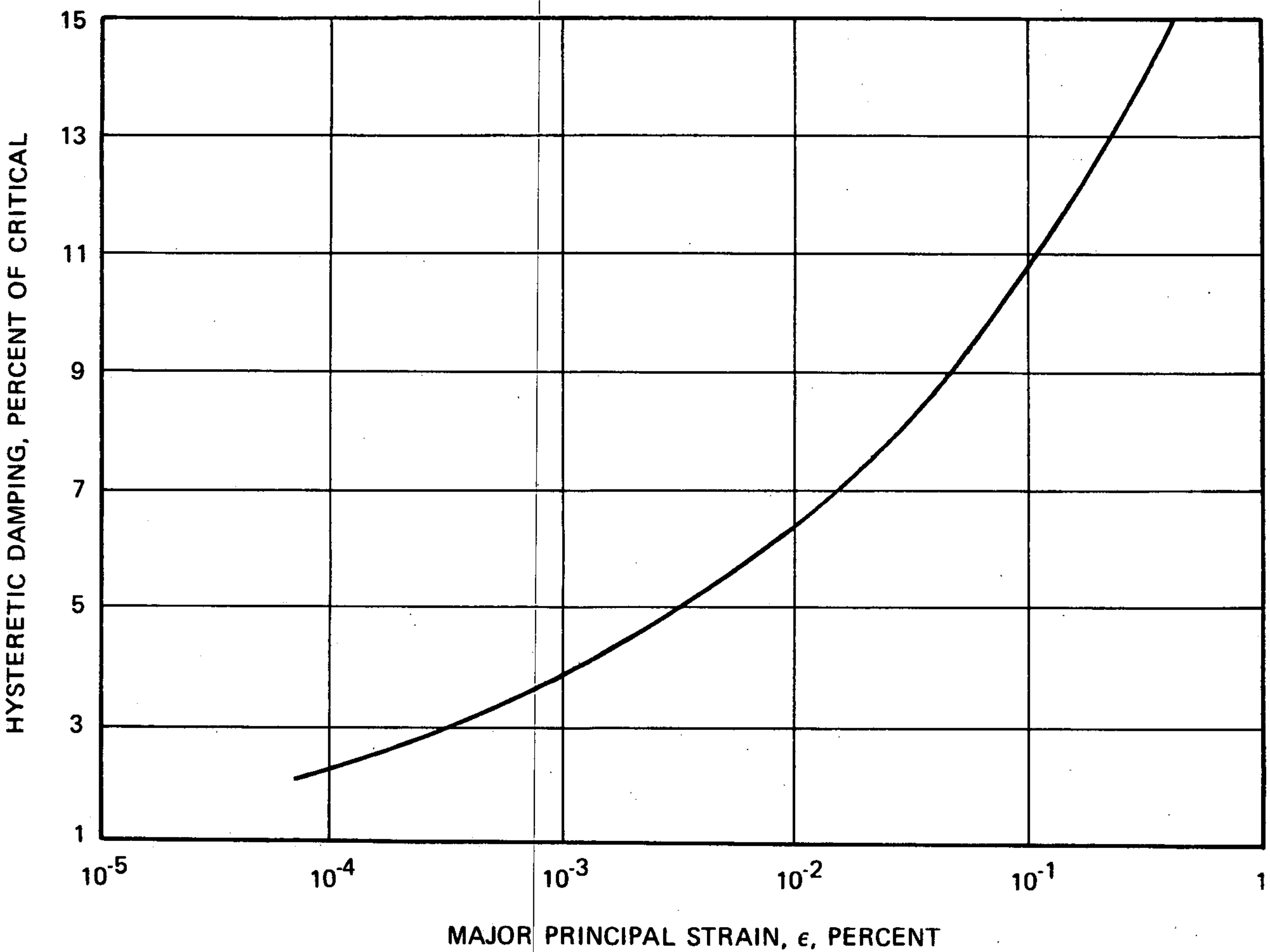
Project: SONGS UNIT 1 SEISMIC RE-EVALUATION **Project No.** 413521	SHEAR MODULI OF SANDS AT DIFFERENT RELATIVE DENSITIES	**Fig.** F-3

WOODWARD–CLYDE CONSULTANTS



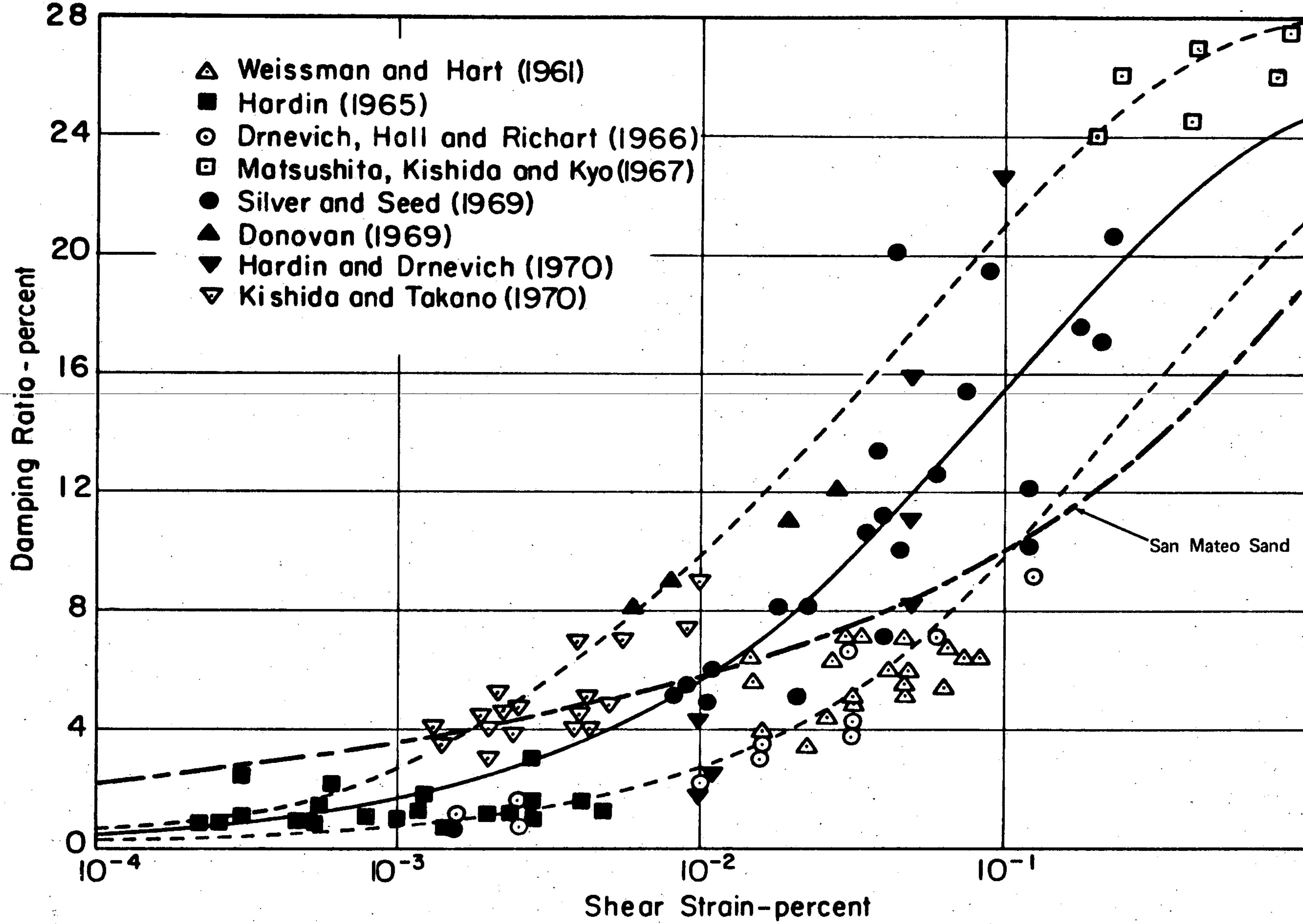
Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 41352I	MODULUS AS A FUNCTION OF STRAIN AND PEAK GROUND ACCELERATION	Fig. F-4

WOODWARD–CLYDE CONSULTANTS



Project: Project No.	SONGS UNIT 1 SEISMIC RE-EVALUATION 413521	DAMPING RATIO VS MAJOR PRINCIPAL STRAIN SAN MATEO FORMATION SAND	Fig. F-5

WOODWARD–CLYDE CONSULTANTS



Project: SONGS UNIT 1 SEISMIC RE-EVALUATION Project No. 413521	DAMPING RATIOS FOR SANDS (after Seed and Idriss 1972) COMPARED TO THE SAN MATEO SAND DAMPING RATIOS	Fig. F-6

WOODWARD–CLYDE CONSULTANTS