ENGINEERING DEPARTMENT

SOUTHERN CALIFORNIA EDISON COMPANY
Los Angeles, California

REPORT ON FOUNDATION INVESTIGATION

AT THE

SAN ONOFRE NUCLEAR GENERATING STATION SITE

Report No. 176

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TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF APPENDICES

FOUNDATION INVESTIGATION
SAN ONOFRE NUCLEAR GENERATING STATION

UNIT NO. 1
CAMP PENDLETON, CALIFORNIA

A. INTRODUCTION

This report presents the results of the foundation investigation authorized by the Mechanical Division, Nuclear Design Section per Form 68, dated March 1, 1963.

The purpose of this investigation was to develop site grading and foundation engineering data pertinent to the design, construction, operation and maintenance of the San Onofre Nuclear Generating Station facilities.

The scope according to the "Definitive Scope of Work" was interpreted to include the following:

1. Undertake field exploration, laboratory testing program and office studies to provide specific information relative to (a) the design of foundations for all structures, and (b) site grading requirements with particular reference to stability.

2. Investigate ground water conditions at the site and develop conclusions regarding (a) the possible effects of ground water upon dewatering operations for deep excavations during construction, and (b) the configuration of the water table in the vicinity of the site as it may influence the nature and range of movement of possible groundwater contamination.

3. Perform geologic field and office work as required to provide a detailed analysis of local geologic conditions.

4. ✓ Perform aseismic investigation and provide recommendations regarding earthquake design criteria for use in structural design.

Prior to this investigation, a preliminary site investigation was performed, the results of which were submitted in a report entitled, "Coast Nuclear Steam Station, Site C", Report 167, dated November 8, 1962. A copy of that report is included in Appendix A.

B. SITE DESCRIPTION

The proposed site is located on Camp Pendleton Marine Corps Base approximately five miles south of the City of San Clemente, in San Diego County. The property consists of approximately 90 acres extending 5,000 feet along the coastline between U. S. Highway 101 and the beach. The main line of the A.T.&.S.F. Railroad is adjacent to and easterly of the highway.

As a result of natural drainage and attendant erosion, the site is traversed by deeply cut barrancas, extending from the highway at an elevation of approximately 90 feet to the beach at an elevation of approximately 20 feet (Datum is MLLW). The area adjacent to the barrancas on both sides comprises a narrow, gently sloping coastal plain extending seaward from the uplands and terminating abruptly at the shoreline by a sea cliff up to 70 feet high. A narrow beach separates the cliffs from the ocean. The side slopes of the barranca and bluff in the immediate vicinity are very steep, ranging from 60 to 80 degrees from the horizontal.

The subsurface soils underlying the site to the explored depth of 340 feet consist of two major soil formations:

1. The upper formation between the approximate elevations of 90 and 50 feet (MLLW) consists of terrace deposits which range in gradation from clayey sands to cobble layers.

2. The lower formation extending to the explored elevation of -253 feet (MLLW) and to an estimated depth of 500 to 1000 feet, consists of San Mateo sands, which are buff colored, well graded, and very dense.

The surface layer of the offshore sediments (See Preliminary Site Investigation in Appendix A) consists of tightly packed gravels, cobbles and boulders, varying in thickness from one to three feet with local pockets of sand up to 10 feet thick overlying the San Mateo sand formation. The distribution of the offshore sediments varies from place to place and is strongly influenced by wave and current conditions.

The barranca presently serves as a natural drainage outlet to the ocean for a highway culvert which drains the area north of the highway.

The average groundwater table at the site is approximately at elevation of 5 feet (MLLW) and has a gradient towards the ocean.

C. PROPOSED STRUCTURES

The proposed facility is a 395 MW closed cycle water-cooled nuclear power plant. Pertinent structures as shown in Figure 1 include: Containment sphere, turbine-generator, spent fuel storage building, reactor auxiliary building, feedwater heater platform, storage tanks, intake structure, switchyard components, access roads, railroad spur, sea wall, and other more minor structures.

Extensive earthwork operations involving cuts up to 70 feet high are required in the preparation of the foundation sites for the various structures. Elevations for the foundations vary from the lowest at -26 feet for the containment sphere and intake structure to the highest at +90 feet for the switchyard facilities. Elevation of the plant area is +20 feet. All elevations are based on the MLLW datum.

Structural loads estimated by Bechtel Corporation are as follows: Column loads for major structures will range from a minimum of 200 kips to a maximum of 2000 kips, including dead plus live loads.

The total of all loads for major structures may vary up to a maximum of 2900 kips.

For the containment sphere, the maximum column load will be 300 kips during construction. After construction, a uniform bearing pressure of 5 kips per square foot may be expected at the bottom of the sphere for dead plus live loads.

D. RECOMMENDATIONS

1. Foundation Design for Major Structures

All major structures may be supported by spread footings founded on the undisturbed San Mateo sand which possesses extremely high bearing capacity values. The recommended bearing values are presented on Figure 4. The values shown on the chart may be used for the total of dead plus frequently applied loads. They may be increased by one-third when considering the total of dead, live, and wind or seismic loads. The computed factor of safety against shear failure is approximately four.

Lateral forces imposed on the foundations may be resisted by the combination of (1) frictional resistance between the base of the footings and the underlying soils, and (2) passive pressures of the soil on the sides of footings. A coefficient of friction of 1.0 may be used for design. The passive pressures may be assumed to be be equal to the lateral pressure imposed by an equivalent fluid weighing 750 pounds per cubic foot. These design values should be reduced by an appropriate factor of safety.

Footing excavations above the ground water table will probably stand safely at vertical slopes without lateral support. Excessive seepage pressures, however, that could develop during prolonged dewatering operations may make it necessary to provide lateral support for the foundation excavation below ground water.

Because of (1) the extremely low compressibility of the foundation soils, and (2) high preconsolidation stresses due to extensive earthwork operations, the estimated total settlements for the heaviest foundations will be quite small, probably not greater than one-half inch. The corresponding differential settlements would be significantly smaller. As the foundation loads are applied, the settlements will occur almost instantaneously.

2. Foundation Design for Switchyard Structures

Foundations for switchyard structures may be supported by spread footings, founded on fill compacted to at least 95 percent of the maximum dry density, as determined by the ASTM Designation D 1557-58T, Method C laboratory compaction test procedure. (See Section G for detailed description). Material consisting of either the upper terrace deposits or the San Mateo sands may be used for fill.

The recommended bearing values for the total of dead plus frequently applied loads are presented on Figure 2. They may be increased by one-third when considering the total of dead, live and wind or seismic loads. The computed factor of safety is approximately four.

Where overturning loads on switchyard structures are great, drilled cast-in-place concrete piling may be used. For computing the lateral capacities of short piles, an allowable bearing value of 1600 pounds per square foot of depth may be used.

3. Slope Stability

Based on results of stability analyses and field observations of the natural slopes in the immediate area of the site, it is recommended that slopes up to 70 feet in height be cut to 1/2 horizontal to 1 vertical, provided a bench 15 feet wide is cut at the top surface of the San Mateo formation. Since the slope materials, especially the terrace deposits, are susceptible to erosion, measures for slope protection to control erosion should be provided. Included in such measures should be lined drainage ditches parallel to the edges of both the upper and lower slopes to intercept surface runoff before going over the respective slopes. Methods of stabilizing the slope-forming materials to improve the resistance against erosion may include planting suitable ground cover, guniting, chemical stabilization or other equally effective method.

Excavations below the groundwater table to the same slopes stated above should remain stable during the construction period provided significant seepage pressures do not develop as a result of dewatering operations.

Fill slopes consisting of terrace materials or the San Mateo sands should not be greater than 1:1. The terrace materials should be placed in loose lifts not more than eight inches in thickness, brought to optimum moisture content, and compacted to at least 95% of the maximum density obtained by the ASTM Designation D1557-58T, Method C laboratory test procedure. See Figure C-7 for compaction curve.

The San Mateo sands, if utilized for embankment purposes, should be compacted so as to achieve a relative density of at least 95% of maximum dry density obtained by ASTM Designation D1557-58T, Method C laboratory compaction test procedure. Compaction moisture is not critical.

4. Pavement Design

For those areas that will eventually be paved, among which are included access roads from the main highway to the plant area and various visitor and construction parking areas, it is envisioned that the traffic will include various classes of traffic with single axle loads up to 18,000 pounds. Recommendations for the total thickness of asphalt pavement structure required when placed directly above either of the two soil formations to support various traffic loads are summarized below in Table 1.

Table 1. Total Thickness of Asphaltic Pavement Structures

Traffic Classification	Daily Volume Per Lane of Passenger Cars and Light Trucks (Single Axle Loads Less Than 6,000 pounds)	Daily Volume Per Lane of Commercial Trucks (Single Axle Loads up to maximum of 18,000 pounds)	San Mateo Formation (Inches)	Terrace Deposits (Inches)
Heavy	Unlimited	250	4	8
Medium	500	25	3	7
Light	25	5	2	5

A minimum three-inch asphaltic concrete layer is recommended to sustain the maximum indicated single axle loading. San Mateo sand compacted to a minimum of 95% of maximum dry density based on the ASTM Designation D1557-58T, Method C, may be used as base course material above the terrace deposits. No imported base material is required.

5. Disposal of Excess Excavated Materials

Grading operations involving deep cuts and minor fills will result in large quantities of excess materials which must be disposed of in an economical manner. The soils at the site are considered highly suitable for highway embankment, base materials for pavements, beach replenishment and for other uses demanding select fill. Disposal of these materials into the ocean should be considered only after all other possibilities for more beneficial utilization have been exhausted.

However, should disposal of excess materials into the ocean prove to be most economical, the sedimentation characteristics of the excess materials, as determined by special hydrometer tests employing a salt water settling medium, indicate that over 90% of all soil particles will have settled in 20 minutes. After one hour only approximately two percent of the soil particles remain in suspension. Reference is made to the memorandum, entitled "Sedimentation Characteristics of the Soils at San Onofre," which was submitted to Mr. P. M. Horrer of Marine Advisers, Inc. for evaluation of effects of such disposal activities on the aquatic life in the adjacent ocean waters. A copy of this memornadum is found in Appendix C.

6. Seismic Design Factors Developed by Dames & Moore

A very comprehensive seismic investigation of the site was conducted by Dames & Moore. The full report is included herein as Appendix D. Briefly, the more pertinent conclusions and recommendations concerning seismic history and engineering seismology of the site are summarized below:

a. Seismic History

The seismic history of the Southern California area dates back to 1769. Some 22 earthquakes, since that date, large enough to have been strongly felt at the site, have been studied. Catalogues and lists of other Pacific Coast earthquakes have been reviewed to describe the probable intensity of historic earthquakes at the site.

The broad conclusion that can be developed from the seismic history of the site is that, for an adjusted period of 100 years, there were six Intensity VI quakes, two Intensity VII quakes, and one Intensity VIII quake. An Intensity IX quake was not experienced at the site during the period of recorded history.

b. Earthquake Epicenters

The distribution of earthquake epicenters, which is indicative of the seismicity of a particular area, was reviewed from published U.S.C.&.G. survey information. From such studies, it is concluded that the site is relatively clear of epicenters, and that occurrences of earthquakes have been much less frequent and much less severe at the site than at the neighboring communities of Los Angeles, Bakersfield, Santa Barbara, and El Centro. It is concluded from this survey that areas encompassing known, active faults, possess clusters of epicenters because of the presence there of such faults. The San Onofre site is some 25 miles from known active faults.

c. Intensity Values

The intensity and corresponding expectancy of earthquakes at the site have been determined from historical data. It is concluded that the maximum intensity values expectable at the site may be based on the following:

1. The expectancy of an Intensity IX earthquake in Southern California is once per 3,000 years, based on recorded shocks of this intensity and equal seismicity in all of Southern California.

2. The historical earthquake activity at the site is about five times as great as the average for Southern California, although other areas are many more times as active.

3. Therefore, an Intensity IX might be expected at the site once per 600 years, although such intensity level has not occurred at the site within historic time.

4. One Intensity VIII earthquake can be expected once per 100 years.

d. Engineering Seismology

Dr. Kyoshi Kanai of the Earthquake Research Institute in Tokyo, has developed a procedure based upon wave propagation theory and subsurface conditions, which was used to determine probable ground motion at the site.

This method takes into account:

1. Earthquake magnitude,

2. Epicentral distance, and

3. The elastic and physical properties of the underlying soils and/or rock.

From the calculations, a selection was made of the largest shallow shocks that would produce the intensities derived or estimated from the historical earthquake studies. The design earthquakes are as follows:

1. A magnitude 6-1/2 shock on the Elsinore, San Clemente or Newport-Inglewood faults, or

2. A magnitude 7-1/2 shock on the San Jacinto fault, or

3. A magnitude 8 shock on the San Andreas fault north of Cajon Pass.

It appears that the smaller shocks (M= 6-1/2 at a distance of 25 miles from the site) would be the most critical in the short period range of the spectrum. Such design earthquake would be classified as a "once-per-600-year quake" and would develop a calculated maximum ground acceleration of 0.25 g at a period slightly less than 0.3 second.

Such earthquake would result in about Intensity IX at the site. A M = 8 shock on the distant San Andreas would develop a similar acceleration and an Intensity of about IX.

e. <u>Aseismic Design Factor</u>

The recommended maximum ground motion acceleration value is as follows:

> 0.25 g for an earthquake having an intensity of about IX once per 500 to 1000 years at the site.

It is the prerogative of the designer to select overload factors to accommodate differing levels of required safety. For ground motion and response spectra see Appendix D.

7. Sea Wall

Sheet piling, if used for sea wall construction, will require jetting during placement. The development of passive pressures below the scour level will not be seriously influenced by jetting operations. For piling design, the passive resistance may be assumed to be equal to the lateral pressure imposed by an equivalent fluid weighing 600 pounds per cubic foot above the ground water table.

Lateral active pressures imposed by the soil may be obtained by using Rankine's conventional relationship where due consideration should be given to the physical characteristics of the soils utilized for backfill material.

8. Dewatering

It is recommended that a well point system be used during the excavation of those portions of the foundation which will be below the water table. Well points are best adapted to dewatering in formations of medium permeability such as exists at the site and can be easily jetted into the San Mateo sand formation. The water table at Unit 1 is at elevation +5 feet MLLW and dewatering will be necessary down to an elevation of -31 feet MLLW, which is five feet below invert grade.

Analysis of data from a pumping test at the site indicates that permeability values between 25 feet per day and 180 feet per day should be used for foundation dewatering studies.

9. Offshore Construction

The offshore sediments should present no unusual problems during the construction of cooling water lines or sea walls. Investigation shows that only a thin sand, cobble and boulder veneer covers a wave cut terrace of dense San Mateo sand. The composition and thickness of this veneer, however, is subject to change depending on changing wave conditions. The thin cobble and boulder layer and the underlying sediments were easily penetrated by jetting; therefore, the installation of piles or the placement of sheet piling during construction should not be difficult. Excavation of the pipeline trench can be accomplished by the use of conventional equipment. It is anticipated that trench side slopes will stand as steep as 3:1 or 4:1.

10. Piezometer Observation Program

It is recommended that the four main observation wells (T.H.'s 6, 8, 9, and 10) be maintained as permanent observation wells. Water level observations were made and water samples were taken during the foundation investigation as a basis for future comparisons. These 2 inch plastic pipe piezometers can be used to check the elevation of the groundwater table for future studies of direction and gradient of groundwater flow. These wells may also be used to sample the groundwater for chemical quality and radioactivity determinations.

Frequent water level observations should be made during construction so that the effects of changes in ground water quality or gradient can be evaluated.

E. GEOLOGY

1. Geomorphology

The coastal zone near the site consists of a rather narrow plain which slopes southwesterly from the mountains and terminates abruptly at the shoreline where high coastal cliffs have been cut into the underlying sediments. Only a very narrow band of beach sand separates the ocean from these cliffs. The top of the terrace ranges from above elevation +200 near the mountains to about +90 at the bluffs.

In places, ephemeral streams are actively eroding gullies into the uncemented terrace deposits of the plain, and many deep barrancas have been cut into the soft sediments from the beach as far back as Highway 101.

About 1 mile north of the site the westerly flowing San Onofre Creek crosses the plain and discharges into the ocean near San Onofre. The more gentle gradient near the coast has allowed lateral cutting of the stream banks and the formation of a valley nearly 1/4 mile in width. This valley forms a natural groundwater basin which furnishes water for the San Onofre Recreation Area and for truck farming in the valley.

2. Stratigraphy

Within close proximity of the site are the following distinct lithologic units: (1) The Capistrano formation of Lower Pliocene age, (2) The San Mateo formation of Pliocene age, (3) Pleistocene terrace deposits and (4) Recent alluvial sediments and beach sand. Older units although shown on the geologic map are not pertinent to the foundation investigation of this site and are not discussed in this report.

a. Capistrano Formation

Along the beach starting about 1 mile south of the site, outcrops of the Capistrano formation have been exposed by vertical movement along the Cristianitos fault. The formation was also recorded in the log of the R1 well about 1 mile north of the site (see Geologic Map) at about elevation -114 feet. This deep occurrence of Capistrano formation is probably related to uplift along the fault zone.

The Capistrano formation, as seen along the coastline and as logged in borings made by Edison at Horno Canyon in 1960, varies from a gray, thin-bedded, fissile shale to a dark green or black, massive siltstone, with stringers of gypsum and layers of fine sand. The Capistrano formation can often be distinguished by thin layers of jarosite, slickensided clay layers, and great numbers of fish scales. The thickness of this formation is estimated by Dames & Moore to be between 1,000 and 1,200 feet in the vicinity of the site.

b. San Mateo Formation

The San Mateo formation is present along the coast west of the Cristianitos fault and south of San Mateo Creek in an undisturbed sequence of Pleistocene terrace materials overlying Pliocene ? San Mateo sand.

The formation is reported to have a slight regional dip to the northwest but because of the massive character of the formation no definite bedding attitudes were observed at the site.

The San Mateo formation is a massive, well graded, fine to coarse sand with occasional pebbles to 1 inch diameter. Grains consist of subrounded quartz and feldspar with some dark rock fragments. The sand is buff colored, and lightly cemented by a hydrophillic clay.

Lenses of very dense silt varying from a few inches to several feet in thickness occur throughout the formation but lateral extent of these layers appears to be limited. The San Mateo formation is distinctive because of its buff color, lack of bedding, and uniformity of grain size both laterally and vertically.

Estimates of thickness vary from 500 to 1,000 feet. Information from Test Hole 1 and from observations made in the barranca near Unit 1 indicate a thickness of at least 325 feet at the site. Because the Capistrano formation was encountered west of the fault zone in the R1 well at 164 feet below the ground surface, it appears probable that the thickness of San Mateo formation may be closer to 500 feet than 1,000 feet in the vicinity of the site.

c. Terrace Deposits

The Pleistocene terrace deposits are light brown, silty to clayey fine sands with layers of gravel, subrounded cobbles and occasional small boulders. The formation is crudely stratified with alternating fine and coarse layers.

The deposits at the site are generally about 40 feet thick extending from about elevation +50 at the top of the San Mateo formation to +90 on the top of the bluff. Consolidation tests on undisturbed samples from the site show the terrace deposits to be highly over consolidated which indicates there was a much greater thickness of terrace material in the past. This is substantiated by terrace outcrops on the foothill slopes at elevations above 200 feet.

These terraces extend many miles to the north and south along the coast. They are described as being partly of marine and partly of fluvial origin. The deposits are continuous and unbroken across the fault contact between the San Mateo formation and the Capistrano formation.

Recent Alluvium and Beach Sand

Recent alluvium in the San Onofre Creek basin consists of alternating layers of sand, gravel, silts and clays. These alluvial deposits are important in any consideration of groundwater because of the cut and fill relationship of stream deposits with the San Mateo formation. The stream bed deposits are almost certainly in hydraulic continuity with the underlying sands. All five of the Marine Corps wells shown on the Geology and Groundwater Contour Map encountered San Mateo formation at elevations ranging from -10 feet MSL in Well R3 to -83 MSL in Well D1. Logs of borings indicate the deepest part of the channel to be somewhat south of its present location.

A thin veneer of beach sand and cobbles covers the San Mateo formation along the shore below the bluffs. At Test Hole 7 the sand is only about 3 feet thick. This sand is gradually moving south and the depth of sand at the site is continually changing and at times thick beds of cobbles and boulders, eroded from the terrace deposits, are exposed on the beach and in the surf.

3. Faulting and Landslides

No indications of active or inactive faults and landslides were observed on the Edison property as evidenced by the lack of offset formation contacts or fracture planes in the numerous exposures in the vicinity of the site. Careful inspection of the surf zone, sea cliff face and barranca walls show a uniform sequence of San Mateo formation with overlying Pleistocene terrace deposits. The only faulting observed in the San Onofre Basin area was the pre-Pleistocene disturbance of Pliocene and older rocks along the Cristianitos fault zone about a mile south of the site.

The Cristianitos fault intersects the coastline approximately one mile southeast of the plant site. The fault extends from the coastline in a northerly direction through the low foothills and on east of the City of San Clemente.

An excellent exposure of the fault can be seen on the vertical sea cliff at the beach. At this point a narrow fault zone separates San Mateo sand from the uplifted Capistrano siltstones and shales. Attitudes taken on the beach show the fault plane striking N57°E and dipping 72° to the NW. The fault plane has been truncated by post faulting erosion and now horizontally stratified terrace deposits lie unconformably over both the Capistrano formation and the San Mateo formation.

Careful observations made on the beach show that no vertical movement of the fault has occurred at this point since the San Mateo sand and the Capistrano formation were cut by transgressive seas to a common level. This indicates that faulting has not occurred since before the deposition of the terrace deposits and this dates the fault as being inactive for at least the last 10,000 years, and probably for 35,000 years. Independent observations made by geologists on other portions of the Cristianitos fault substantiate these minimum ages for the fault.

Total vertical movement along the fault at this point is at least 500 feet and probably greater than 1,000 feet.

Near the Cristianitos fault zone several associated minor normal faults and intersecting joint planes were observed in the San Mateo formation. The San Mateo sand, at this point, is highly cemented and very dense.

In the Capistrano formation, contorted, highly fractured, slickensided beds appear to represent drag folding in the relatively incompetent shales. Such minor faulting, folding, and jointing were undoubtedly associated with interformational faulting.

South of the fault, along the coastline, there are evidences of both deep seated and shallow, active landslides. Evidences of complex, deep seated sliding there includes the presence in the surf zone of (1) contorted and locally sheared landward dipping Capistrano beds which vary through some 90° of strike and which dip easterly from about 20° - 30° to nearly vertical, and (2) downdropped blocks of Pleistocene terrace materials which are now being actively forced seaward and upward along deep surfaces of rupture.

The generally hummocky nature of the surface inland from the surf zone and the presence of springs and phreatophytes are also evidences of landslide phenomena. The slide areas are also generally marked by the presence of cobbles in the surf zone. The cobbles were derived from the Pleistocene terrace deposits which have been down-dropped by slide activity, and which have been subsequently subjected to beach erosion.

4. Offshore Geology

Offshore jet probing, along the proposed Unit 1 intake structure, was performed by Pacific Towboat and Salvage in September, 1962 in order to determine the types of sediment which will be encountered during the excavation of the offshore portion of the cooling water lines.

Small diameter jet pipe, with water flowing under high pressure, was used to penetrate the bottom sediments. In most cases, the ocean floor could be easily jetted after a surficial hard layer was penetrated. The overlying dense layer was later identified by diving geologists as being composed mostly of cobbles and boulders.

The probe locations and depths are summarized in Section IV of Appendix A in this report.

The geologic exploration of the sea floor offshore from the Edison property was conducted by General Oceanographics, San Diego, California. Reference is made to their report titled "Sea Floor Geology and Sonoprobe Survey of an Area off San Onofre, California" and dated October 21, 1962.

This survey consisted of both underwater geologic mapping and bottom penetrating echo soundings with an acoustic reflection device called a Sonoprobe. Geologic and Sonoprobe surveys indicate that the near shore ocean floor consists of a gently seaward sloping surface composed of cobbles and boulders with an overlying patchy, thin sheet of sand. Sonoprobe readings indicate this layer of cobbles and boulders in places attains a thickness of at least 3 feet and possible 10 feet. The results of the sonoprobe survey and the jet probing indicates that San Mateo formation lies below the cobble and boulder mantle.

The cobbles and boulders noted in the offshore, underwater geologic traverses ranged in diameter from 6 inches to over 2 feet with the largest rocks located near shore. The cobbles and boulders were sandstone, shales, volcanics and metamorphics, which indicates they were probably derived mostly from the erosion of the terrace deposits.

Hydrographic surveys were undertaken by Pafford and Associates in October of 1962 and July and August of 1963. The offshore contours for 1962 are shown on drawing E-17284 and for 1963 on drawing E-72170.

F. FIELD EXPLORATION

1. Exploratory Drilling

Preliminary exploratory drilling was performed by the J. L. Helton Drilling Company in September and October of 1962 and final foundation exploratory drilling was done by the J. N. Pitcher Drilling Company in May of 1963. Logs of borings are contained in Appendix B.

A total of 14 test holes has been drilled at the proposed site. Locations of all borings completed to date are shown on Figure 1. The test hole depths, locations and elevations are summarized below:

Table 2. Location of Borings

Test Hole Number	Location	*Elevation Ground Surface MLLW	Depth
1	N440,349 E1,600,820	+22	395
2	N440,209 E1,601,898	+98	155

Test Hole Number	Location	*Elevation Ground Surface MLLW	Depth
3	N439,008 E1,603,443	+97	340
4	N440,447 E1,600,898	+35	140
5	N440,373.5 E1,601,028.0	88.2	125
6	N440,610.5 E1,600,599.5	88.7	126
7	N440,192 (Approx.) E1,600,871 (Approx.)	12± (Est.)	55
8	N440,833.0 1,600,695.0	91.7	125
9	N440,488.5 E1,601,718.5	98.9	125
10	N439,936.5 E1,600,615.5	88.2	125
11	N440,236.5 E1,600,814.5	*14.80	50
12	N440,290.0 E1,600,757.5	15.99	50
13	N440,277.0 E1,600,815.0	15.09	49
14	N440,351.5 E1,600,831.0	22.7	50

*Elevations for Test Holes 6 and (8 to 14) are for tops of piezometers. Other elevations are for ground surface.

Final foundation exploration tests holes 5-14 were drilled in May, 1963 by the J. N. Pitcher Drilling Company, using a Failing 1000 and a Failing CFD-1 rotary drill rigs. The holes varied in diameter from 4-7/8" to 6" and in depth from 49' to 126'.

Twenty-five undisturbed soil samples were taken using a Pitcher rotary core barrel during the final investigation. Samples recovered in the 3" diameter tubes varied in length from 6" to 30". Good recovery was obtained in the silty layers of the terrace deposits, however, in the San Mateo formation recovery was only fair and sample lengths averaged 12" to 18".

Penetration tests were made in the terrace and San Mateo formation using a standard 2" O.D. split tube penetrometer driven by a 140 pound weight falling 18". Cumulative blow counts were recorded at 6" increments. Penetration resistance tests are shown on the drill logs and summarized under the section on Engineering Characteristics.

Test borings in the terrace deposits at about elevation +90 generally encountered dense, silty fine sand in the top 10 to 15 feet. In the borings near the proposed Unit I containment sphere, dense cobble layers can be seen in the cliff face in the barranca, and on the beach in the vicinity of Unit I. The upper layer is generally about 10 feet to 20 feet below the top of the terrace and the lower layer extends from 30 feet below the top of the terrace to the top of the San Mateo formation.

The cobble layers contain rounded cobbles to about 8" diameter with occasional 12" boulders in a matrix of gravel, sand, silt and clay. The cobbles are mostly hard metamorphic rocks which were eroded from the San Onofre formation.

Drilling in the terrace deposits was very difficult because of hard rounded cobbles and it was necessary to case Test Hole 5 to 38 feet in order to stop caving in the cobble zones. In most instances, penetration tests and core sampling were discontinued in the cobble layers and were not resumed until the San Mateo formation was encountered and the cobble zones sealed off. These cobble layers appear to thin out down coast and easy drilling was encountered in the terrace deposits at Test Hole 8.

In all of the borings near Unit I, the contact between the San Mateo formation and the overlying terrace deposits was found between elevations +43 and +54. The contact as seen in the barranca and along the beach appears to be at elevation +50 with a slight dip toward the ocean. Because of caving and difficult drilling conditions, cliff face exposures are more reliable than drill logs for determining the elevation of the formation contact.

Drilling in the San Mateo formation was very fast and drilling rates of 1 foot per minute were common. Mechanical analyses show the sand to be homogeneous and uniform with depth with occasional lenses of silty fine sand or dense clayey silt. In most cases, layers of fines encountered in one boring could not be correlated with adjacent borings. However, in Test Holes 11 and 13 and in the 10 inch diameter test well, a 2 foot silt layer was encountered at about elevation -33 feet.

2. Ground Water and Permeability Studies

A. Piezometers and Observation Wells

Two inch diameter, plastic pipe piezometers, 125 feet deep were set in Test Holes 6, 8, 9 and 10 during the final field exploration. These observation wells were perforated below the water table, gravel packed, flushed with water and developed with compressed air. Samples of ground water from TH 8 were analyzed and are shown in Appendix E.

These observation wells were used to calculate ground water gradients and can be used in the future to obtain ground water samples for radioactivity, chemical and biological quality determinations.

Ground water data for the San Onofre Basin was obtained from the Office of Ground Water Resources at Camp Pendleton. The Marine Corps furnished logs and water levels for 5 wells to the north and west of the site. These wells and ground water contours based on Marine Corps and Edison Company Observation wells are shown on Figure 2, Geology and Ground Water Contour Map.

The ground water gradient between the site and the R-1 well is approximately .0032 or about 17 feet per mile. The contours indicate a flatter gradient between the H-1 well and the site. This is probably due to the higher permeability in the San Onofre Stream deposits which allows the water to drain westerly toward the ocean rather than south toward the plant site.

It appears that a normal ground water gradient from the site to the San Onofre Creek Basin will not occur. The Marine Corps policy is to maintain the ground water table throughout the camp above Elevation +5 MSL or Elevation +7.6 MLLW because of the danger of saline water intrusion into the fresh water aquifer. The water table at the site is slightly below Elevation +6 MLLW at Test Holes 8 and 9 so that even under extreme pumping conditions in San Onofre Creek, the ground water flow will be toward the ocean. Ground water gradients should be carefully observed, however, so that activities at the plant site do not disturb the existing flow toward the ocean. Ground water depth measurements are shown in Table E-1.

B. Field Permeability Test

Field permeability studies were required to determine effects of ground water on dewatering operations for deep excavations during construction. It is estimated that dewatering, down to about elevation -30, will be necessary for the intake structure and part of the plant.

It was determined that a single test well with surrounding observation piezometers would be effective and the most economical method for determining permeability values of the San Mateo formation.

During the foundation exploration in May 1963, four 50-feet deep, 1-1/2 inch diameter, plastic pipe piezometers, (Test Holes 11-14) were placed in the barranca near Unit I for future use in permeability studies for site dewatering. The piezometers, were perforated below the water table, gravel packed and developed with compressed air. An analysis of water from Test Hole 13 is shown in Appendix E.

In August 1963 a 10-inch diameter gravel packed test well, 50 feet in depth, was drilled between Test Holes 11 and 14 as shown in Figure I. The 24 inch diameter hole was drilled by the J. L. Helton Drilling Co. with a bucket auger. The hole was overdrilled to 55 feet and the bottom 5 feet was backfilled with 2 inch gravel and small cobbles. Fifty feet of 3/16" wall, 10" diameter casing with a steel plate welded over the bottom and 3/32" perforations on the lower 30 feet was set on the gravel base. The annular space between the casing and hole was backfilled with pea gravel, forming a 7 inch thick gravel pack between the formation and the casing.

The well was developed by surging and pumping with a 21 foot bailer, 8-1/2 inches in diameter for approximately 8 hours. During development approximately 1/2 foot of surge was noted in Test Hole 13 after each bailer cycle.

A water level recorder was placed on Test Hole 13, one week before the well was drilled, so that tidal fluctuations could be recorded. A maximum change of about one foot in amplitude occurred in this hole due to the effect of the tides. The time lag between tidal highs and lows and maximum and minimum water levels in the well was about 2 hours. Test Holes 11 and 12 also showed the effect of tidal change but little or no change was noted in Hole 14.

On August 28 and 29, 1963 the 10 inch diameter Test Well was pumped by Orange County Pump Company and water level observations were made in Holes 11, 12, 13, and 14. After 6-1/2 hours of pumping, a maximum drawdown of about 1 foot was noted in Test Hole 13 located 17-1/2 feet away from the pumped well. Drawdowns from 0.4 foot to 0.5 foot were observed in the remaining wells at distances ranging from 55 feet to 63 feet from the pumping well.

The Test Well was pumped 2-1/2 hours on August 28 and 6-1/2 hours on August 29. The pumping rate was approximately 84 gallons per minute with a drawdown of 24 feet. An increase of the pumping rate to 90+ gallons per minute caused the well to break suction at 39-1/2 feet.

During the pumping test, approximately 40,000 gallons of water were pumped from the San Mateo formation and throughout the test no increase of salinity could be detected with a conductivity meter. The conductivity indicated fresh water with about 800 ppm total dissolved soilds after 6-1/2 hours of pumping. In addition, fluorescein dye, placed in Test Hole 13 at a distance of 17-1/2 feet from the well, could not be detected in the discharge. The high drawdown and apparent steep gradient between Test Hole 13 and the pumping well plus the lack of saline and fluorescein-stained water indicates a relatively low permeability for the San Mateo formation.

Discharge of the pumping well was measured by recording manometer water levels on a 6 inch discharge line with a 3 inch orifice plate and converting these levels to flow in gpm by using tables furnished by the contractor. Volume measurements were made with a Sparling flowmeter.

No attempt was made to pump the well to equilibrium because of the apparent great length of time which would be required to stabilize the drawdown in the observation wells.

All calculations for permeability were therefore made from the Theis non-equilibrium equation using data from the Test Hole 13 water level recorder.

Calculated permeability values range from 103 feet per day to 365 feet per day with the upper limit based on less reliable data. The permeability value estimated by the John Stang Corporation on the basis of the mechanical analyses was 23 feet per day. A permeability analysis based on assumed equilibrium conditions between the well and TH 13 came out to be 8 feet per day.

On the basis of calculated and estimated permeability values, it appears that permeability values between 25 feet per day and 180 feet per day should be used for site dewatering studies in the San Mateo formation. It is believed that the lower limit of permeability is more nearly correct.

Pumping test date are shown in Appendix E.

G. LABORATORY TESTING PROCEDURES

1. General

Laboratory tests were performed on selected soil samples to determine their significant engineering characteristics and physical properties. The testing program was formulated with emphasis on obtaining the compressibility and shear strength characteristics of the undisturbed and remolded samples as required for the design of foundation, slopes, roads, sea walls, and other related structures.

A summary of all the laboratory test results is shown in Table 3, "Summary of Laboratory Test Results."

2. Testing Program and Procedures

a. Natural Moisture Content

Water contents of the various strength and consolidation test specimens were obtained and are recorded in Table 3.

b. Gradation

The distribution of particle sizes in typical samples was determined by the combined sieve and hydrometer analysis. Special hydrometer tests utilizing sea water as a medium with no deflocculant were conducted to determine the sedimentation characteristics of the materials at the site if such materials were disposed of in the ocean waters. Gradation curves are shown in Figures C-1 to C-6.

c. Unit Weight Determinations

The unit weight, or density, of representative undisturbed samples was determined by determining the volume and weight of the specimens. Moisture content determinations of the specimens were made to permit calculation of the dry densities.

Moisture-density relationships were obtained in accordance with ASTM Designation D1557-58T. For the San Mateo sands, the minimum and maximum dry densities were also obtained based on the relative density concept.

d. Consolidation Tests

Consolidation characteristics of the materials were investigated by means of consolidation tests performed on representative samples from various depths. These tests were performed in fixed ring consolidometers, with drainage permitted from the top and base of the specimen, and with loads applied in the conventional manner. Specimens were 2.87 inches in diameter and were tested in rings one inch high. The test results are presented in the form of Normal Pressure vs. Void Ratio and Percent Compression in Figures C-8 and C-9.

e. Direct Shear Tests

Direct shear tests were performed with a strain-controlled machine on selected samples to obtain the shear strength characteristics of the material. Shearing displacement was manually controlled at a rate approximating 0.01 inches per minute. The specimens were tested in rings, 2.87 inches in diameter and one inch high. A porous stone at the base provided drainage for the sample.

Samples were sheared at either saturated or field moisture conditions under several values of normal stress. As all samples were initially highly consolidated, no attempt was made to insure full consolidation under the normal stresses before shearing. Although drainage may not have been completed, most of the tests could be considered as approaching consolidated-drained tests.

Results of these tests are presented as Shear Stress vs. Horizontal Displacement Curves and Shear Strength vs. Normal Pressure, all shown in Figures C-10 to C-20. Composite strength envelopes are presented in Figures 6 and 7.

f. Permeability Tests

Permeability tests of the falling head type were conducted on samples compacted to the appropriate values of relative compaction as determined from the results of the compaction tests.

g. Engineering Characteristics of Soil Types Determined From Laboratory Tests

Discussed below are the pertinent engineering characteristics of the two major soil formations as determined by results of laboratory tests, summarized in Table 3.

The terrace materials found in the upper fifteen feet consist of silty to clayey fine and medium sands. Standard Penetration Tests summarized in Figure 5, indicate these sediments to be dense to very dense. Dry densities summarized in Table 3 show significant variations, ranging from 94 to 123 pcf. Below this stratum to the approximate elevation of 45 feet, the terrace materials are considerably coarser, ranging in gradation from sands to cobbles. Although penetration tests were not possible in this stratum due to the interference of gravel and cobbles, field observations indicate these coarse sediments to be quite densely packed.

The San Mateo sands consist of buff-colored, well graded sands containing scattered pebbles. The consistently high blow counts encountered during the standard penetration tests (Figure 5) indicate this formation to be extremely dense. This is confirmed by the high dry densities, ranging from 110 to 121 pcf, as determined in the laboratory. Although the formation is generally quite homogeneous both laterally and vertically, scattered pockets or lenses of stiff, silty fine and gravelly sands are encountered. One such isolated lense, encountered only in Boring No. 1 at elevation -58 feet, consists of 20 feet of highly consolidated silty sand.

Results of qualitative and spectrographic analyses conducted by Truesdail Laboratories, Inc. reveal that a clay binder is present in the San Mateo sands and is very similar in composition to the clay found in the terrace deposits. The clay was found to contain sufficient iron as well as silicon and aluminum to render it as a cementing agent for both the terrace sediments and the San Mateo sands. For more details, see Appendix C.

Results of direct shear tests reveal high shear strengths for all soil samples. The equations of the composite shear strength envelopes, plotted in Figures 6 and 7 are found to be as follows:

Terrace Deposits:

$$S = 500 + N \tan 35^\circ$$

San Mateo Sands:

$$S = 400 + N \tan 45^\circ$$

Although it was not possible to conduct shear tests on the more cobbly sediments, their ability to withstand slopes as steep as the other sediments indicates that their shear strengths are comparable to those stated above.

Analysis of consolidation tests indicates that all samples tested were highly precompressed and, consequently, experienced very small percentages of compression under stresses up to those which will be exerted by the anticipated structural loads.

Approved:

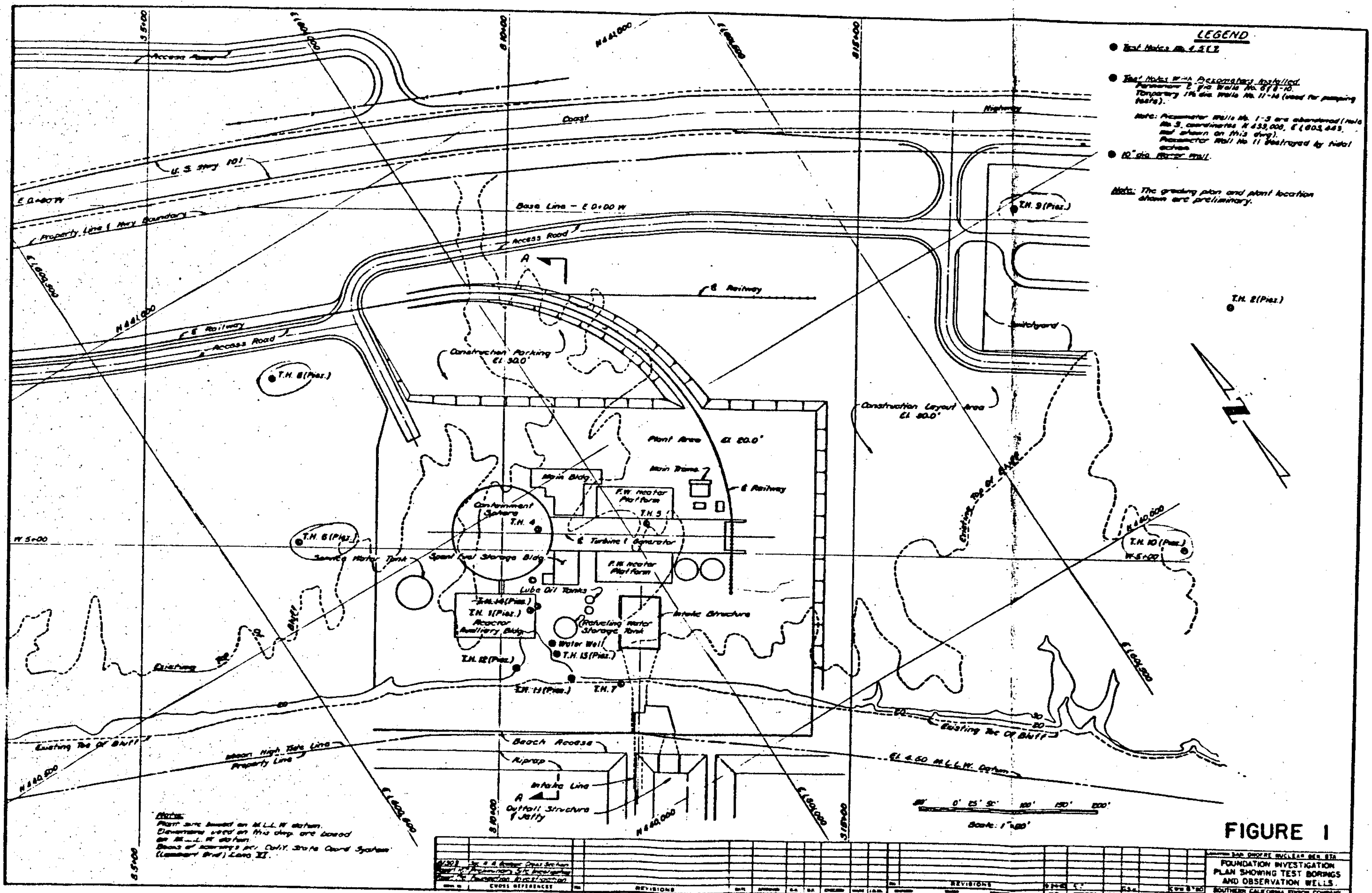
P. J. West
Senior Engineering Geologist

Robert Chieruzzi
Civil Engineer

G. S. Hunt
Asst. Engineering Geologist

TABLE 3 - SUMMARY OF LABORATORY TESTS

TYPE	HOLE NO.	DEPTH (ft.)	ELEVATION (MLLW) (ft.)	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SPECIFIC GRAVITY	GRAIN SIZE PERCENTAGES Gravel Sand Silt Clay	FRICTION ANGLE (deg.)	COHESION (psf)	PERMEABILITY (ft./yr.)	CURVES INCLUDED IN APPENDIX C Grain Size	Shear	Consol.	REMARKS
Terrace	9	11	88	111	6								x	
Terrace	9	13	86	115-123	15-19	2.70	3-43-26-30	30	1020		x	x		Sheared under sat. condition
Terrace	5	5	83							0.1	x			Sea water used in hyd. test; permeability on undist. sample
Terrace	10	15	74	97-101	26-30			39.5	824		x	x		Sheared under sat. condition; sea water used in hyd. test
Terrace	10	30	59	94-98	8-13			32.5	400			x		
S.M. Sand	6	66	24	110-116	7.3		34-51-5-0	33.5	1170		x	x		Sheared at field moist
S.M. Sand	7	6	6	111-115	12.6	2.65	35-60-5-0	45	250		x	x		Sheared at field moist
S.M. Sand	13	20	-5	117-119	19-21	2.64		44.5	380			x		Sheared under sat. condition
M. Sand	6	115	-26	114-119	17-20		24-70-6-0	44	300		x	x		Sheared under sat. condition
S.M. Sand	6	115	-26	114.5	13.6								x	
S.M. Sand	5	123	-36	115-121	19-21			45	220			x		Sheared under sat. condition
Silty Sand	1	81	-59	115	16	2.75	0-60-20-20	35	1700		x	x		Sheared at field moist. - thin, stiff silt layer
Lense	1	110	-88										x	
S.M. Sand	1	120	-98	113-116	18-20			47.5	780			x		Sheared under sat. condition
S.M. Sand	1	123	-101	100	15								x	
Terrace Composite	-			115-118	14			48.5	500			x		Sheared under sat. condition; initial moisture given
Terrace Composite	-			123	10					0.04			x	Max. dry unit weight and optimum moisture content obtained by modified AASHO method.
S.M. Sand Composite	-			118	—									Max. dry unit weight obtained by modified AASHO method; moisture content not critical
[illegible] Sand Composite	-			93-119	—					380				Min. and max. dry unit weights obtained by Relative Density method. Moisture content not critical; permeability based on 119 pcf sample


LEGEND
Test Holes No. 4, 5 & 7
Test Holes With Piezometers Installed
Note: The grading plan and plant location shown are preliminary.
Coast Highway
U. S. Hwy. 101
Base Line - E 0+00 W
Access Road
Railway
Property Line & Hwy Boundary
Switchyard
Construction Parking El. 30.0'
Construction Layout Area El. 50.0'
Plant Area El. 20.0'
Main Bldg.
Main Trans.
F. W. Heater Platform
Containment Sphere
T. H. 4
T. H. 5
T. H. 6 (Piez.)
T. H. 8 (Piez.)
T. H. 9 (Piez.)
T. H. 2 (Piez.)
T. H. 10 (Piez.)
T. H. 1 (Piez.)
T. H. 12 (Piez.)
T. H. 13 (Piez.)
T. H. 7
Turbine & Generator
Service Water Tank
Spent Fuel Storage Bldg.
Lube Oil Tanks
Reactor Auxiliary Bldg.
Refueling Water Storage Tank
Intake Structure
Water Well
Existing Toe Of Bluff
Beach Access
Riprap
Intake Line
Outfall Structure & Jetty
Mean High Tide Line
Property Line
El. 4.60 M.L.L.W. Datum
Scale: 1" = 80'
FIGURE 1
SAN ONOFRE NUCLEAR GEN STA
FOUNDATION INVESTIGATION
PLAN SHOWING TEST BORINGS
AND OBSERVATION WELLS.
SOUTHERN CALIFORNIA EDISON COMPANY
SCALE AS SHOWN

569284-0

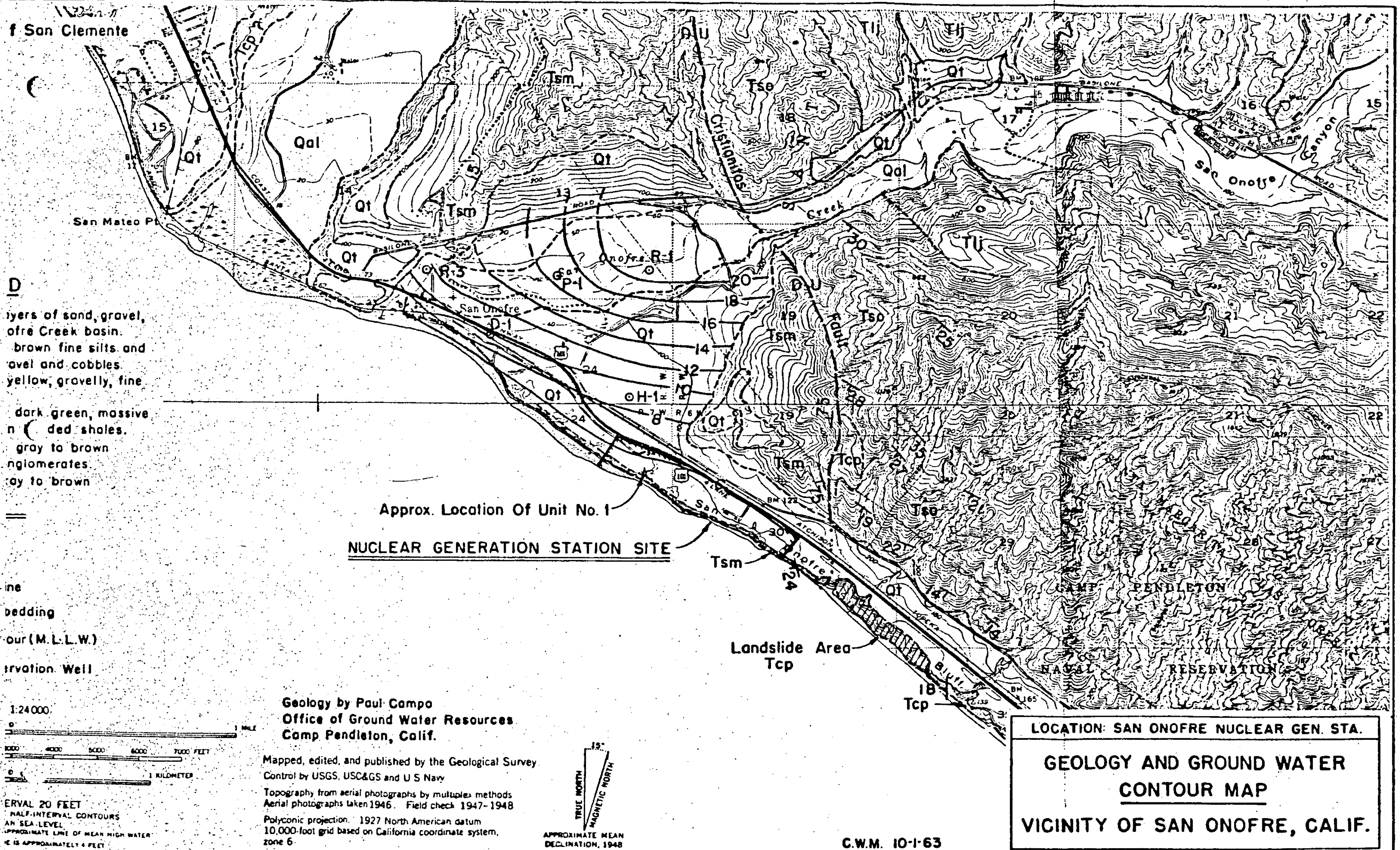
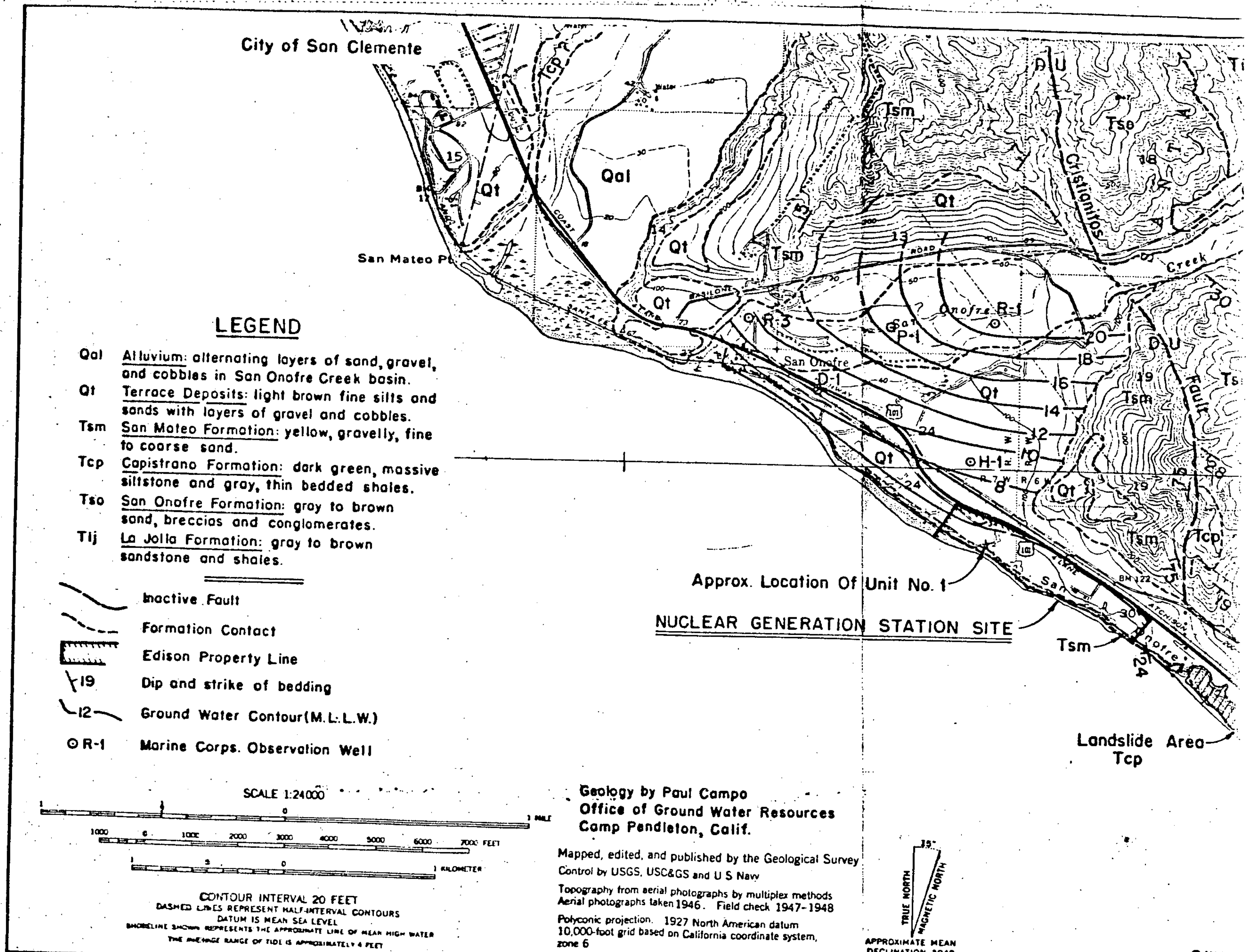
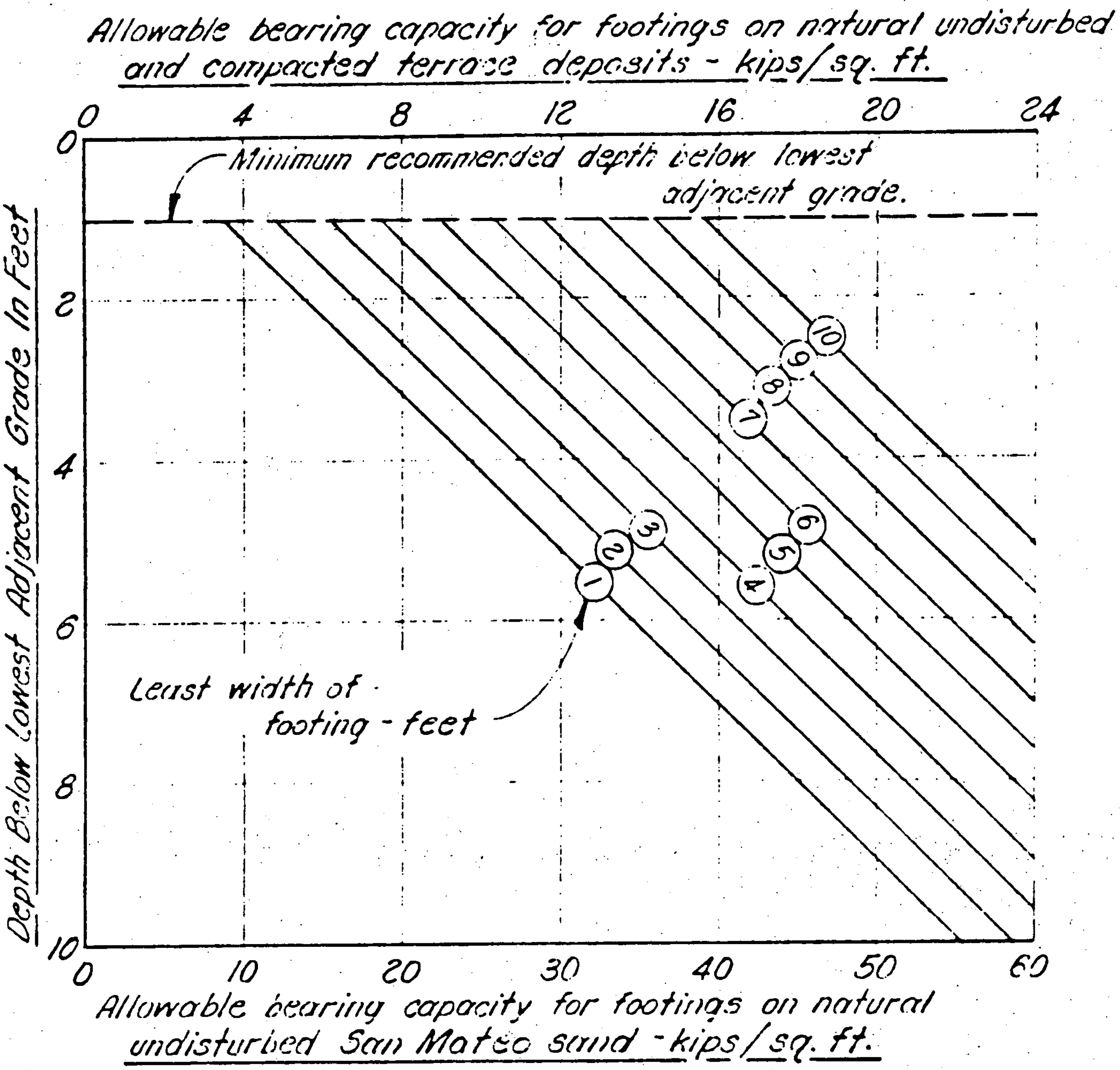

f San Clemente
San Mateo Pt
Qal
Qt
Tsm
Tso
Tli
Tcp
Cristianitos
Fault
San Onofre Creek
San Onofre Canyon
Basilone Road
San Onofre
R-1
R-3
P-1
D-1
H-1
D
yers of sand, gravel,
ofre Creek basin.
brown fine silts and
avel and cobbles.
yellow, gravelly, fine
dark green, massive
ded shales.
gray to brown
nglomerates.
ay to brown
ine
bedding
our (M.L.L.W.)
rvation Well
Approx. Location Of Unit No. 1
NUCLEAR GENERATION STATION SITE
Landslide Area
Camp Pendleton
Naval Reservation
1:24000
ERVAL 20 FEET
Geology by Paul Campo
Office of Ground Water Resources
Camp Pendleton, Calif.
Mapped, edited, and published by the Geological Survey
Control by USGS, USC&GS and U S Navy
Topography from aerial photographs by multiplex methods
Aerial photographs taken 1946. Field check 1947-1948
Polyconic projection. 1927 North American datum
10,000-foot grid based on California coordinate system, zone 6
TRUE NORTH
MAGNETIC NORTH
APPROXIMATE MEAN DECLINATION, 1948
C.W.M. 10-1-63
LOCATION: SAN ONOFRE NUCLEAR GEN. STA.
GEOLOGY AND GROUND WATER
CONTOUR MAP
VICINITY OF SAN ONOFRE, CALIF.

FIGURE 2


City of San Clemente
San Mateo Pt.
Qal
Qt
Tsm
Tso
Tcp
Cristianitos
Creek
San Onofre
Onofre R-1
R-3
P-1
D-1
H-1
Fault
BM 122
Approx. Location Of Unit No. 1
NUCLEAR GENERATION STATION SITE
Tsm
Landslide Area
Tcp
LEGEND
Qal Alluvium: alternating layers of sand, gravel, and cobbles in San Onofre Creek basin.
Qt Terrace Deposits: light brown fine silts and sands with layers of gravel and cobbles.
Tsm San Mateo Formation: yellow, gravelly, fine to coarse sand.
Tcp Capistrano Formation: dark green, massive siltstone and gray, thin bedded shales.
Tso San Onofre Formation: gray to brown sand, breccias and conglomerates.
Tlj La Jolla Formation: gray to brown sandstone and shales.
Inactive Fault
Formation Contact
Edison Property Line
19 Dip and strike of bedding
12 Ground Water Contour (M.L.L.W.)
R-1 Marine Corps. Observation Well
SCALE 1:24000
1 MILE
7000 FEET
1 KILOMETER
CONTOUR INTERVAL 20 FEET
DASHED LINES REPRESENT HALF-INTERVAL CONTOURS
DATUM IS MEAN SEA LEVEL
SHORELINE SHOWN REPRESENTS THE APPROXIMATE LINE OF MEAN HIGH WATER
THE AVERAGE RANGE OF TIDE IS APPROXIMATELY 4 FEET
Geology by Paul Campo
Office of Ground Water Resources
Camp Pendleton, Calif.
Mapped, edited, and published by the Geological Survey
Control by USGS, USC&GS and U S Navy
Topography from aerial photographs by multiplex methods
Aerial photographs taken 1946. Field check 1947-1948
Polyconic projection. 1927 North American datum
10,000-foot grid based on California coordinate system, zone 6
TRUE NORTH
MAGNETIC NORTH
APPROXIMATE MEAN DECLINATION, 1948

C.W.M. -



Notes:

1. These values apply for footings above ground water table. For submerged footings apply a factor of one-half.
2. All fill should be compacted to 95% of maximum dry density, A.S.T.M. D1557-58T.
3. Values shown apply for total dead plus frequently applied live loads.
4. Values shown are net bearing values.

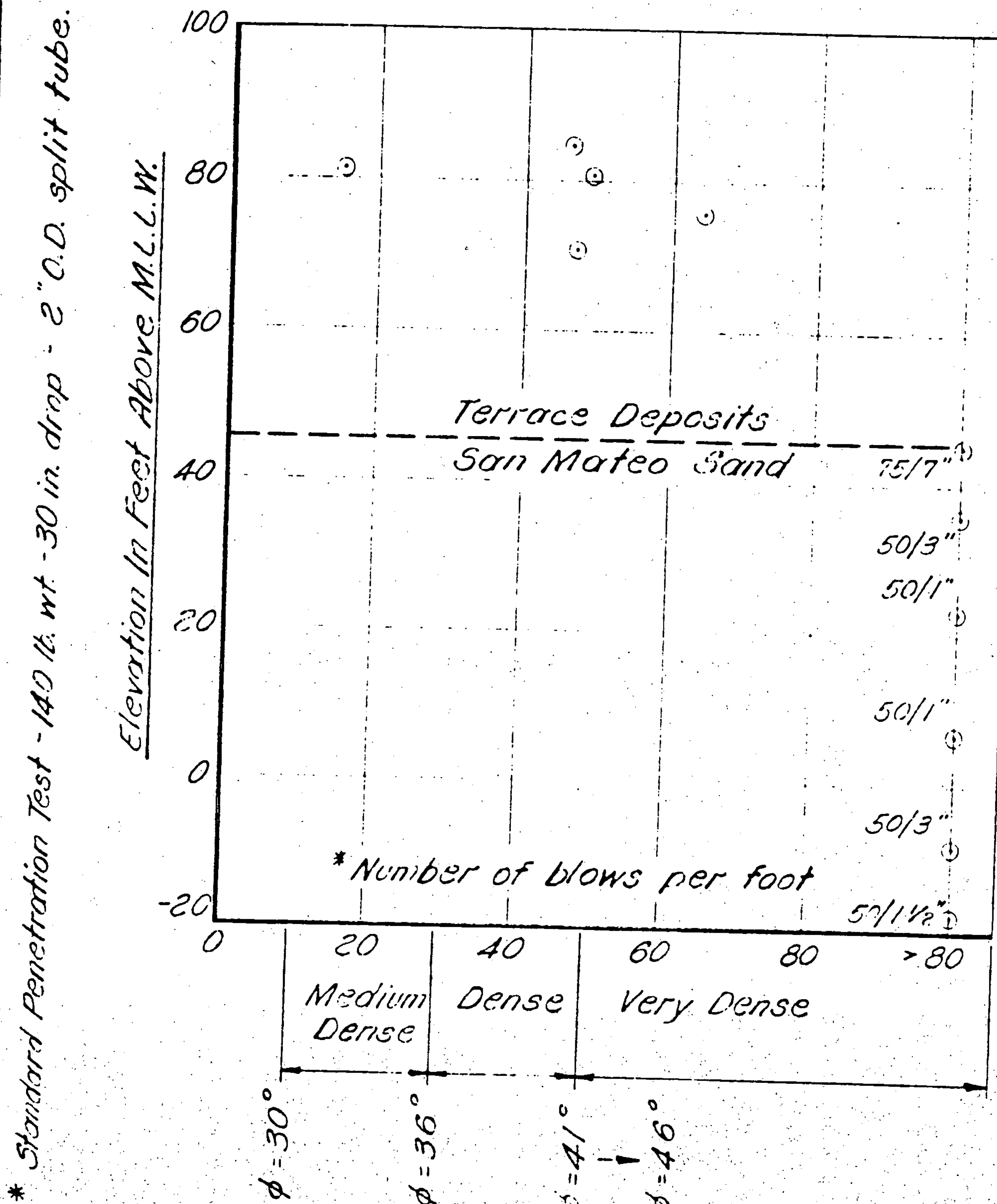
NO.	REVISIONS	5780	10 7 63	LOCATION San Onofre Nuclear Gen. Sta.
		JOB ORDER	DATE APPROVED	ALLOWABLE BEARING CAPACITY VALUES

APPROVED	O. K.	O. K.	CHECK	MADE
				CWM

SOUTHERN CALIFORNIA EDISON COMPANY
LOS ANGELES, CALIF.

FORM OD 334 2M 8-58 1000H-10

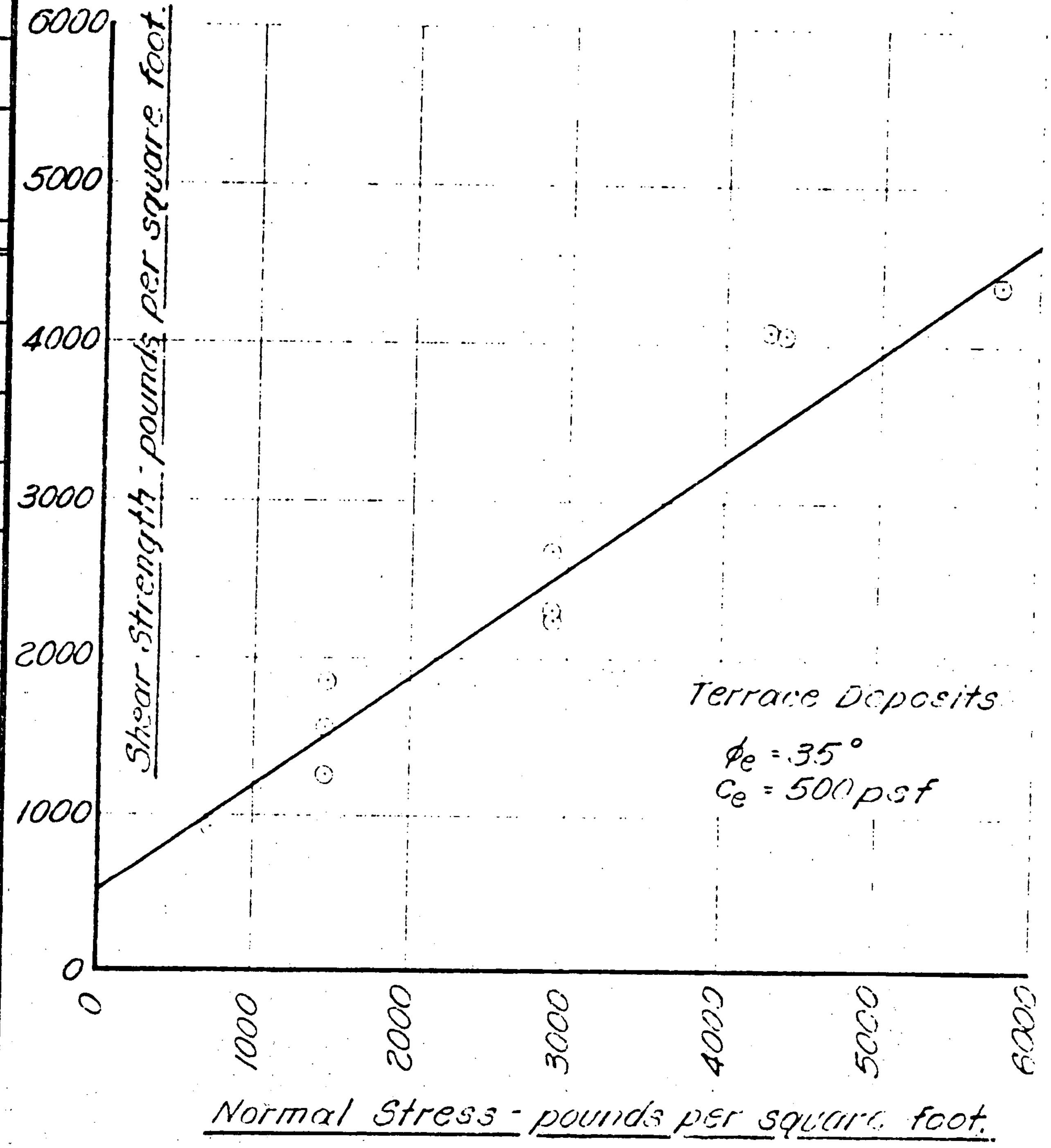
FIGURE 4



				LOCATION San Onofre Nuclear Gen. Sta.
				SUMMARY OF STANDARD PENETRATION TESTS
No.	REVISIONS	5780	10-7-63	
APPROVED / O. K. / O. K. / CHECK	CWM MADE	JOB ORDER	DATE APPROVED	SOUTHERN CALIFORNIA EDISON COMPANY LOS ANGELES, CALIF.

FORM OD 334 2M 5-56 1000H-10

FIGURE 5



LOCATION San Onofre Nuclear Gen. Sta.

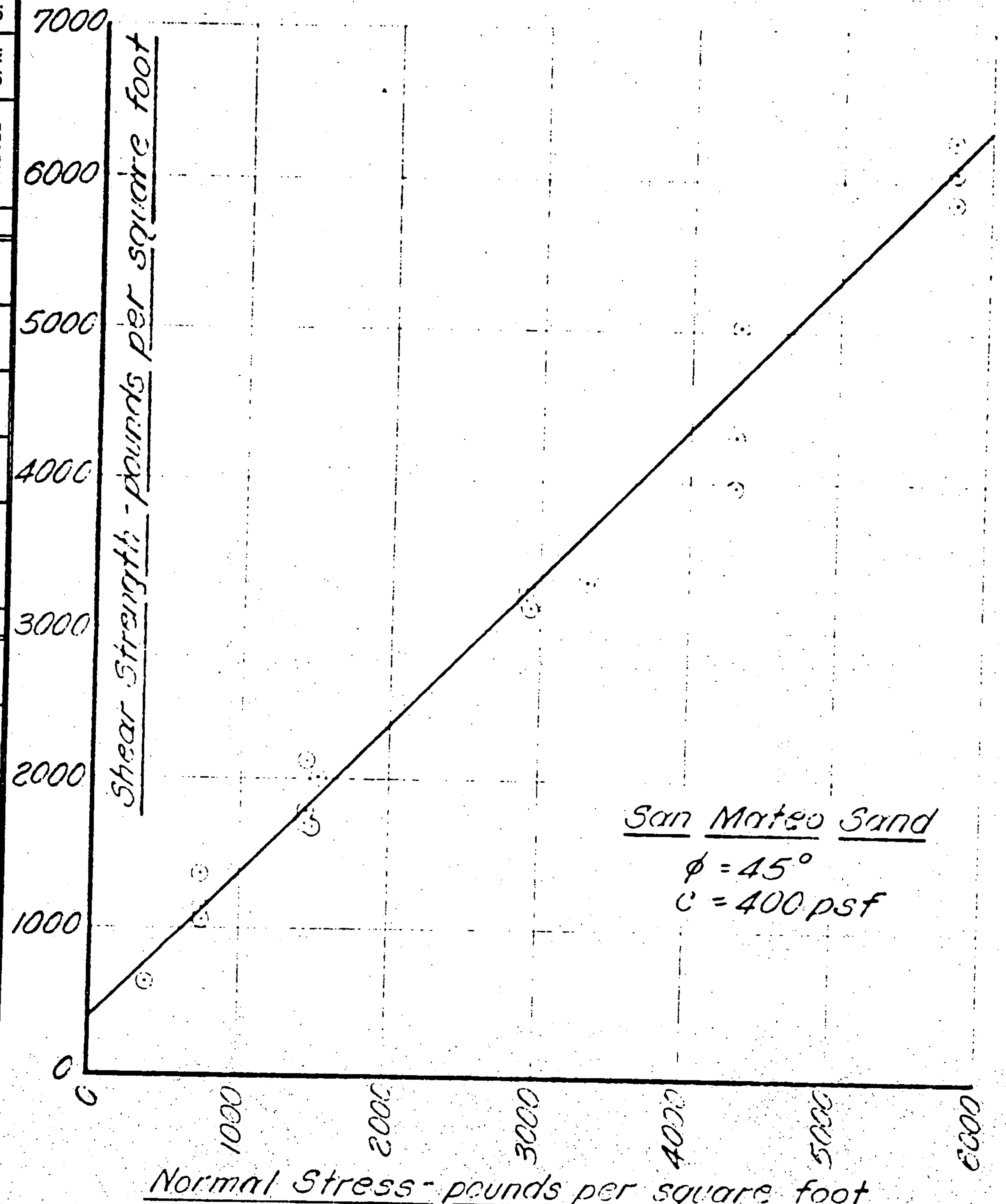
COMPOSITE STRENGTH ENVELOPE FOR TERRACE DEPOSITS

No.	REVISIONS					5780	10-7-63
APPROVED	O. K.	O. K.	CHECK	MADE CWM		JOB ORDER	DATE APPROVED

SOUTHERN CALIFORNIA EDISON COMPANY
LOS ANGELES, CALIF.

FORM OD 334 2M 5-56 1000H-10

FIGURE 6



NO.	REVISIONS	5780	10-7-63
APPROVED / O. K. / O. K. / CHECK / MADE CWM		JOB ORDER	DATE APPROVED

LOCATION San Onofre Nuclear Gen. Sta.

COMPOSITE STRENGTH ENVELOPE FOR SAN MATEO SANDS

SOUTHERN CALIFORNIA EDISON COMPANY
LOS ANGELES, CALIF.

1 APPROVED O. K. O. K. CHECK MADE 2 APPROVED O. K. O. K. CHECK MADE 3 APPROVED O. K. O. K. CHECK MADE 4 APPROVED O. K. O. K. CHECK MADE

F... OD 334 2M 8-58 1000H-10

FIGURE 7