UPDATE GEOTECHNICAL INVESTIGATION
1615 OCEAN FRONT STREET
SAN DIEGO, CALIFORNIA

Prepared for
Mr. John J. Lormon
San Diego, California

TerraCosta
Consulting Group

Prepared by
TERRACOSTA CONSULTING GROUP, INC.
San Diego, California

Project No. 2660
November 8, 2016

TerraCosta Consulting Group
Geotechnical Engineering
Coastal Engineering
Maritime Engineering

Project No. 2660
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Mr. John J. Lormon
c/o Procopio, Cory, Hargreaves & Savitch LLP
525 B Street, Suite 2200
San Diego, California 92101

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1615 OCEAN FRONT STREET
SAN DIEGO, CALIFORNIA

Dear Mr. Lormon:

In accordance with your request and our Proposal No. 16072 dated July 11, 2016, TerraCosta Consulting Group, Inc. (TerraCosta) has performed an update geotechnical investigation for the planned upgrade/remodel of your single-story residence at 1615 Ocean Front Street in the Sunset Cliffs/Ocean Beach area of San Diego, California.

The accompanying report presents the results of our project document review, field investigative work, engineering analyses of subsurface conditions at the site, and presents our conclusions and recommendations pertaining to the geotechnical aspects of site development.

We appreciate the opportunity to work with you on this project, and trust this information meets your present needs. If you have any questions or require further information, please give us a call.

Very truly yours,
TERRACOSTA CONSULTING GROUP, INC.

Walter F. Crampton, Principal Engineer
R.C.E. 23792, R.G.E. 245

Braven R. Smillie, Principal Geologist
C.E.G. 207, P.G. 402

REGISTERED PROFESSIONAL ENGINEER
WALTER F. CRAMPTON
No. 245
GEOTECHNICAL
STATE OF CALIFORNIA

PROFESSIONAL GEOLOGIST
BRAVEN R. SMILLIE
No. 207
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

WFC/BRS/sr
Attachments

cc: Mr. Scott Bernet
Scott Bernet Architects

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1 INTRODUCTION AND PROJECT DESCRIPTION

The subject property is situated on the coastal terrace at the top of the coastal bluffs at 1615 Ocean Front Street between Coronado and Del Mar Avenues in the Sunset Cliffs/Ocean Beach area of the City of San Diego, California (see Vicinity and Geologic Map, Figure 1).

More specifically, the site is located at 32.742 north latitude and 117.255 west longitude atop the westerly-facing coastal bluff, which descends approximately 53 feet from the top of the bluff down to the Pacific shoreline. Because the property is located within the City of San Diego's "Coastal Overlay Zone" (COZ), and adjacent to "sensitive coastal bluffs," the City code requires a site development permit be obtained prior to the start of any work or improvements.

As we understand, the proposed project will include raising up and supporting the existing single-story residential structure on temporary timber cribbing, excavation for, and construction of a basement-level structure utilizing driller pier shoring, and lowering the original single-story residence back to lot grade and integrating it with the new basement-level structure.

1.1 Project Site and Site-Area History

Following a period of increased coastal erosion and accelerated coastal bluff retreat (generally from the 1940s through the 1970s), the City of San Diego developed and implemented the Sunset Cliffs Shoreline and Upper Cliffs Stabilization Project which achieved greater public access and public safety along the coastline, and also improved the stability of the bluffs at the site to a minimum factor-of-safety of 1.5. A review of the General Plan in the Del Mar Avenue area of the City project (Woodward Clyde, 1981) indicates that the segment of coastline beginning just northerly of the intersection of Bacon and Coronado Avenues, continuing south to Orchard Avenue, was revetted. Additionally, a mid-bluff wall was constructed beginning approximately at 1621 Ocean Front Street and extending south to approximately 1569 Ocean Front Street. This mid-slope wall and riprap supports and protects a new fill slope (upwards of 50+ feet deep, constructed in 1982) that provides lateral support to the properties along this segment of the coastline, including 1615 Ocean Front Street.

During the late 1970s and early 1980s, both of the undersigned performed a significant part of the geotechnical and civil engineering design services for the Sunset Cliffs shoreline and upper cliff stabilization project (Walt Crampton as Geotechnical Engineer and Project Manager for Woodward Clyde and Bob Smillie as Project Engineering Geologist for Woodward Clyde on the project). In 1982, the City of San Diego, in part funded by the State of California Department of Boating and Waterways, implemented the Sunset Cliffs Shoreline and Upper Cliffs Stabilization Project, and, as a result, marine erosion was arrested, thus allowing the City to construct mid-bluff lateral public access along this reach of the coastline.

In early 2010, Walt Crampton and Bob Smillie, Principal Engineer and Principal Geologist (having formed TerraCosta Consulting Group, Inc. in 2001), performed a preliminary geotechnical investigation in support of a remodel project on the subject property. The result of that study were also reviewed as part of the current work.

2 PURPOSE AND SCOPE OF WORK

The purpose of our study is to provide geotechnical information to assist you and your consultants in project design, and to address City of San Diego and California Coastal Commission concerns regarding the proposed project.

For input in performing our studies and preparing this report, we have reviewed geologic literature, maps, historic aerial stereographic and oblique photographs, and other relevant reports and documents in our files. References are provided at the end of this report.

In particular, our investigation is designed to address the following geotechnical issues:

- The geologic setting of the site;
- Potential geologic hazards;
- Gross stability of the coastal bluff;

- Geotechnical characteristics of the on-site soils;
- Groundwater;
- Proposed site grading;
- Foundation design, including allowable soil bearing and earth pressure values;
- Construction-period stability of the basement excavation; and
- Concrete flatwork recommendations.

3 FIELD INVESTIGATION

A limited geologic reconnaissance was performed on the subject site and immediately adjacent areas. Our subsurface investigation included the drilling of a single 6-inch-diameter hollow-stem auger boring to a depth of 29.5 feet on July 25, 2016, using a limited-access track-mounted drill rig. The location of the auger boring is indicated on the Site Plan (Figure 2). A key to the boring log is presented in Appendix A as Figure A-1. The final log of our test boring is presented on Figure A-2. Geologic Cross-Section A (Figure 3) is based on our prior geotechnical experience in the project site area and on the data obtained from Test Boring B-1, drilled July 25, 2016. Figure 4, Site Area Geology, indicates present day soil and geologic units exposed at the surface in the project site area on an aerial photo base.

4 SITE AREA GEOLOGY

4.1 Geologic Setting

The coastal plain and coastal bluffs throughout the majority of San Diego County are characterized by thick sequences of interbedded Eocene marine siltstones, claystones, sandstones, and conglomerates; however, the coastal bluffs from Point La Jolla (on the south side of the Rose Canyon Fault Zone) to the southern tip of Point Loma are all formed by the Cretaceous Point Loma Formation. Coastal bluff retreat, a geomorphic process that has operated for millions of years and continues today along most of San Diego's coastline, in part combined with tectonic forces, has formed steep coastal bluffs ranging up to as high as 300 feet in elevation in parts of San Diego County. Locally, the project site is situated at the westerly bluff-terminated edge of a ±1/2-mile-wide gently westerly sloping coastal terrace, one of a sequence of well-defined, wave-cut abrasion terraces created primarily by higher eustatic sea stands during Pleistocene age interglacial episodes and, to a lesser degree, by tectonic uplift.

Point Loma is a 6-mile-long promontory, extending southward from the low land adjacent to the mouth of the San Diego River. The Point Loma coastal bluffs are bordered by a narrow wave-cut terrace or bench, with elevations ranging from 25 to 95 feet MSL. Wave impact erosion has etched out the less resistant rock along faults and fractures in the coastal bluff resulting in the shallow coves and sea caves, which punctuate the Point Loma coastline. The more resistant rocks of the Point Loma Formation form the lower cliffed section of the coastal bluff and shore platform, which extends seaward. The relatively flat surface of the modern-day abrasion platform is interrupted by isolated erosion-resistant rock, which forms sea stacks and topographic highs. Further seaward, the abrasion platform becomes progressively deeper, and is locally incised by surge channels that have formed along the trends of major joint sets or faults, which have locally decreased the erosion resistance of the lower sea cliff.

4.2 Site Conditions

The 50-foot-wide property is bounded on the east by Ocean Front Street, on the north and south by adjoining residential lots, and on the west by the Pacific Ocean. Topographic relief across the site is relatively flat at an average elevation of 53 feet. A review of 1928 aerial photographs indicates that the site was likely developed in the early part of the last century with few substantial changes (to the original structure footprint) over the years.

4.3 Subsurface Conditions

Two geologic formations are present in the general area. Exposed in the lower bluff, the Point Loma Formation is a member of the 70 to 80 million year old Cretaceous-age Rosario Group, which is exposed along the coastline from southern San Diego County to northern Baja, California. The late to middle Pleistocene coastal bluff terrace deposits, which overlie the Point Loma Formation at the site, are in-turn locally overlain by overburden soils including alluvium and colluvium and man-placed fill soils. The following paragraphs describe these units from oldest to youngest.

<u>Point Loma Formation (Kp)</u>: The Cretaceous-age Point Loma Formation is an approximately 900-foot-thick sedimentary rock layer that discontinuously crops out in coastal areas of northern Baja, California to as far north as Carlsbad (Kennedy, 1975). Where not affected by fractures and jointing in the rock, this cliff-forming unit is relatively resistant to erosion. A short distance north of the site, where exposed in the sea cliff, the Point Loma Formation extends up to an elevation of approximately 24 feet, MSL. The Point Loma Formation extends seaward comprising the shore platform adjacent to the cliff. The Point Loma Formation consists of well-indurated marine sediments deposited by an offshore, deep-water submarine fan. Offshore deposits are represented by the thin-bedded siltstone and fine sandstone exposed in the sea cliff. Deep water deposits are represented by the erosion-resistant, thick-bedded mudstone and sandstone exposed at the base of the cliffs.

<u>Old Paralic Terrace Deposits (Qop_6)</u>: Late to middle Pleistocene terrace deposits overlie the gently westerly-inclined platform on the Point Loma Formation, which was formed by wave-abrasion during the last interglacial period when worldwide sea level was approximately 20 feet higher. This Pleistocene unit consists of both marine and non-marine, poorly consolidated, fine- to medium-grained, light brown fossiliferous sandstone. The slope of the Bay Point Formation provides an indication of the relative rate of marine erosion of the underlying cliff-forming Point Loma Formation, with relatively steep slopes in the upper terrace deposits, again suggesting relatively high marine erosion rates prior to the City of San Diego's Shoreline Stabilization Project completed in 1983.

<u>Artificial Fill (Qaf)</u>: Extensive shoreline stabilization measures have been undertaken in the study area, led by the City of San Diego as part of their 1983 Sunset Cliffs Shoreline Stabilization Project, including relatively extensive rock revetments placed at the base of the sea cliff, along with the construction of reinforced earth walls and a reconstructed upper bluff to stabilize the section of coastline between Coronado Avenue and Orchard Avenue.

5 GROUNDWATER

No groundwater seepages were encountered in our test boring, and the soil samples collected throughout the boring were dry to damp with no free moisture.

6 GEOLOGIC HAZARDS

6.1 Faulting and Seismicity

The site is located in a moderately active seismic region of southern California that is subject to significant hazards from moderate to large earthquakes. Ground shaking from several major active fault zones could affect the site in the event of an earthquake. The nearest of these, the Rose Canyon Fault Zone, trends north-northwest and has been mapped approximately 4 miles east-northeast of the site. No known active faults have been mapped, nor were any observed during our geotechnical investigation at, or near, the site.

6.2 Seismic Design Parameters

For seismic design based on the 2013 California Building Code, we recommend the following design parameters, which were determined using the USGS Seismic Hazard Calculator. These parameters may be used to construct both the maximum considered and design response spectra. The two spectra are generally quantified in terms of the short period spectral acceleration and the spectral acceleration at a period of vibration of a single degree freedom system of 1 second. For this project, we located the project site at 32.742 north latitude and 117.255 west longitude. In addition, the site is classified as Type D (“Stiff Soil”).

Using the USGS calculator and a site classification of Type D, the S_{MS} (short period spectral acceleration) and S_{M1} (the spectral acceleration at 1 second) are 1.175 g and 0.666 g, respectively. Additionally, the design spectral accelerations of S_{DS} and S_{D1} are 0.783 g and 0.444 g, respectively.

7 GEOTECHNICAL CONCLUSIONS AND RECOMMENDATIONS

7.1 General

Our investigation did not reveal the presence of any adverse geologic conditions on the site, such as faulting, adverse bedding, or a high groundwater table, which would adversely affect the existing development.

7.2 Gross Stability of Coastal Bluff

In 1982, the City of San Diego implemented the Sunset Cliffs Shoreline and Upper Cliffs Stabilization Project. As part of that project, a new rock revetment and mid-bluff seawall was constructed to prevent loss of property along this reach of the coastline. As indicated in the referenced reports, extensive engineering design efforts went into the stabilization of the coastal bluffs. These improvements resulted in the bluffs having a dramatically reduced rate of erosion and a factor of safety against failure of greater than 1.5.

7.3 Predicted Bluff Retreat Over Next 75 Years

Prior to the implementation of the Sunset Cliffs Shoreline and Upper Cliffs Stabilization Project, this reach of Sunset Cliffs was locally experiencing rates of bluff retreat of greater than 1 foot per year (around Del Mar Avenue). Following the stabilization project and establishment of vegetation, we estimate the average bluff-top erosion rate is less than 1 inch per year.

Based on the results of our study, the contemporary top-of-bluff is located 25 to 32 feet seaward of the subject residence. The average setback is approximately 28 feet. It is our belief that, due to the City's stabilization project, both marine and subaerial erosion rates are currently significantly lower along this reach of the coastline. Following the stabilization project and establishment of vegetation, we estimate the average bluff-top erosion rate to be less than 1 inch per year. As importantly, the reconstructed bluff appears to have been conservatively designed with an intended minimum design life of 100 years. This area of Sunset Cliffs is now one of the more stable sections of coastline, providing foundation support for the subject residence.

It is our opinion that the work performed on the property does not affect the gross stability of the bluff. Since vegetation has been established, the existing bluff top, as it exists today, will in our professional opinion provide a minimum of 75 years of continued stability for the subject property.

7.4 Earthwork and Grading

All grading and site preparation should be performed under observation of the geotechnical engineer and in accordance with the attached Specifications for Controlled Fill, Appendix B.

All vegetation, debris, and other deleterious material should be removed from the site prior to site regrading. All structural fill and backfill soils should be compacted to a minimum 90 percent of the maximum dry density as determined by ASTM Test Method D 1557. Moisture content in the fill should be maintained between the optimum moisture content and 3 percent over optimum. The geotechnical engineer should review the foundation and grading plans to evaluate whether the intent of the recommendations presented herein has been properly interpreted and incorporated into the contract documents. It is further recommended that the geotechnical engineer observe the site regrading (including areas of overexcavation), foundation excavations, construction of retaining walls, and subgrade preparation under concrete slabs and paved areas.

7.5 **Construction-Period Shoring**

The proposed site development consists of the excavation for, and construction of, a basement under the existing residential structure, with the general limits of the basement shown on Figure 2. As indicated on Figure 2, we are proposing the use of a cantilevered drilled pier perimeter wall system constructed with alternating 2-foot-diameter and 12-inch-diameter drilled piers, with the 2-foot-diameter drill spacing ranging from 5.5 to 8 feet on center, and with the widest pier spacing along the westerly edge of the basement.

All shoring systems deform during excavation; the level and magnitude of deformation being a function of the pre-stressing used in the system and the skill and workmanship of the shoring contractor. For a cantilevered system, we anticipate construction-period lateral movements of the shoring to range from 0.2 to 0.3 inch of the ground surface, with vertical settlements adjacent to the shoring system on the order of the lateral displacement of the shoring. In addition, we anticipate that vertical settlement of the area adjacent to the excavation will occur over a distance equal to the approximate height of the excavation or, in this instance, 10 feet, with the magnitude of settlement behind the shoring decreasing with distance.

Resistance to lateral loads applied to a drilled pier shoring system is developed through deflection in the pier, which mobilizes the reaction of the soil into which the drilled pier is embedded. The resisting pressure applied by the soil to the pier depends upon the relative stiffness of the pier and soil, as well as depth of embedment.

Failure of a laterally-loaded pier takes place either when the maximum bending moment in the loaded pier reaches the ultimate or yield resistance of the pier section, or when the lateral earth pressures reach the ultimate lateral resistance of the soil along the total length of the pier. For purposes of definition, failure of piers with relatively "short embedment" takes place when the pier rotates as a unit with respect to a point located close to its toe. Failures of piers with relatively "long embedment" occur when the maximum bending moment applied to the pier exceeds the yield resistance of the pier section, and a plastic hinge forms at the section of maximum bending moment. Investigators have suggested that piers be grouped relative to their dimensionless depth of embedment L/T where:

L = embedment length of the pier in feet, and

$$T = \left(\frac{EI}{f}\right)^{\frac{1}{5}} \ (divided\ by\ 12\ to\ convert\ inches\ to\ feet$$

Short piers are generally defined as L/T being less than 2.0, and long piers are generally defined as L/T being larger than 4.0.

The quantity EI is the stiffness of the pier section, and f (coefficient of variation of soil modulus) would be on the order of 40 pounds per cubic inch for the Pleistocene marine terrace deposits.

In order to determine the structural requirements for the proposed drilled pier shoring, we have evaluated the soil-induced moment, shear, and deflection of a vertical wall using the elastic theory approach developed by Matlock and Reese (1962). A condensed version of this approach is outlined in the NAVFAC Design Manual DM-7.2, Chapter 5, Section 7. Both the NAVFAC outline and supporting calculations are provided in Appendix C.

For temporary shoring, and recognizing the cohesive nature of the formational terrace deposits, we have used an equivalent fluid pressure of 15 pcf for design, while as indicated in Section 7.10, we have used a long-term design equivalent fluid pressure of 30 pcf. We have analyzed both the minimum and maximum drilled pier spacing of 5.5 feet and 8 feet on center, respectively, for both the construction-period and long-term design condition, with computed maximum soil-induced bending moments within the pier of approximately 42 kip-feet with a corresponding top-of-wall deflection of 0.3 inch during construction. For long-term design conditions, the maximum soil-induced moment would be approximately 85 kip-feet with a calculated top-of-pile deflection of 0.61 inch. This deflection, however, assumes that the basement is not in place, with the actual basement construction likely limiting post-construction deflections to only slightly above the original construction-period deflections.

Although not a code requirement, we have also calculated the maximum seismic design loading condition described in Section 7.10.2 and have also conservatively added the 60 pcf surcharge recommended in Section 7.10.1, which results in a maximum seismically induced moment of approximately 195 kip-feet in the 24-inch-diameter drilled piers. Accordingly, we would suggest that all of the 24-inch-diameter drilled piers be reinforced with sufficient steel reinforcing to develop a transient seismically induced moment capacity of 195 kip-feet.

The 12-inch-diameter intermediate piers, which will be installed primarily to mitigate any possible construction-period ground loss between the 2-foot-diameter drilled piers, will be structurally connected to a perimeter grade beam securing the adjacent 2-foot-diameter drilled piers. With fixity at the top of the pier providing most of the lateral support, we recommend a minimum of 3 feet of embedment beyond the 10-foot excavation depth, for a total intermediate 12-inch-diameter pier depth of 13 feet, braced at the top with 3 feet of embedment below the bottom of the excavation. We recommend that steel reinforcing for the 12-inch-diameter intermediate drilled piers be sized to accommodate a nominal deisgn moment capacity of 5 kip-feet. Total required pier depths are summarized in Table 1 and shown graphically on Figure 2 for all of the drilled piers.

7.6 **Soil and Excavation Characteristics and Shoring Considerations**

After the installation of the perimeter drilled piers, the subsurface formational soils on the lot may be excavated with medium effort by conventional grading equipment. Although the 24-inch-diameter drilled piers are generally spaced at 6 feet on center, maximum 24-inch-diameter pier spacing is 8 feet between Pier Nos. 12 and 13. With the 12-inch-diameter intermediate Pier S12 located midway between these two piers, the resulting exposed clear space between the 24-inch and 12-inch-diameter piers is 2.5 feet (1.5 feet for typical 6-foot spacing).

Closely spaced drilled piers having a center-to-center spacing less than three pier diameters will tend to bridge the soil behind a row of closely spaced piers with full load transfer into the piers, with each pier designed to accommodate the soil's unilateral earth pressure times the pier spacing. As discussed in the previous section, the 2-foot-diameter drilled piers have been designed to resist the lateral earth pressures from a 10-foot-deep vertical excavation with a maximum 2-foot-diameter drilled pier spacing of 8 feet. While closely spaced drilled piers typically restrain the entire soil mass behind the closely spaced piers, depending upon the material type, the soil exposed between drilled piers may still slough into the excavation. Accordingly, and to minimize potential soil sloughing between adjacent drilled piers, without having to install continuous shoring, we have recommended the installation of 12-inch-diameter intermediate drilled piers to control nuisance sloughing between the adjacent 24-inch-diameter cantilevered drilled piers. While we believe that the intermediate 12-inch-diameter drilled piers should eliminate any nuisance sloughing, there still remains the possibility of some localized nuisance slough of some of the cleaner sands comprising the Bay Point formational terrace deposits. Importantly, the wider pier spacing is generally limited to the westerly basement wall, with the northerly and southerly basement walls adjacent to neighboring residential structures utilizing a maximum pier spacing of 6.5 feet, resulting in a maximum 1.75-foot clear space.

7.7 **Foundations**

7.7.1 *Conventional Spread and Footings*

The proposed basement walls can be supported on conventional shallow foundations. Continuous or spread footings founded in undisturbed terrace deposits may be designed for an allowable soil bearing pressure of 3,000 psf. These bearing capacities may be increased by no more than one third for loads that include wind or seismic forces.

All exterior basement footings should be continuous, founded a minimum of 6 inches below adjacent basement subgrade, and have a minimum width of 12 inches. Exterior footings should be reinforced at top and bottom with at least two No. 4 bars, four bars total. This recommendation provides minimum requirements; the actual reinforcement should be in accordance with the structural engineer's design. Interior footings, if utilized, should extend to a depth of at least 12 inches; spread footings should be a minimum width of 24 inches. All footing excavations should be free of loose soil prior to placement of concrete. Footing excavations should be observed by the geotechnical engineer to evaluate dimensions and bearing material.

7.7.2 *Settlements*

Estimated settlements are expected to be less than approximately 1/2 inch for both spread and continuous footings. We anticipate that differential settlements across a 10-foot span could be up to one-half of the estimated total settlement of the footing.

7.8 **Concrete Slabs-On-Grade**

We recommend that concrete slab-on-grade floors be a minimum of 4-inches thick and be at least nominally reinforced. Actual slab thickness and reinforcement should be designed by the structural engineer.

All exterior flatwork should also be a minimum of 4 inches in thickness and be reinforced with 6 x 6 6/6 welded wire mesh. Prior to pouring concrete, the upper subgrade soils should be moistened to minimize the extraction of water from the concrete. All concrete slabs should be provided with expansion joints at regular intervals of approximately 15 feet each way to help control shrinkage cracks and thermal expansion/contraction.

If moisture-sensitive floor coverings are to be used, we recommend providing a suitable vapor barrier consisting of a plastic membrane sandwiched between 4 inches of sand.

7.9 **Lateral Resistance**

To provide resistance for lateral loads applied to footings and shear keys poured neat against vertical excavations, we recommend using an equivalent fluid pressure of 300 or 450 pcf for properly compacted granular fill or competent formational materials, respectively. These values assume a horizontal surface for the soil mass extending at least 10 feet from the face of the footing or three times the height of the surface generating the passive pressure, whichever is greater. The upper 12 inches of soil in areas not protected by floor slabs or pavements should not be included in design for passive resistance to lateral loads.

If friction is to be used to resist lateral loads, we recommend a coefficient of friction of 0.35 between soil and concrete for either compacted fill or formational soil. If it is desired to combine friction and passive resistance in design, we recommend reducing the friction coefficient by 25 percent.

7.10 **Retaining Walls**

7.10.1 *Retaining Wall Design - Static Conditions*

In selecting lateral earth pressures, active lateral earth pressures should only be used for cantilevered walls where a horizontal movement of at least 0.002H can be accommodated at the top of the wall, where H is the height of the wall in feet. If this condition is not satisfied, design criteria for the restrained or partially restrained condition should be used. We recommend providing all retaining walls with a backfill drainage system adequate to prevent the buildup of hydrostatic pressures. Recommended earth pressures for walls with select granular backfill are presented below.

Cantilevered Walls - For a cantilevered retaining wall with level granular backfill extending a minimum horizontal distance equal to the height of the wall, we recommend designing the wall for an active earth pressure equivalent to a fluid pressure of 30 pcf. This value assumes that no clayey soils are utilized for backfill and that no surcharge loads, such as adjacent footings or vehicle traffic, will act on the wall.

The materials that will be generated from the basement excavation in general consist of non-expansive granular sands characteristic of the Bay Point Formation. These materials are considered suitable for use as wall backfill.

If imported granular soils are used for wall backfill, we recommend that they conform to the Structure Backfill requirements outlined in the "Standard Specifications for Public Works Construction," Section 300-3.5.1.

Cantilevered retaining walls subjected to vehicular loads should be designed to resist an equivalent fluid pressure for the active case described above, plus an additional uniform lateral pressure equal to 60 psf.

Restrained Walls - We recommend that walls restrained from movement at the top, such as basement walls, be designed for the active case equivalent fluid pressure given above, plus an additional uniform load of 8H psf.

In our opinion, partially restrained retaining walls can be designed for a load reduction if they can be assumed to deflect. The additional uniform pressure that is added to the active condition equivalent fluid pressure should vary linearly from 8H psf uniform pressure to zero (0), as the calculated deflection varies from zero (0) to 0.002H.

It should be noted that while the perimeter drilled pier shoring wall will remain in place, the drilled pier shoring wall is likely significantly more flexible than the proposed basement wall, and thus the design earth pressures are expected to be fully transferred to the basement wall, requiring the basement wall to be designed to resist the above-noted design earth pressures.

7.10.2 *Retaining Wall Design - Seismic Conditions*

Dynamic lateral forces are imposed upon retaining structures during seismic shaking. Although it is not mandatory to include seismic loading in the sizing of structures, consideration should be given to mitigating a potential failure from overstressing foundation components during a design earthquake, such as the maximum probable earthquake. If it is desired to include this additional force, we recommend that the increased earth pressure due to seismic conditions be modeled as a point load acting at a point one-third of the height below the top of the wall. This increased force can be computed by assuming an inverted hydrostatic pressure equivalent to a fluid density of 29 pcf (assuming a design site acceleration of 0.32g, corresponding to the California Building Code design level earthquake). This value was based upon Mononobe-Okabe's modification of Coulomb's theory (Prakash, 1981). On the basis of Mononobe-Okabe's pseudo-static analysis, the additional seismic-induced lateral loading can be considered an upper-bound increase in lateral load due to seismic loading.

7.11 **Surface Drainage**

It is recommended that positive measures be taken to properly finish grade the lot after structures and other improvements are completed, in order that drainage waters from the pad and adjacent properties are directed off the site and away from foundations and floor slabs. Even when these measures have been taken, experience has shown that a shallow groundwater or surface water condition can and may develop in areas where no such water condition existed prior to site development. This is particularly true where relatively impervious soils are present at shallow depths, and where a substantial increase in surface water infiltration results from landscape irrigation.

To further reduce the possibility of moisture-related problems, we recommend that landscaping and irrigation be kept as far away from the building perimeter as possible. Irrigation water, especially close to the building, should be kept to the minimum required level. If large landscaped areas are planned next to the building, subdrains should be installed to intercept and drain excess infiltrated irrigation water away from the structure. We recommend that the ground surface in all areas be graded to slope away from the building foundations and floor slabs, and that all runoff water be directed to proper drainage areas and not be allowed to pond. A minimum ground slope of 2 percent is recommended for unpaved areas, and 1 percent for paved areas.

8 LIMITATIONS

Geotechnical engineering and the earth sciences are characterized by uncertainty. Professional judgments presented herein are based partly on our evaluation of the technical information gathered, partly on our understanding of the proposed construction, and partly on our general experience. Our engineering work and judgments rendered meet the current professional standards. We do not guarantee the performance of the project in any respect.

We have investigated only a small portion of the pertinent soil, rock, and groundwater conditions of the subject site. The opinions and conclusions made herein were based on the assumption that those rock and soil conditions do not deviate appreciably from those encountered during our field investigation. We recommend that a soil engineer from our office observe construction to assist in identifying soil conditions that may be significantly different from those encountered in our borings. Additional recommendations may be required at that time.

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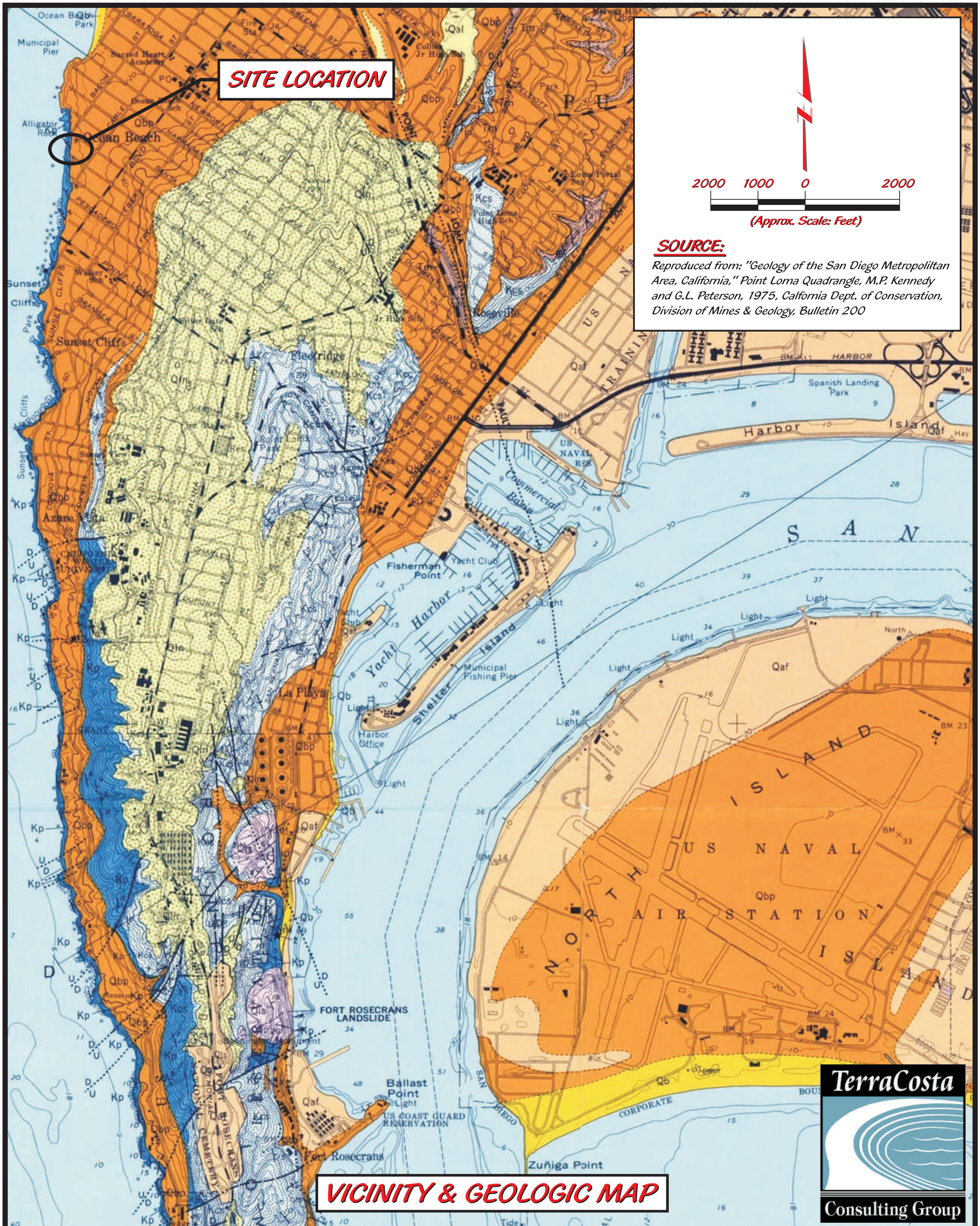
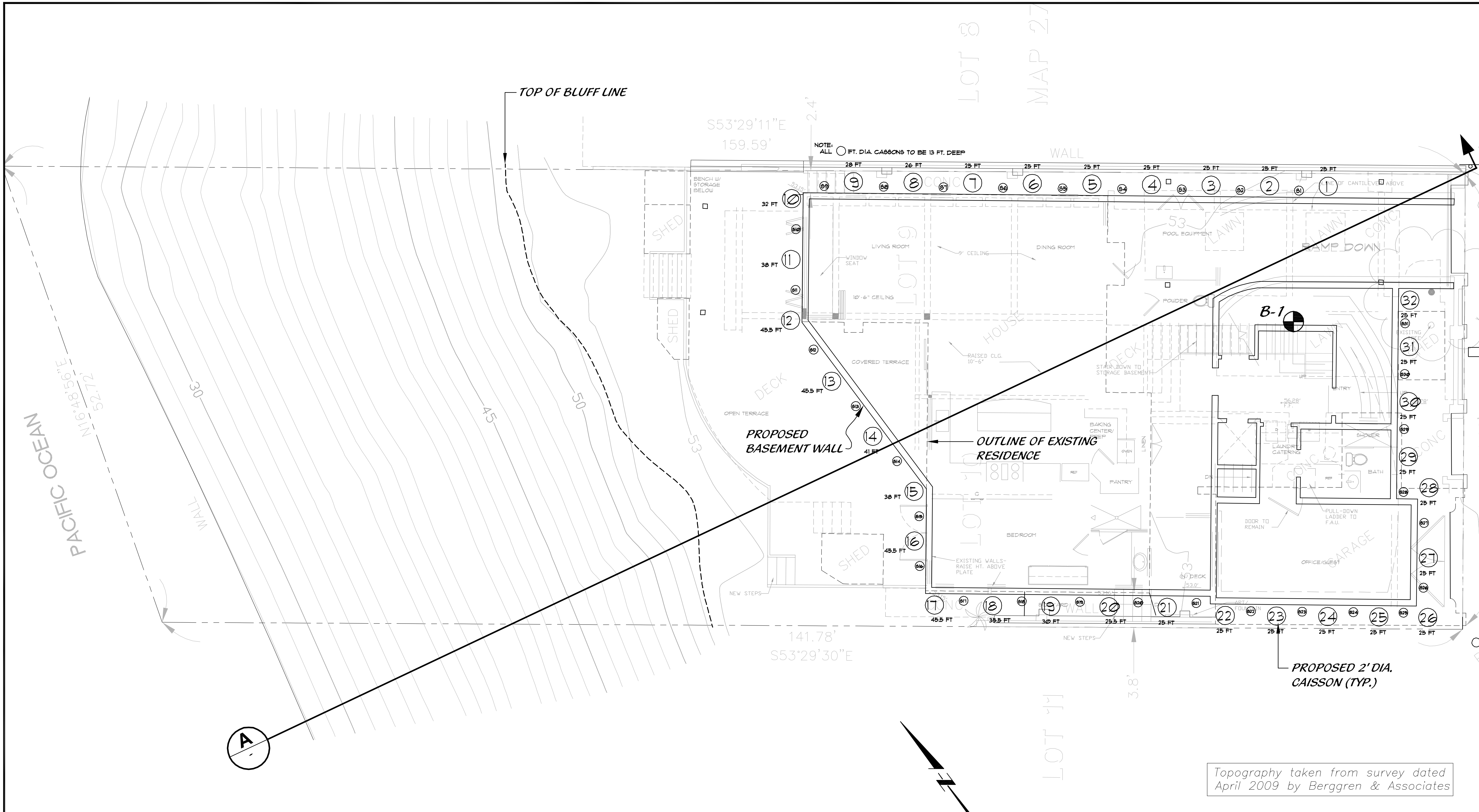
U.S. Army Corps of Engineers, Stereographic Aerial Photograph Nos. AXN-1-21 and 22.

USGS-EROS, 1966, Stereographic Aerial Photograph Nos. 21155 1-35 and 36, flown 10-26-66.

TABLE 1
DRILLER PIER DEPTHS
(Below Existing Grade)

Pier #	Depth (feet)
1 – 7	25
8	26
9	28
10	32
11	38
12	45.5
13	45.5
14	41
15	38
16	45.5
17	45.5
18	35.5
19	30
20	25.5
21 – 32	25
51 – S31	13


SITE LOCATION
2000 1000 0 2000
(Approx. Scale: Feet)
SOURCE:
Reproduced from: "Geology of the San Diego Metropolitan Area, California," Point Loma Quadrangle, M.P. Kennedy and G.L. Peterson, 1975, California Dept. of Conservation, Division of Mines & Geology, Bulletin 200
FORT ROSECRANS LANDSLIDE
VICINITY & GEOLOGIC MAP
TerraCosta
Consulting Group



LEGEND

B-1 APPROXIMATE LOCATION OF TCG SOIL BORING

TerraCosta Consulting Group

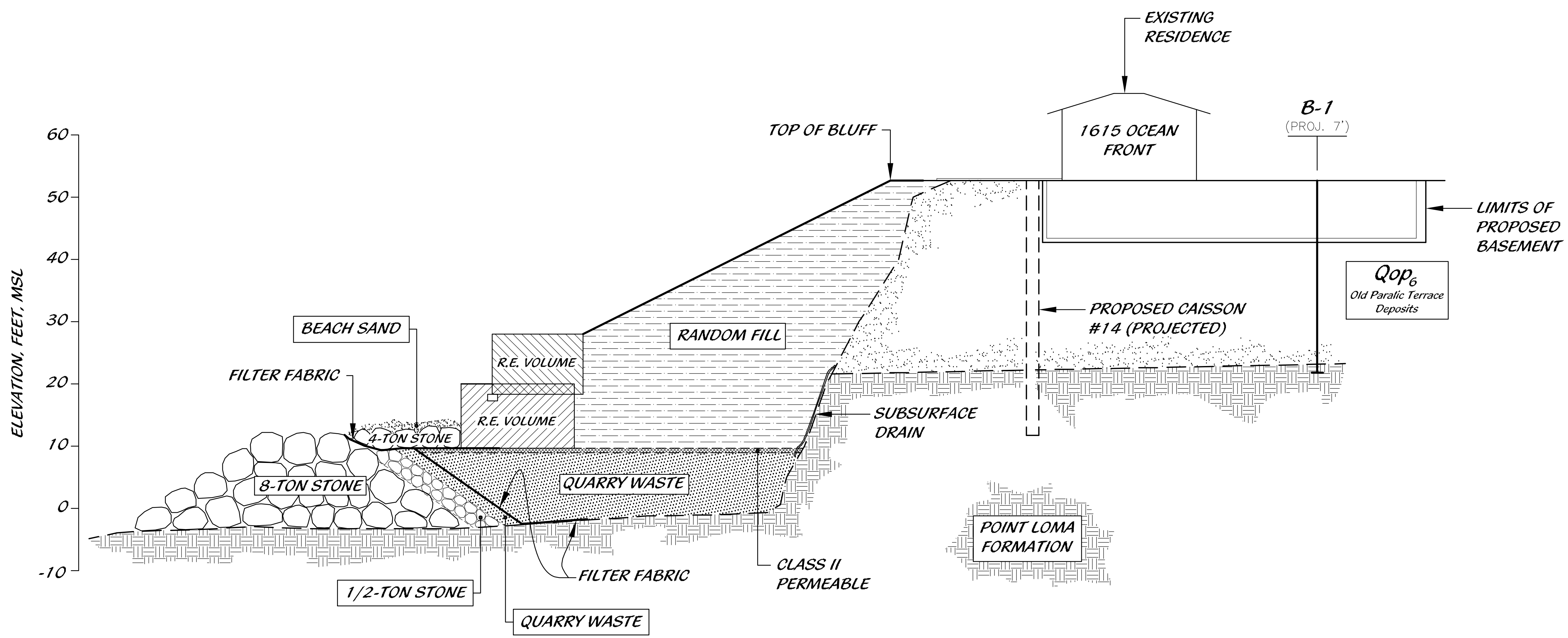
TERRACOSTA CONSULTING GROUP
ENGINEERS AND GEOLOGISTS
3890 MURPHY CANYON ROAD, SUITE 200
SAN DIEGO, CA 92123 (858) 573-6900

FIGURE NUMBER 2

PROJECT NAME: 1615 OCEAN FRONT

PROJECT NUMBER: 2660

SITE PLAN



CROSS SECTION 'A'

SCALE: 1"=20' (HORIZ. & VERT.)

TerraCosta Consulting Group

TERRACOSTA CONSULTING GROUP ENGINEERS AND GEOLOGISTS 3890 MURPHY CANYON ROAD, SUITE 200 SAN DIEGO, CA 92123 (858) 573-6900		FIGURE NUMBER 3
PROJECT NAME 1615 OCEAN FRONT		PROJECT NUMBER 2660
CROSS SECTION 'A'		


CORONADO AVENUE
DEL MAR AVENUE
1605
1615
1621-23
Qop6
Qaf
Kp
SW
RR
LEGEND
RR RIPRAP
Qaf FILL
Qop6 OLD PARALIC TERRACE DEPOSITS
Kp POINT LOMA FORMATION
SW SEAWALL
APPROXIMATE GEOLOGIC CONTACT
GEOLOGIC CONTACT OBSCURED
REFERENCE:
Date of Aerial Photo: October 29, 2001
TerraCosta
Consulting Group

APPENDIX A

LOG OF EXPLORATORY EXCAVATION

LOG OF TEST BORING

PROJECT NAME	PROJECT NUMBER	BORING
1615 OCEAN FRONT STREET	2660	LEGEND

SITE LOCATION	START	FINISH	SHEET NO.
1615 Ocean Front Street, San Diego	7/25/2016	7/25/2016	1 of 1

DRILLING COMPANY	DRILLING METHOD	LOGGED BY	CHECKED BY
Pacific Drilling	Hollow Stem Auger	B. Smillie	

DRILLING EQUIPMENT	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GROUND WATER (ft)
Mole Rig Hollow Stem Augers	8	20		▼ n/a

SAMPLING METHOD	NOTES
Standard Penetration Tests	

DEPTH (ft)	ELEVATION (ft)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS/ft)	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
5									
		S	1						
		PB	2						
10									
15									

KEY TO EXCAVATION LOGS

▼ WATER TABLE MEASURED AT TIME OF DRILLING

PENETRATION RESISTANCE (BLOWS/ft)

Number of blows required to advance the sampler 1 foot.

California Sampler blow counts can be converted to equivalent SPT blow counts by using an end-area conversion factor of 0.67 when using a 140-pound hammer and a 30-inch drop.

SAMPLE TYPE

S ("SPT") - a.k.a. Standard Penetration Test, an 18-inch-long, 2-inch O.D., 1-3/8-inch I.D. drive sampler.

PB ("Plastic Bag") - A disturbed, but representative sample obtained from a specific depth interval placed in a small sealable plastic bag.

NOTES ON FIELD INVESTIGATION

Borings were advanced using a track-mounted limited-access drill rig with a 6-inch hollow-stem auger.

Standard Penetration Tests (SPT) were used to obtain soil samples. The SPT Sampler was driven into the soil at the bottom of the borings with a 140-pound hammer falling 30 inches. When the sampler was withdrawn from the boring, the sample was removed, visually classified, sealed in plastic containers, and taken to the laboratory for detailed inspection.

No free groundwater was encountered in the boring as shown on the log.

Classifications are based upon the Unified Soil Classification System and include color, moisture, and consistency. Field descriptions have been modified to reflect results of laboratory inspection where deemed appropriate.

TerraCosta Consulting Group, Inc.
3890 Murphy Canyon Road, Suite 200
San Diego, California 92123

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED.

FIGURE A-1

LOG OF TEST BORING

PROJECT NAME	PROJECT NUMBER	BORING
1615 OCEAN FRONT STREET	2660	B-1

SITE LOCATION	START	FINISH	SHEET NO.
1615 Ocean Front Street, San Diego	7/25/2016	7/25/2016	1 of 2

DRILLING COMPANY	DRILLING METHOD	LOGGED BY	CHECKED BY
Pacific Drilling	Hollow Stem Auger	B. Smillie	

DRILLING EQUIPMENT	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GROUND WATER (ft)
Mole Rig Hollow Stem Augers	8	29.5		▼ n/a

SAMPLING METHOD	NOTES
Standard Penetration Tests	

DEPTH (ft)	ELEVATION (ft)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS/ft)	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
									FILL (Qaf) **Silty SAND and GRAVEL (SM/GM)**, light gray, dry to damp
		PB	1						
									OLD PARALIC TERRACE DEPOSITS (Qpo_6) **Silty Fine SAND (SM)**, medium dense to dense, red-brown, damp
		PB	2						
5									
									- Damp to moist
		PB	3						- Red-brown to light gray-brown
10									
		S	4	36					
15									
		PB	5						
									Silty Fine SAND (SM), dense, brown, damp

TCG_METRIC_LOG(3) 2660.GPJ GDCLOGMT.GDT 8/23/16

LOG OF TEST BORING

PROJECT NAME	PROJECT NUMBER	BORING
1615 OCEAN FRONT STREET	2660	**B-1**

SITE LOCATION	START	FINISH	SHEET NO.
1615 Ocean Front Street, San Diego	7/25/2016	7/25/2016	2 of 2

DRILLING COMPANY	DRILLING METHOD	LOGGED BY	CHECKED BY
Pacific Drilling	Hollow Stem Auger	B. Smillie	

DRILLING EQUIPMENT	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GROUND WATER (ft)
Mole Rig Hollow Stem Augers	8	29.5		▼ n/a

SAMPLING METHOD	NOTES
Standard Penetration Tests	

DEPTH (ft)	ELEVATION (ft)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS/ft)	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
		PB							
		S	6	43					
25									
		PB	7						
									POINT LOMA FORMATION **Cemented SANDSTONE (SM)**, very dense, gray, damp
30									*Boring terminated at depth of 29.5 feet due to practical refusal.* *Attempted Standard Penetration Test at 29.5 feet (50 blows for 1-inch - no recovery).* *No free groundwater encountered at time of excavation.*
35									

TerraCosta Consulting Group, Inc.
3890 Murphy Canyon Road, Suite 200
San Diego, California 92123

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED.

FIGURE A-2 b

TCG_METRIC_LOG(3) 2660.GPJ GDCLOGMT.GDT 8/23/16

APPENDIX B

SPECIFICATIONS FOR ENGINEERED FILL

TerraCosta
Consulting Group

APPENDIX B

SPECIFICATIONS FOR ENGINEERED FILL

These specifications present the usual and minimum requirements for grading operations performed under observation and testing of TerraCosta Consulting Group, Inc.

No deviation from these specifications should be allowed, except where specifically superseded in the preliminary geology and soils report, or in other written communication signed by the Geotechnical Engineer or Engineering Geologist.

I. <u>GENERAL</u>

A. The Geotechnical Engineer and Engineering Geologist are the Owner's or Builder's representative on the project. For the purpose of these specifications, observation and testing by the Geotechnical Engineer includes that observation and testing performed by any person or persons employed by, and responsible to, the licensed Geotechnical Engineer signing the soils report.

B. The Contractor under the observation of the Geotechnical Engineer shall conduct, all clearing, site preparation, or earthwork performed on the project.

C. It is the Contractor's responsibility to prepare the ground surface to receive the fills and to place, spread, mix, water, and compact the fill in accordance with the specifications of the Geotechnical Engineer. The Contractor shall also remove all material considered unsuitable for use in the engineered fill by the Geotechnical Engineer.

D. It is also the Contractor's responsibility to have suitable and sufficient compaction equipment on the job-site to handle the amount of fill being placed. If necessary, excavation equipment will be shut down to permit completion of compaction. Sufficient watering apparatus will also be provided by the Contractor, with due consideration for the fill material, rate of placement, and time of year.

E. The Geotechnical Engineer and Engineering Geologist will issue a final report summarizing their observations, test results, and comments regarding the Contractor's conformance with these specifications.

II. SITE PREPARATION

A. In areas to be graded, all vegetation and deleterious material such as rubbish and any construction debris from previous structures shall be disposed of off site. This removal must be concluded prior to placing fill.

B. The Civil Engineer shall locate all sewage disposal systems and large structures on the site or on the grading plan to the best of his knowledge prior to preparing the ground surface.

C. Soil, alluvium, or rock materials determined by the Geotechnical Engineer as being unsuitable for placement in compacted fills shall be removed and wasted from the site. The Geotechnical Engineer is to approve any material incorporated as a part of a compacted fill.

D. After the ground surface to receive fill has been cleared, it shall be scarified, disced, or bladed by the Contractor until it is uniform and free from ruts, hollows, hummocks or other uneven features that may prevent uniform compaction.

The scarified ground surface shall then be brought to optimum moisture, mixed as required, and compacted as specified. If the scarified zone is greater than 12 inches in depth, the excess shall be removed and placed in lifts on the order of 6 to 8 inches, depending upon material type and available construction equipment.

Prior to placing fill, the ground surface to receive fill shall be inspected, tested, and approved by the Geotechnical Engineer.

E. Any abandoned building, foundations, or underground structures, such as pipelines, or others not located prior to grading, are to be removed or treated in a manner prescribed by the Geotechnical Engineer.

III. COMPACTED FILLS

A. Any material imported or excavated on the property may be utilized in the fill, provided each material has been determined to be suitable by the Geotechnical Engineer. Roots, tree branches, and other matter missed during clearing shall be removed from the fill.

B. Rock fragments less than 6 inches in diameter may be utilized in the fill provided:

1. They are not placed in concentrated pockets.

2. There is a sufficient percentage of fine-grained material to surround the rocks.

3. The distribution of the rocks is to be observed by the Geotechnical Engineer.

C. Rocks greater than 12 inches in diameter shall be taken off site.

D. Material that is spongy, subject to decay, or otherwise considered unsuitable shall not be used in the compacted fill.

E. Representative samples of materials to be utilized as compacted fill shall be analyzed in the laboratory by the Geotechnical Engineer to determine their physical properties. If any material other than that previously tested is encountered during grading, the appropriate analysis of this material shall be conducted by the Geotechnical Engineer as soon as possible.

F. Material used in the compacting process shall be evenly spread, watered or dried, processed and compacted in thin lifts to obtain a uniformly dense layer. Lift thickness shall be on the order of 6 to 8 inches. The fill shall be placed and compacted on a horizontal plane, unless otherwise approved by the Geotechnical Engineer.

G. If the moisture content or relative compaction varies from that required by the Geotechnical Engineer, the Contractor shall rework the fill until it is approved by the Geotechnical Engineer.

H. Each layer shall be compacted to 90 percent (90%) of the maximum density in compliance with the testing method specified by the controlling governmental agency. (In general, ASTM D 1557 will be used.)

IV. GRADING CONTROL

A. Inspection of the fill placement shall be provided by the Geotechnical Engineer during the progress of grading.

B. In general, density tests should be made at intervals not exceeding 2 feet of fill height. An adequate number of field density tests determined by the Geotechnical Engineer shall be made to verify that the required compaction is being achieved. The number of tests will vary depending on the soil conditions and the size of the job.

C. Density tests should also be made on the surface of the soils to receive fill as required by the Geotechnical Engineer.

D. All cleanout, processed ground to receive fill, key excavations, subdrains and rock disposal must be inspected and approved by the

Geotechnical Engineer (and often by the governing authorities) prior to placing any fill. It shall be the Contractor's responsibility to notify the Geotechnical Engineer and governing authorities when such areas are ready for inspection.

V. CONSTRUCTION CONSIDERATIONS

A. Erosion control measures, when necessary, shall be provided by the Contractor during grading prior to the completion and construction of permanent drainage controls.

B. Upon completion of grading and termination of observations by the Geotechnical Engineer, no further filling or excavating, including that necessary for footings, foundations, large tree wells, retaining walls, or other features shall be performed without the approval of the Geotechnical Engineer or Engineering Geologist.

C. Care shall be taken by the Contractor during final grading to preserve any berms, drainage terraces, interceptor swales, or other devices of a permanent nature on or adjacent to the property.

VI. ON-PAD UTILITY TRENCH BACKFILL RECOMMENDATIONS

A. SHALLOW TRENCHES: (Maximum Trench Depth of 2 Feet). Use soils approved by the Geotechnical Engineer. The soils should be compacted to 90 percent of the maximum dry density, as determined by ASTM Test Method D 1557, and tested by the Geotechnical Engineer. Compaction by flooding or jetting will be permitted only when, in the opinion of the Geotechnical Engineer, the backfill materials have a Sand Equivalent of at least 30 and the foundation materials will not soften or be damaged by the applied water.

B. DEEP TRENCHES: (Depth of Trench Greater than 2 Feet). The soils should be compacted to 90 percent of the maximum density, as determined by ASTM Test Method D 1557, and tested by the Geotechnical Engineer. The backfill placement method should consist of mechanically compacting the backfill soils throughout the trench depth.

If trench depth extends 5 feet, placement/compaction method should be reviewed by the Geotechnical Engineer. Contractor should exercise, and is responsible for, necessary and required safety precautions in all trenching operations.

C. TRENCHES UNDER VEHICLE PAVEMENTS: A minimum of 3 feet of fill should be placed over conduit, apply criteria B, above.

D. TRENCHES NEAR FOOTINGS: Approved backfill soils must be mechanically compacted to 90 percent of the maximum density, as determined by ASTM Test Method D 1557, and tested by the Geotechnical Engineer. The general backfill technique will be in accordance with the applicable criteria stated in A, above.

E. REPORTING: If the Geotechnical Engineer will be providing a written opinion as to adequacy of soil compaction and trench backfill, the entire operation should be performed under the Geotechnical Engineer's observation and testing.

APPENDIX C

LATERALLY LOADED SHORING CALCULATIONS

TerraCosta Consulting Group

24" Pile @ 8' On Center

Laterally Loaded Shoring Analysis - 1615 Ocean Front - 4/20/16																	
24" Diameter CIDH Shafts @ 8' OC																	
Reese & Matlock solution - DM7.02																	
Pile Moment of Inertia, I (in^4):				16,278													
Pile Diameter, D (in):				24.00													
Pile Modulus, E (psi):				3,000,000		Ultimate lateral soil capacity ref: Brom's 1964											
Soil Modulus, f (pci):				40.00		Pult=0.5*soil-density*D*L^3*Kp/(H+L) for L/T<2											
Unsupported Cantilevered Height, H (ft):				10.00		Pult=M/(H+0.54(P/soil-density*D*Kp)^0.5) for L/T>4											
Depth of Embedment, L (ft):				15.00													
Point of load application, b (ft)				3.33		Soil phi, degrees		33									
						Soil density, pcf		125									
Effective Depth, T (in):				65.66						Pult(kips)	21.86	Long Pile					
Effective Depth, T (ft):				5.47						Pult(kips)	57.24	short Pile					
Lateral Load, P (kips):				6.00		lever arm		3.33		Note: Use the smaller of the two							
Load Induced Moment, M (Kip-ft):				19.98		Kp		3.39		Also note: to abtain the ultimate capacity for a long pile,							
Embedment Depth Ratio, L/T:				2.74		Myield,Mtotal(Kip-ft);		250		you must balance E15 and L13 to obtain the correct answer							
Computation of Variation in Soil Induced Moment with L/T = 4										Brom's embedment FS =		13.01					
Depth,T	Depth,ft	Fmm	Fpt	Mm	Mpt	Mtotal		Fiber Bending, Fb (psi)		FS=0.5*soil-density*D*L^3*Kp/P(L+b) ref. Coduto eq. 17-4							
0.00	0.00	1.000	0.000	19.98	0.00	19.98		177									
0.25	1.37	0.992	0.240	19.82	7.88	27.70		245									
0.50	2.74	0.970	0.467	19.38	15.33	34.71		307									
0.75	4.10	0.926	0.627	18.50	20.59	39.09		346									
1.00	5.47	0.859	0.732	17.16	24.03	41.20		364									
1.25	6.84	0.753	0.767	15.04	25.18	40.23		356									
1.50	8.21	0.640	0.747	12.79	24.53	37.31		330									
Computation of Pile Deformation with L/T = 4																	
Depth, T	Depth, ft	Fdm	Fdp	DEF.m	DEF.pt	DEF tot,"		SLOPE		Top of Pile Def (in)							
0.00	0.00	1.56	2.50	0.03	0.09	0.12	"	0.00143902			0.30	"					
0.25	1.37	1.16	2.07	0.02	0.07	0.10	"	0.00134824									
0.50	2.74	0.82	1.65	0.02	0.06	0.07	"	0.001146461		NOTE: Top of pile deflection is the combination of:							
0.75	4.10	0.52	1.30	0.01	0.05	0.06	"	0.00098794		Ground surface deflection, DEF tot." PLUS						0.12	"
1.00	5.47	0.30	0.97	0.01	0.03	0.04	"	0.000851531		Deflected pile due to angular rotation only, slope*Ht. PLUS						0.17	"
1.25	6.84	0.12	0.67	0.00	0.02	0.03	"	0.000579162		Deflected pile due to loading,Pb^2/6EI(3*L-b)						0.01	"
1.50	8.21	0.03	0.44	0.00	0.02	0.02	"			where: L=lever arm							

<u>24" Pile Loading @ 8' On Center</u>

Laterally Loaded Shoring Analysis - 1615 Ocean Front - 4/20/16

24" Diameter CIDH Shafts @ 8' OC

Reese & Matlock solution - DM7.02

|||||||||||||||||||||||||||||||||||||||||||||||||||||||||||||||||||||||||||||||||||

<table>
<tr><td>Pile Moment of Inertia, I (in^4):</td><td>16,278</td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>Pile Diameter, D (in):</td><td>24.00</td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td>Pile Modulus, E (psi):</td><td>3,000,000</td><td>Ultimate lateral soil capacity ref: Brom's 1964</td><td></td><td></td><td></td><td></td></tr>
<tr><td>Soil Modulus, f (pci):</td><td>40.00</td><td>Pult=0.5*soil-density*D*L^3*Kp/(H+L) for L/T<2</td><td></td><td></td><td></td><td></td></tr>
<tr><td>Unsupported Cantilevered Height, H (ft):</td><td>10.00</td><td>Pult=M/(H+0.54(P/soil-density*D*Kp)^0.5) for L/T>4</td><td></td><td></td><td></td><td></td></tr>
<tr><td>Depth of Embedment, L (ft):</td><td>15.00</td><td>||||||||||||||||||||||||||||||||||||||||||||||||||||||||||||</td><td></td><td></td><td></td><td></td></tr>
<tr><td>Point of load application, b (ft)</td><td>3.33</td><td>Soil phi, degrees</td><td>33</td><td></td><td></td><td></td></tr>
<tr><td></td><td></td><td>Soil density, pcf</td><td>125</td><td></td><td></td><td></td></tr>
<tr><td>Effective Depth, T (in):</td><td>65.66</td><td></td><td></td><td>Pult(kips)</td><td>20.78</td><td>Long Pile</td></tr>
<tr><td>Effective Depth, T (ft):</td><td>5.47</td><td></td><td></td><td>Pult(kips)</td><td>57.24</td><td>short Pile</td></tr>
<tr><td>Lateral Load, P (kips):</td><td>12.00</td><td>lever arm</td><td>3.33</td><td>Note: Use the smaller of the two</td><td></td><td></td></tr>
<tr><td>Load Induced Moment, M (Kip-ft):</td><td>39.96</td><td>Kp</td><td>3.39</td><td>Also note: to abtain the ultimate capacity for a long pile,</td><td></td><td></td></tr>
<tr><td>Embedment Depth Ratio, L/T:</td><td>2.74</td><td>Myield,Mtotal(Kip-ft);</td><td>250</td><td>you must balance E15 and L13 to obtain the correct answer</td><td></td><td></td></tr>
</table>

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Computation of Variation in Soil Induced Moment with L/T = 4

Brom's embedment FS = 6.51

FS=0.5*soil-density*D*L^3*Kp/P(L+b) ref. Coduto eq. 17-4

Depth,T	Depth,ft	Fmm	Fpt	Mm	Mpt	Mtotal	Fiber Bending, Fb (psi)
0.00	0.00	1.000	0.000	39.96	0.00	39.96	353
0.25	1.37	0.992	0.240	39.64	15.76	55.40	490
0.50	2.74	0.970	0.467	38.76	30.67	69.43	614
0.75	4.10	0.926	0.627	37.00	41.17	78.17	692
1.00	5.47	0.859	0.732	34.33	48.07	82.39	729
1.25	6.84	0.753	0.767	30.09	50.36	80.45	712
1.50	8.21	0.640	0.747	25.57	49.05	74.63	660

::

Computation of Pile Deformation with L/T = 4

Depth, T	Depth, ft	Fdm	Fdp	DEF.m	DEF.pt	DEF tot,"		SLOPE
0.00	0.00	1.56	2.50	0.07	0.17	0.24	"	0.00287803
0.25	1.37	1.16	2.07	0.05	0.14	0.19	"	0.00269648
0.50	2.74	0.82	1.65	0.03	0.11	0.15	"	0.002292922
0.75	4.10	0.52	1.30	0.02	0.09	0.11	"	0.001975879
1.00	5.47	0.30	0.97	0.01	0.07	0.08	"	0.001703062
1.25	6.84	0.12	0.67	0.00	0.05	0.05	"	0.001158324
1.50	8.21	0.03	0.44	0.00	0.03	0.03	"	

Top of Pile Def (in) 0.61 "

NOTE: Top of pile deflection is the combination of:		
Ground surface deflection, DEF tot." PLUS	0.24	"
Deflected pile due to angular rotation only, slope*Ht. PLUS	0.35	"
Deflected pile due to loading,Pb^2/6EI(3*L-b)	0.02	"
where: L=lever arm		

24" Pile Seismic Design @ 8' On Center

Laterally Loaded Shoring Analysis - 1615 Ocean Front - 4/20/16																	
24" Diameter CIDH Shafts @ 8' OC																	
Reese & Matlock solution - DM7.02																	
Pile Moment of Inertia, I (in^4):				16,278													
Pile Diameter, D (in):				24.00													
Pile Modulus, E (psi):				3,000,000		Ultimate lateral soil capacity ref: Brom's 1964											
Soil Modulus, f (pci):				40.00		Pult=0.5*soil-density*D*L^3*Kp/(H+L) for L/T<2											
Unsupported Cantilevered Height, H (ft):				10.00		Pult=M/(H+0.54(P/soil-density*D*Kp)^0.5) for L/T>4											
Depth of Embedment, L (ft):				15.00													
Point of load application, b (ft)				5.42		Soil phi, degrees		33									
						Soil density, pcf		125									
Effective Depth, T (in):				65.66						Pult(kips)	19.57	Long Pile					
Effective Depth, T (ft):				5.47						Pult(kips)	57.24	short Pile					
Lateral Load, P (kips):				22.40		lever arm		5.42		Note: Use the smaller of the two							
Load Induced Moment, M (Kip-ft):				121.41		Kp		3.39		Also note: to abtain the ultimate capacity for a long pile,							
Embedment Depth Ratio, L/T:				2.74		Myield,Mtotal(Kip-ft);		250		you must balance E15 and L13 to obtain the correct answer							
//																	
Computation of Variation in Soil Induced Moment with L/T = 4										Brom's embedment FS =		3.13					
Depth,T	Depth,ft	Fmm	Fpt	Mm	Mpt	Mtotal		Fiber Bending, Fb (psi)		FS=0.5*soil-density*D*L^3*Kp/P(L+b) ref. Coduto eq. 17-4							
0.00	0.00	1.000	0.000	121.41	0.00	121.41		1074									
0.25	1.37	0.992	0.240	120.44	29.42	149.85		1326									
0.50	2.74	0.970	0.467	117.77	57.24	175.01		1548									
0.75	4.10	0.926	0.627	112.42	76.85	189.28		1674									
1.00	5.47	0.859	0.732	104.29	89.72	194.01		1716									
1.25	6.84	0.753	0.767	91.42	94.01	185.43		1640									
1.50	8.21	0.640	0.747	77.70	91.56	169.26		1497									
::																	
Computation of Pile Deformation with L/T = 4																	
Depth, T	Depth, ft	Fdm	Fdp	DEF.m	DEF.pt	DEF tot,"		SLOPE		Top of Pile Def (in)							
0.00	0.00	1.56	2.50	0.20	0.32	0.53	"	0.00660903			1.41	"					
0.25	1.37	1.16	2.07	0.15	0.27	0.42	"	0.00610708									
0.50	2.74	0.82	1.65	0.10	0.21	0.32	"	0.005228561		NOTE: Top of pile deflection is the combination of:							
0.75	4.10	0.52	1.30	0.06	0.17	0.23	"	0.004364617		Ground surface deflection, DEF tot." PLUS						0.53	"
1.00	5.47	0.30	0.97	0.03	0.13	0.16	"	0.003684694		Deflected pile due to angular rotation only, slope*Ht. PLUS						0.79	"
1.25	6.84	0.12	0.67	0.01	0.09	0.10	"	0.002377226		Deflected pile due to loading,Pb^2/6EI(3*L-b)						0.10	"
1.50	8.21	0.03	0.44	0.00	0.06	0.06	"			where: L=lever arm							

24" Pile @ 5.5' On Center

Laterally Loaded Shoring Analysis - 1615 Ocean Front - 4/20/16						
24" Diameter CIDH Shafts @ 5.5' OC						
Reese & Matlock solution - DM7.02						
||||||||||||||||||||||||||||||||||||||||||||||||||||||||||||||||||||||||||||||||||						
Pile Moment of Inertia, I (in^4):	16,278					
Pile Diameter, D (in):	24.00					
Pile Modulus, E (psi):	3,000,000	Ultimate lateral soil capacity ref: Brom's 1964				
Soil Modulus, f (pci):	40.00	Pult=0.5*soil-density*D*L^3*Kp/(H+L) for L/T<2				
Unsupported Cantilevered Height, H (ft):	10.00	Pult=M/(H+0.54(P/soil-density*D*Kp)^0.5) for L/T>4				
Depth of Embedment, L (ft):	15.00	||||||||||||||||||||||||||||||||||||||||||||||||||||||||||||				
Point of load application, b (ft)	3.33	Soil phi, degrees	33			
		Soil density, pcf	125			
Effective Depth, T (in):	65.66			Pult(kips)	22.34	Long Pile
Effective Depth, T (ft):	5.47			Pult(kips)	57.24	short Pile
Lateral Load, P (kips):	4.13	lever arm	3.33	Note: Use the smaller of the two		
Load Induced Moment, M (Kip-ft):	13.74	Kp	3.39	Also note: to abtain the ultimate capacity for a long pile,		
Embedment Depth Ratio, L/T:	2.74	Myield,Mtotal(Kip-ft);	250	you must balance E15 and L13 to obtain the correct answer		

//

Computation of Variation in Soil Induced Moment with L/T = 4									Brom's embedment FS =	18.92
Depth,T	Depth,ft	Fmm	Fpt	Mm	Mpt	Mtotal		Fiber Bending, Fb (psi)	FS=0.5*soil-density*D*L^3*Kp/P(L+b) ref. Coduto eq. 17-4	
0.00	0.00	1.000	0.000	13.74	0.00	13.74		122		
0.25	1.37	0.992	0.240	13.63	5.42	19.04		168		
0.50	2.74	0.970	0.467	13.32	10.54	23.87		211		
0.75	4.10	0.926	0.627	12.72	14.15	26.87		238		
1.00	5.47	0.859	0.732	11.80	16.52	28.32		251		
1.25	6.84	0.753	0.767	10.34	17.31	27.66		245		
1.50	8.21	0.640	0.747	8.79	16.86	25.65		227		

::

Computation of Pile Deformation with L/T = 4											
Depth, T	Depth, ft	Fdm	Fdp	DEF.m	DEF.pt	DEF tot,"		SLOPE	Top of Pile Def (in)		
0.00	0.00	1.56	2.50	0.02	0.06	0.08	"	0.00098932		0.21	"
0.25	1.37	1.16	2.07	0.02	0.05	0.07	"	0.00092691			
0.50	2.74	0.82	1.65	0.01	0.04	0.05	"	0.000788192	NOTE: Top of pile deflection is the combination of:		
0.75	4.10	0.52	1.30	0.01	0.03	0.04	"	0.000679209	Ground surface deflection, DEF tot." PLUS	0.08	"
1.00	5.47	0.30	0.97	0.00	0.02	0.03	"	0.000585428	Deflected pile due to angular rotation only, slope*Ht. PLUS	0.12	"
1.25	6.84	0.12	0.67	0.00	0.02	0.02	"	0.000398174	Deflected pile due to loading,Pb^2/6EI(3*L-b)	0.01	"
1.50	8.21	0.03	0.44	0.00	0.01	0.01	"		where: L=lever arm		

24" Pile Design Loading @ 5.5' On Center

Laterally Loaded Shoring Analysis - 1615 Ocean Front - 4/20/16
24" Diameter CIDH Shafts @ 8' OC
Reese & Matlock solution - DM7.02

Pile Moment of Inertia, I (in^4):	16,278
Pile Diameter, D (in):	24.00
Pile Modulus, E (psi):	3,000,000
Soil Modulus, f (pci):	40.00
Unsupported Cantilevered Height, H (ft):	10.00
Depth of Embedment, L (ft):	15.00
Point of load application, b (ft)	3.33
Effective Depth, T (in):	65.66
Effective Depth, T (ft):	5.47
Lateral Load, P (kips):	8.25
Load Induced Moment, M (Kip-ft):	27.47
Embedment Depth Ratio, L/T:	2.74

Ultimate lateral soil capacity ref: Brom's 1964
Pult=0.5*soil-density*D*L^3*Kp/(H+L) for L/T<2
Pult=M/(H+0.54(P/soil-density*D*Kp)^0.5) for L/T>4

Soil phi, degrees	33
Soil density, pcf	125
lever arm	3.33
Kp	3.39
Myield,Mtotal(Kip-ft);	250

Pult(kips)	21.40	Long Pile
Pult(kips)	57.24	short Pile

Note: Use the smaller of the two
Also note: to obtain the ultimate capacity for a long pile,
you must balance E15 and L13 to obtain the correct answer

Brom's embedment FS = 9.46
FS=0.5*soil-density*D*L^3*Kp/P(L+b) ref. Coduto eq. 17-4

Computation of Variation in Soil Induced Moment with L/T = 4

Depth,T	Depth,ft	Fmm	Fpt	Mm	Mpt	Mtotal	Fiber Bending, Fb (psi)
0.00	0.00	1.000	0.000	27.47	0.00	27.47	243
0.25	1.37	0.992	0.240	27.25	10.83	38.09	337
0.50	2.74	0.970	0.467	26.65	21.08	47.73	422
0.75	4.10	0.926	0.627	25.44	28.31	53.75	475
1.00	5.47	0.859	0.732	23.60	33.05	56.64	501
1.25	6.84	0.753	0.767	20.69	34.63	55.31	489
1.50	8.21	0.640	0.747	17.58	33.72	51.31	454

Computation of Pile Deformation with L/T = 4

Depth, T	Depth, ft	Fdm	Fdp	DEF.m	DEF.pt	DEF tot,"		SLOPE
0.00	0.00	1.56	2.50	0.05	0.12	0.16	"	0.00197865
0.25	1.37	1.16	2.07	0.03	0.10	0.13	"	0.00185383
0.50	2.74	0.82	1.65	0.02	0.08	0.10	"	0.001576384
0.75	4.10	0.52	1.30	0.01	0.06	0.08	"	0.001358417
1.00	5.47	0.30	0.97	0.01	0.05	0.05	"	0.001170855
1.25	6.84	0.12	0.67	0.00	0.03	0.03	"	0.000796348
1.50	8.21	0.03	0.44	0.00	0.02	0.02	"	

Top of Pile Def (in) 0.42 "

NOTE: Top of pile deflection is the combination of:		
Ground surface deflection, DEF tot." PLUS	0.16	"
Deflected pile due to angular rotation only, slope*Ht. PLUS	0.24	"
Deflected pile due to loading,Pb^2/6EI(3*L-b)	0.01	"
where: L=lever arm		

12" Pile @ 6' On Center

Laterally Loaded Shoring Analysis - 1615 Ocean Front - 4/20/16
12" Diameter CIDH Shafts @ 6' OC
Reese & Matlock solution - DM7.02

Pile Moment of Inertia, I (in^4):	1,018
Pile Diameter, D (in):	12.00
Pile Modulus, E (psi):	3,000,000
Soil Modulus, f (pci):	40.00
Unsupported Cantilevered Height, H (ft):	10.00
Depth of Embedment, L (ft):	3.00
Point of load application, b (ft)	3.33
Effective Depth, T (in):	37.72
Effective Depth, T (ft):	3.14
Lateral Load, P (kips):	0.80
Load Induced Moment, M (Kip-ft):	2.66
Embedment Depth Ratio, L/T:	0.95

Ultimate lateral soil capacity ref: Brom's 1964
Pult=0.5*soil-density*D*L^3*Kp/(H+L) for L/T<2
Pult=M/(H+0.54(P/soil-density*D*Kp)^0.5) for L/T>4

Soil phi, degrees	33
Soil density, pcf	125
lever arm	3.33
Kp	3.39
Myield,Mtotal(Kip-ft);	250

Pult(kips)	23.27	Long Pile
Pult(kips)	0.44	short Pile

Note: Use the smaller of the two
Also note: to obtain the ultimate capacity for a long pile,
you must balance E15 and L13 to obtain the correct answer

Brom's embedment FS = 1.13
FS=0.5*soil-density*D*L^3*Kp/P(L+b) ref. Coduto eq. 17-4

Computation of Variation in Soil Induced Moment with L/T = 4

Depth,T	Depth,ft	Fmm	Fpt	Mm	Mpt	Mtotal	Fiber Bending, Fb (psi)
0.00	0.00	1.000	0.000	2.66	0.00	2.66	188
0.25	0.79	0.992	0.240	2.64	0.60	3.25	230
0.50	1.57	0.970	0.467	2.58	1.17	3.76	266
0.75	2.36	0.926	0.627	2.47	1.58	4.04	286
1.00	3.14	0.859	0.732	2.29	1.84	4.13	292
1.25	3.93	0.753	0.767	2.01	1.93	3.93	278
1.50	4.71	0.640	0.747	1.70	1.88	3.58	253

Computation of Pile Deformation with L/T = 4

Depth, T	Depth, ft	Fdm	Fdp	DEF.m	DEF.pt	DEF tot,"		SLOPE
0.00	0.00	1.56	2.50	0.02	0.04	0.06	"	0.00128740
0.25	0.79	1.16	2.07	0.02	0.03	0.05	"	0.00118728
0.50	1.57	0.82	1.65	0.01	0.02	0.04	"	0.001017477
0.75	2.36	0.52	1.30	0.01	0.02	0.03	"	0.000845429
1.00	3.14	0.30	0.97	0.00	0.01	0.02	"	0.000711507
1.25	3.93	0.12	0.67	0.00	0.01	0.01	"	0.000455256
1.50	4.71	0.03	0.44	0.00	0.01	0.01	"	

Top of Pile Def (in) 0.24 "

NOTE: Top of pile deflection is the combination of:

Ground surface deflection, DEF tot." PLUS	0.06	"
Deflected pile due to angular rotation only, slope*Ht. PLUS	0.15	"
Deflected pile due to loading,Pb^2/6EI(3*L-b)	0.02	"

where: L=lever arm

Naval Facilities Engineering Command
200 Stovall Street
Alexandria, Virginia 22332-2300

Foundations & Earth Structures

DESIGN MANUAL 7.02
REVALIDATED BY CHANGE 1 SEPTEMBER 1986

Section 7. LATERAL LOAD CAPACITY

1. DESIGN CONCEPTS. A pile loaded by lateral thrust and/or moment at its top, resists the load by deflecting to mobilize the reaction of the surrounding soil. The magnitude and distribution of the resisting pressures are a function of the relative stiffness of pile and soil.

Design criteria is based on maximum combined stress in the piling, allowable deflection at the top or permissible bearing on the surrounding soil. Although 1/4-inch at the pile top is often used as a limit, the allowable lateral deflection should be based on the specific requirements of the structure.

2. DEFORMATION ANALYSIS - SINGLE PILE.

a. General. Methods are available (e.g., Reference 9 and Reference 31, Non-Dimensional Solutions for Laterally Loaded Piles, with Soil Modulus Assumed Proportional to Depth, by Reese and Matlock) for computing lateral pile load-deformation based on complex soil conditions and/or non-linear soil stress-strain relationships. The COM 622 computer program (Reference 32, Laterally Loaded Piles: Program Documentation, by Reese) has been documented and is widely used. Use of these methods should only be considered when the soil stress-strain properties are well understood.

Pile deformation and stress can be approximated through application of several simplified procedures based on idealized assumptions. The two basic approaches presented below depend on utilizing the concept of coefficient of lateral subgrade reaction. It is assumed that the lateral load does not exceed about 1/3 of the ultimate lateral load capacity.

b. Granular Soil and Normally to Slightly Overconsolidated Cohesive Soils. Pile deformation can be estimated assuming that the coefficient of subgrade reaction, K_h, increases linearly with depth in accordance with:

$$K_h = \frac{fz}{D}$$

where: K_h = coefficient of lateral subgrade reaction (tons/ft^3)

f = coefficient of variation of lateral subgrade reaction (tons/ft^3)

z = depth (feet)

D = width/diameter of loaded area (feet)

Guidance for selection of f is given in Figure 9 for fine-grained and coarse-grained soils.

c. Heavily Overconsolidated Cohesive Soils. For heavily overconsolidated hard cohesive soils, the coefficient of lateral subgrade reaction can be assumed to be constant with depth. The methods presented in Chapter 4 can be used for the analysis; K_h varies between 35c and 70c (units of force/length^3) where c is the undrained shear strength.

d. Loading Conditions. Three principal loading conditions are illustrated with the design procedures in Figure 10, using the influence diagrams of Figure 11, 12 and 13 (all from Reference 31). Loading may be limited by allowable deflection of pile top or by pile stresses.

Case I. Pile with flexible cap or hinged end condition. Thrust and moment are applied at the top, which is free to rotate. Obtain total deflection, moment, and shear in the pile by algebraic sum of the effects of thrust and moment, given in Figure 11.

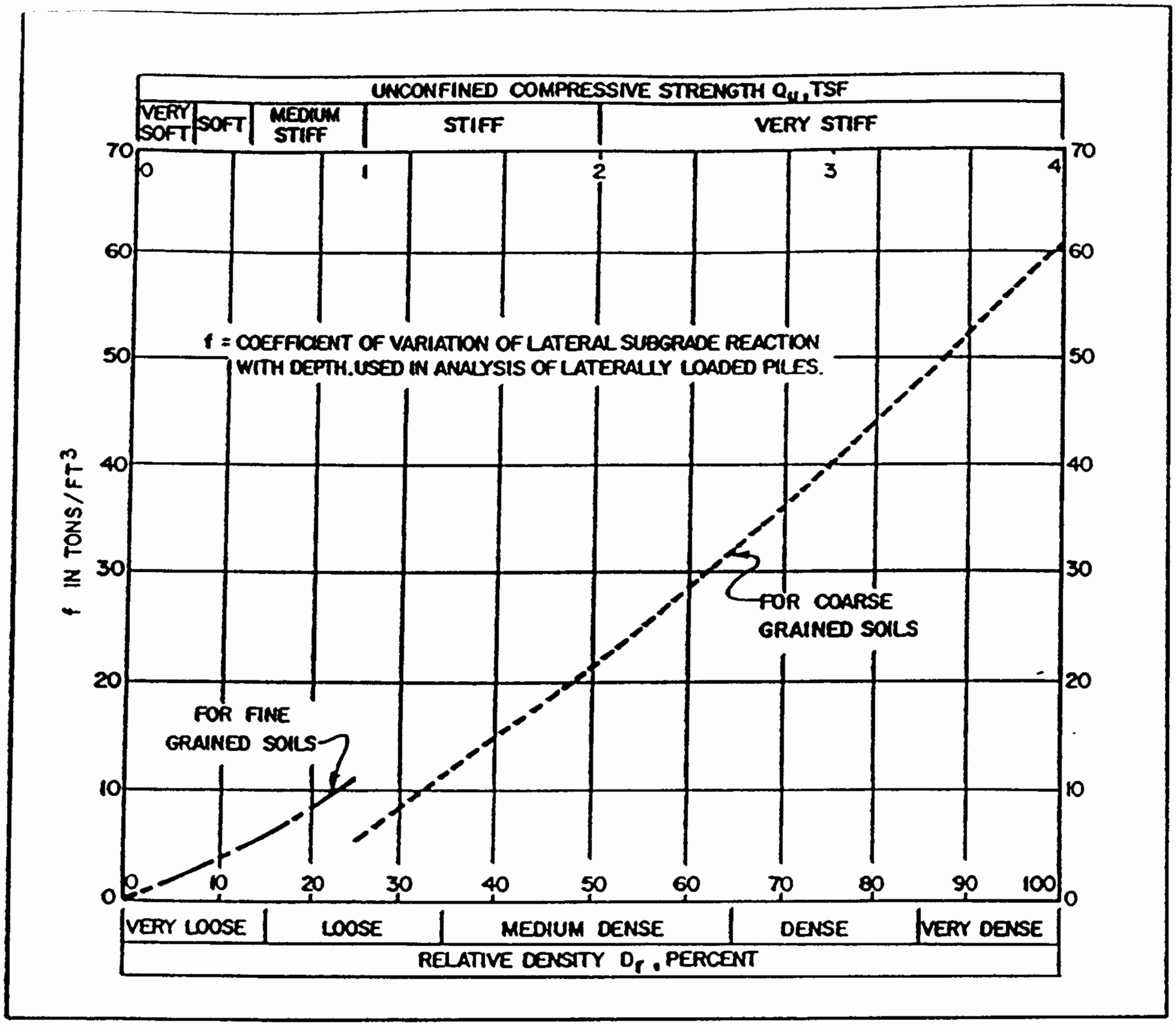
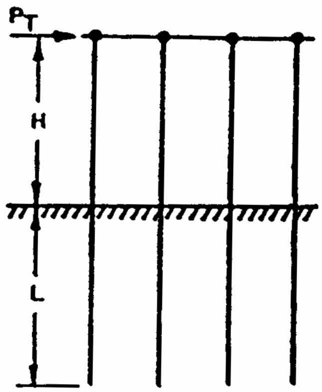
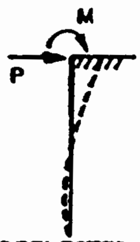
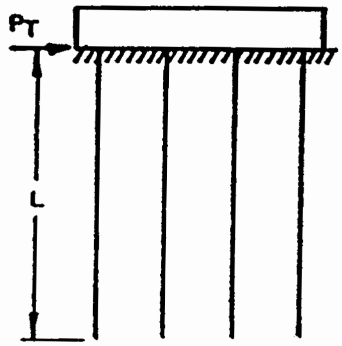


FIGURE 9
Coefficient of Variation of Subgrade Reaction

CASE I. FLEXIBLE CAP, ELEVATED POSITION		
CONDITION	LOAD AT GROUND LINE	DESIGN PROCEDURE
 n = NUMBER OF PILES	FOR EACH PILE: $P = \frac{P_T}{n}$ $M = PH$  DEFLECTED POSITION	FOR DEFINITION OF PARAMETERS SEE FIGURE 12 1. COMPUTE RELATIVE STIFFNESS FACTOR. $T = (\frac{EI}{f})^{1/5}$ 2. SELECT CURVE FOR PROPER $\frac{L}{T}$ IN FIGURE 11. 3. OBTAIN COEFFICIENTS F_δ, F_M, F_V AT DEPTHS DESIRED. 4. COMPUTE DEFLECTION, MOMENT AND SHEAR AT DESIRED DEPTHS USING FORMULAS OF FIGURE 11. NOTE: "f" VALUES FROM FIGURE 9 AND CONVERT TO LB/IN^3.

CASE II. PILES WITH RIGID CAP AT GROUND SURFACE		
		1. PROCEED AS IN STEP 1, CASE I. 2. COMPUTE DEFLECTION AND MOMENT AT DESIRED DEPTHS USING COEFFICIENTS F_δ, F_M AND FORMULAS OF FIGURE 12. 3. MAXIMUM SHEAR OCCURS AT TOP OF PILE AND EQUALS $P = \frac{P_T}{n}$ IN EACH PILE.

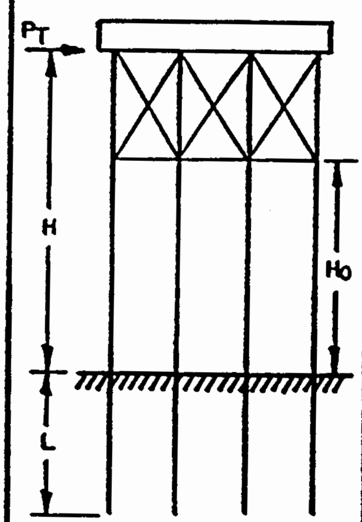
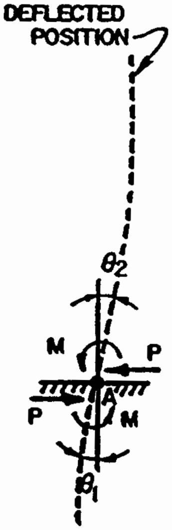
CASE III. RIGID CAP, ELEVATED POSITION		
	DEFLECTED POSITION 	1. ASSUME A HINGE AT POINT A WITH A BALANCING MOMENT M APPLIED AT POINT A. 2. COMPUTE SLOPE θ_2 ABOVE GROUND AS A FUNCTION OF M FROM CHARACTERISTICS OF SUPERSTRUCTURE. 3. COMPUTE SLOPE θ_1 FROM SLOPE COEFFICIENTS OF FIGURE 13 AS FOLLOWS: $\theta_1 = F_\theta (\frac{PT^2}{EI}) + F_\theta (\frac{MT}{EI})$ 4. EQUATE $\theta_1 = \theta_2$ AND SOLVE FOR VALUE OF M. 5. KNOWING VALUES OF P AND M, SOLVE FOR DEFLECTION, SHEAR, AND MOMENT AS IN CASE I. NOTE: IF GROUND SURFACE AT PILE LOCATION IS INCLINED, LOAD P TAKEN BY EACH PILE IS PROPORTIONAL TO I/H_0^3.

FIGURE 10
Design Procedure for Laterally Loaded Piles

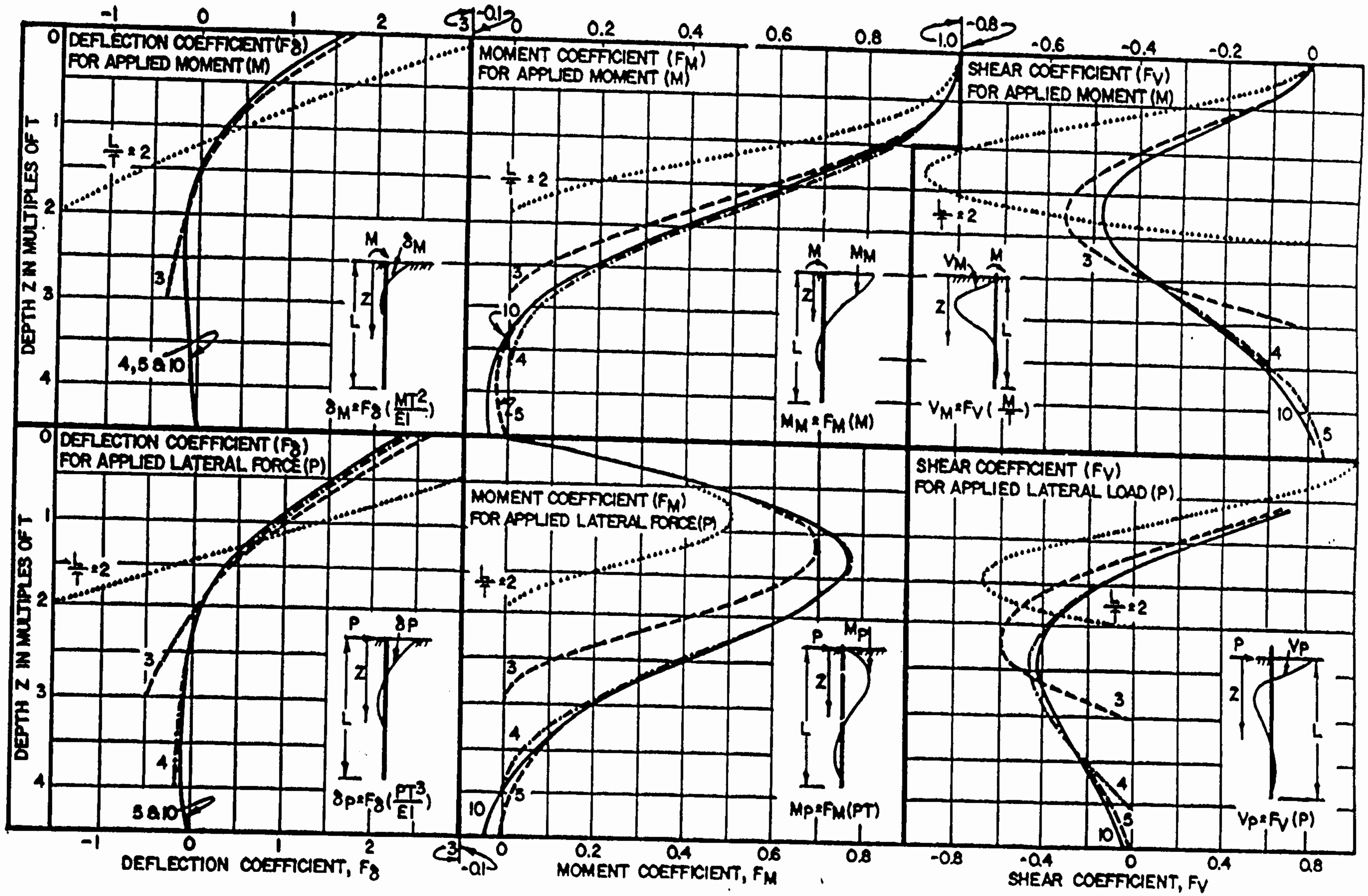


FIGURE 11
Influence Values for Pile with Applied Lateral Load and Moment
(Case I. Flexible Cap or Hinged End Condition)

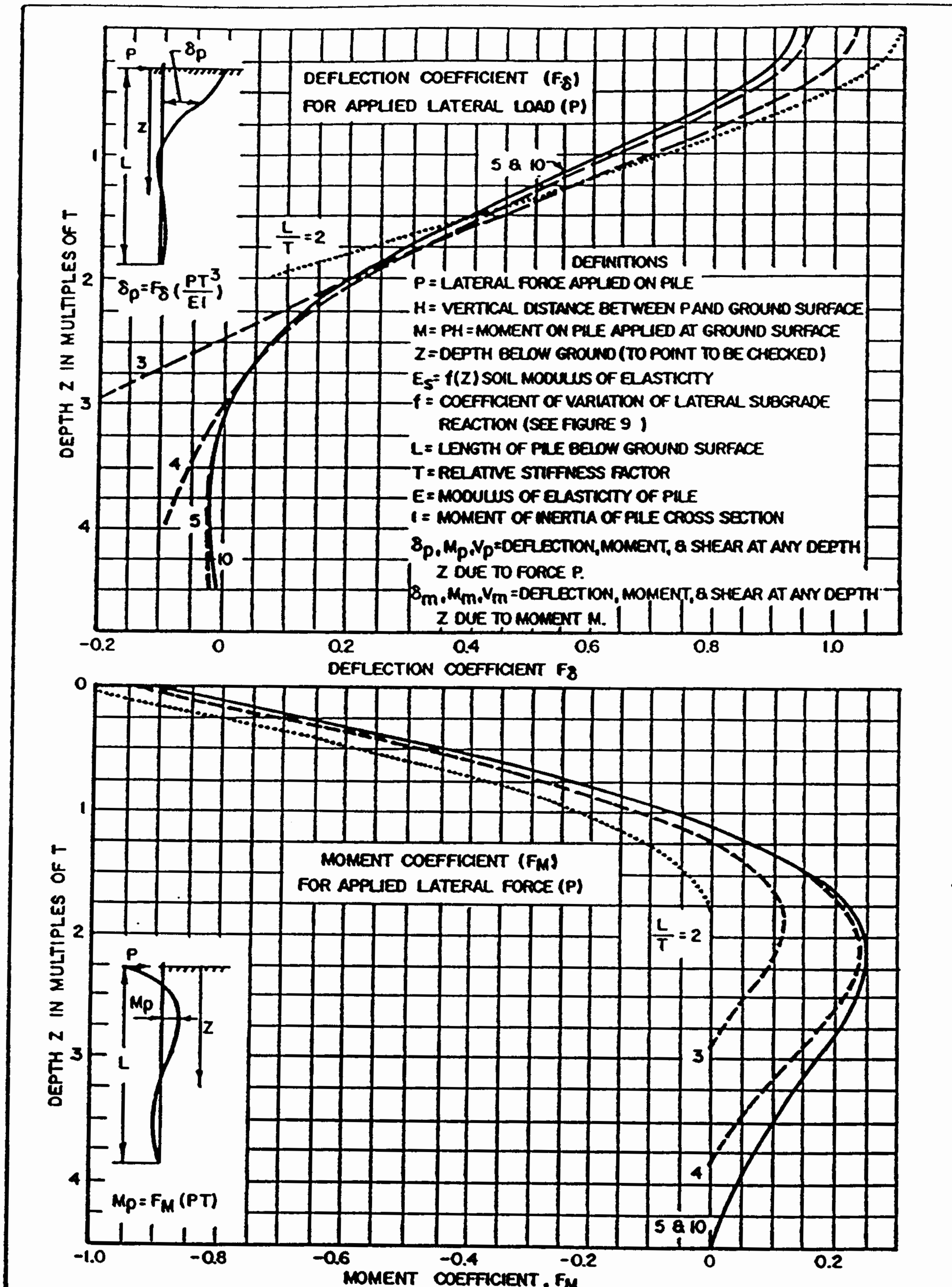


FIGURE 12
Influence Values for Laterally Loaded Pile
(Case II. Fixed Against Rotation at Ground Surface)

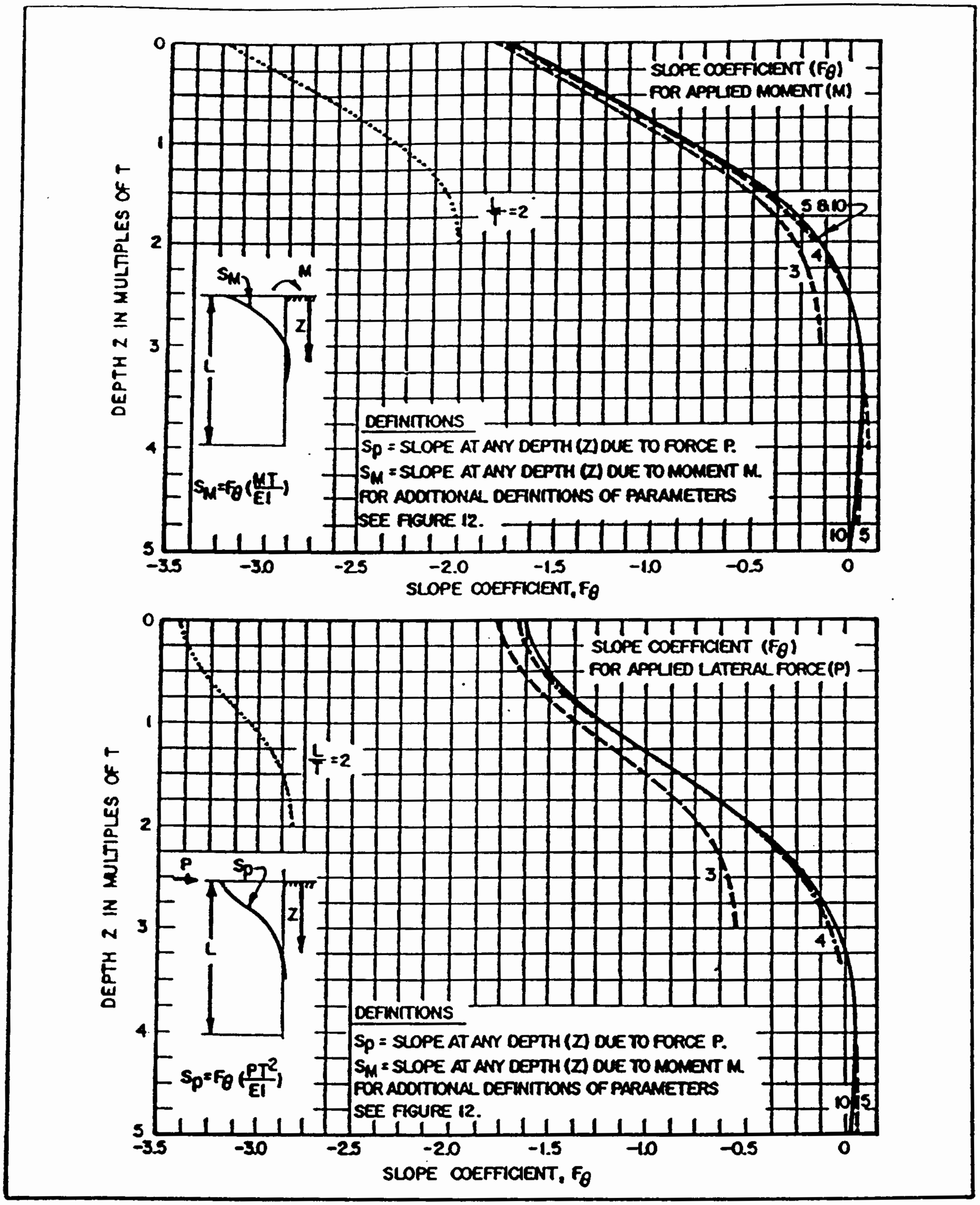


FIGURE 13
Slope Coefficient for Pile with Lateral Load or Moment

Case II. Pile with rigid cap fixed against rotation at ground surface. Thrust is applied at the top, which must maintain a vertical tangent. Obtain deflection and moment from influence values of Figure 12.

Case III. Pile with rigid cap above ground surface. Rotation of pile top depends on combined effect of superstructure and resistance below ground. Express rotation as a function of the influence values of Figure 13 and determine moment at pile top. Knowing thrust and moment applied at pile top, obtain total deflection, moment and shear in the pile by algebraic sum of the separate effects from Figure 11.

3. CYCLIC LOADS.

Lateral subgrade coefficient values decrease to about 25% the initial value due to cyclic loading for soft/loose soils and to about 50% the initial value for stiff/dense soils.

4. LONG-TERM LOADING. Long-term loading will increase pile deflection corresponding to a decrease in lateral subgrade reaction. To approximate this condition reduce the subgrade reaction values to 25% to 50% of their initial value for stiff clays, to 20% to 30% for soft clays, and to 80% to 90% for sands.

5. ULTIMATE LOAD CAPACITY - SINGLE PILES. A laterally loaded pile can fail by exceeding the strength of the surrounding soil or by exceeding the bending moment capacity of the pile resulting in a structural failure. Several methods are available for estimating the ultimate load capacity.

The method presented in Reference 33, Lateral Resistance of Piles in Cohesive Soils, by Broms, provides a simple procedure for estimating ultimate lateral capacity of piles.

6. GROUP ACTION. Group action should be considered when the pile spacing in the direction of loading is less than 6 to 8 pile diameters. Group action can be evaluated by reducing the effective coefficient of lateral subgrade reaction in the direction of loading by a reduction factor R (Reference 9) as follows:

Pile Spacing in Direction of Loading D = Pile Diameter	Subgrade Reaction Reduction Factor R
8D	1.00
6D	0.70
4D	0.40
3D	0.25

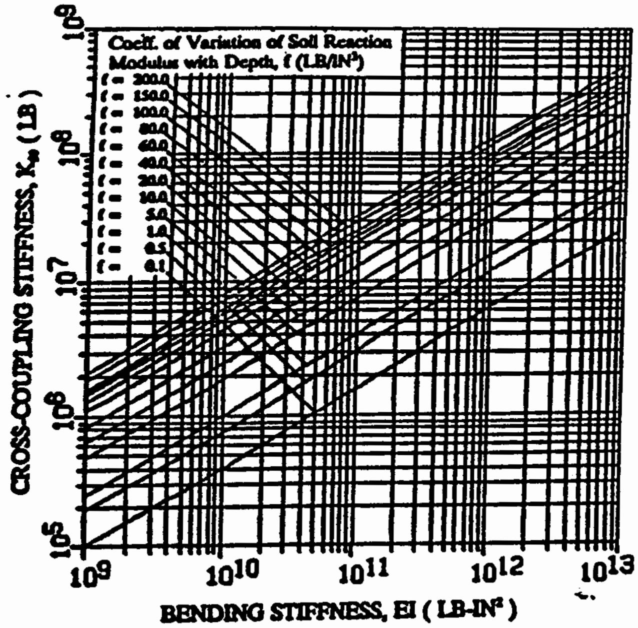


Figure 9. Pile Cross-Coupling Stiffness, $K_{\delta\theta}$

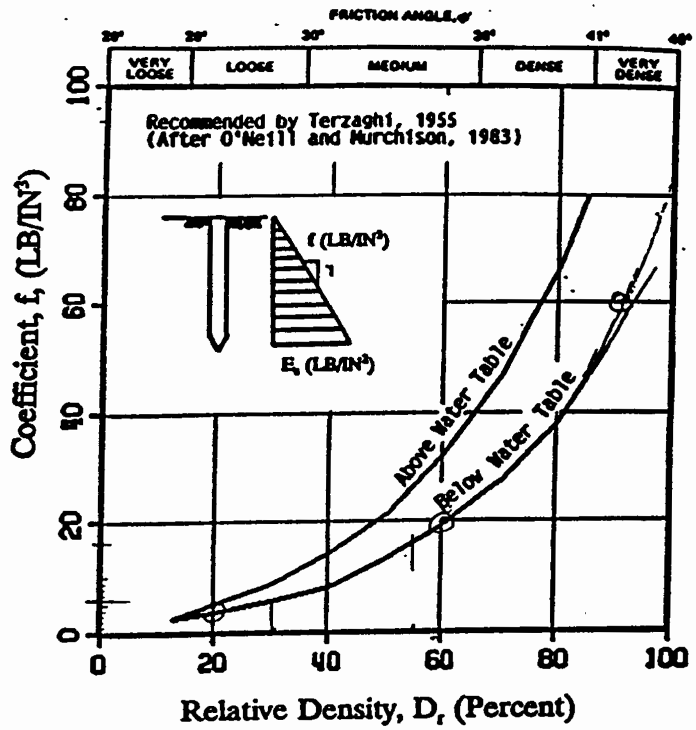


Figure 10. Recommendations for Coefficient f for Sands (Note: 1 LB/IN³ = 0.27 N/cm³)

he authors. This recommendation and results of the correlation for clay are shown in Figure 11. Only the upper five iameters of soils (soil type and ground ater) need to be considered in usage of the presented design charts.

Limitations of Approach. There are everal simplifying assumptions in the presented approach. The coefficient f is not an intrinsic soil parameter. The ecommendations for f presented in Figures) and 11 are appropriate for piles in typical highway bridge foundations (i.e. smaller piles). Furthermore, the embedment ffect has not been taken into account in he procedure. Therefore the recommendations are conservative and appropriate for shallow embedment conditions (say less than feet or 1.5 m).

Although correlations for the coefficient f can be conducted for other conditions e.g. larger piles and bigger embedment epths), the additional complexity negates he merits of the use of simplified linear elastic solutions. For such cases, comuter solutions, which can readily accomoate nonlinear effects and more general oundary conditions, are recommended.

Comparison to Caltrans Practice. The bove procedure can be compared to the ractice adopted by Caltrans. In Caltrans

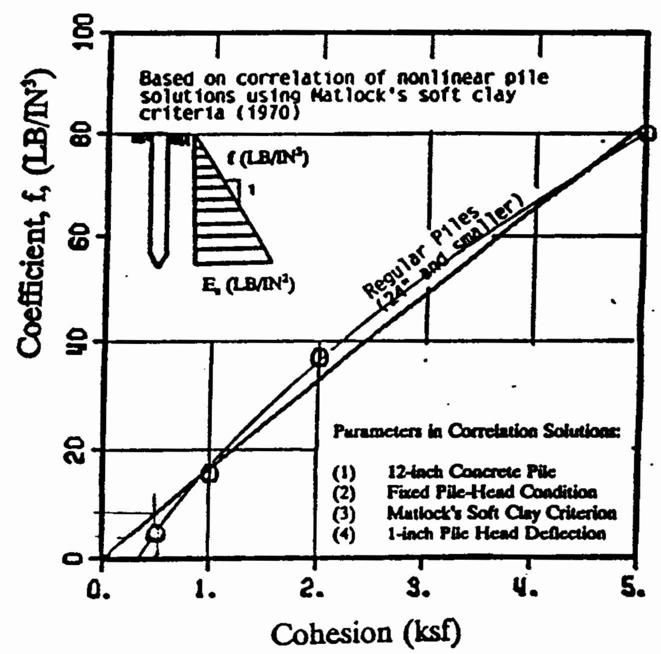


Figure 11. Recommendations of Coefficient f for Clays (Note: 1 LB/IN³ = 0.27 N/cm³)

ird Bridge Engineering Conference, Denver, Colorado, March 10-13, 1991
or more information, contact Earth Mechanics, Inc., Fountain Valley, CA
714) 848-9204