APPENDIX E

REPORT OF PRELIMINARY GEOTECHNICAL EVALUATION, GEOTECHNICAL INVESTIGATION, PRELIMINARY GEOTECHNICAL REPORT, & SOIL CORROSIVIY EVALUATION

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REPORT OF PRELIMINARY GEOTECHNICAL INVESTIGATION

REPORT OF PRELIMINARY GEOTECHNICAL INVESTIGATION PROPOSED BELMONT PLAZA OLYMPIC POOL REVITALIZATION PROJECT

4000 EAST OLYMPIC PLAZA
LONG BEACH, CALIFORNIA

Prepared for:

MDM ARCHITECTS, LLP

Pasadena, California

April 14, 2009

MACTEC Project 4953-09-0301

MACTEC

MACTEC

engineering and constructing a better tomorrow

April 14, 2009

Mr. Richard Dell
MDM Architects, LLP
201 South Lake Avenue, Suite 413
Pasadena, California 91101

Subject: **LETTER OF TRANSMITTAL**
Report of Preliminary Geotechnical Investigation
Proposed Belmont Plaza Olympic Pool Revitalization Project
4000 East Olympic Plaza
Long Beach, California
MACTEC Project 4953-09-0301

Dear Mr. Dell:

We are pleased to submit the results of our preliminary geotechnical investigation for the proposed Belmont Plaza Olympic Pool Revitalization project in Long Beach, California. This investigation was conducted in general accordance with our proposal February 26, 2009, as it was incorporated into the Agreement between Architect (Maple Dell + McClelland Architects, LLP) and Consultant (MACTEC Engineering and Consulting, Inc.) dated October 22, 2008. The Architect's agreement (the Prime Agreement) dated October 21, 2008, with the City of Long Beach provides professional services that included geotechnical services for the Belmont Plaza Olympic Pool Revitalization project. Our services were provided in accordance the terms and conditions contained in those agreements.

The scope of our services was planned with Mr. Marc Hauck of Maple Dell + McClelland Architects, LLP. Mr. Hauck provided us with information regarding the structural features of the existing structures. We have discussed the project with Mr. Jaime Garza of Miyamoto International, Inc., structural engineers for the project.

The results of our investigation and design recommendations are presented in this report. Please note that our report only contains information for use in evaluating the existing structures at the site and for planning development and preliminary design for replacement structures.

MACTEC Engineering and Consulting, Inc.
5628 East Slauson • Los Angeles, CA 90040-1554 • Phone: 323.889.5300 • 323.889-5398

It has been a pleasure to be of professional service to you. Please call if you have any questions or if we can be of further assistance.

Sincerely,

MACTEC Engineering and Consulting, Inc.

Boris O. Korin
Senior Engineer

Rosalind Munro
Senior Geologist

Marshall Lew, Ph.D.
Senior Principal Engineer
Vice President

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(7 copies submitted)

REPORT OF PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED BELMONT PLAZA
OLYMPIC POOL REVITALIZATION PROJECT

4000 EAST OLYMPIC PLAZA
LONG BEACH, CALIFORNIA

Prepared for:

MDM ARCHITECTS, AIA

Long Beach, California

MACTEC Engineering and Consulting, Inc.

Los Angeles, California

April 14, 2009

Project 4953-09-0301

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EXECUTIVE SUMMARY

We have completed our geotechnical investigation of the site of the existing Belmont Plaza Olympic Pool Complex in Long Beach, California for Maple, Dell + McClelland Architects, LLP on behalf of the City of Long Beach. Our results of our subsurface explorations, engineering analyses, and foundation design recommendations are summarized below.

In 1966, under the name of a MACTEC legacy firm, LeRoy Crandall and Associates, Inc., we had performed a foundation investigation for the existing complex. To supplement our previous investigation, we performed a geotechnical investigation that included additional explorations, laboratory testing, and engineering analyses. The current investigation included drilling two borings at the site and laboratory testing of the soil samples obtained. To supplement the borings, five cone penetration test (CPT) soundings were performed. The geotechnical recommendations in this report were developed using information from our current and our previous investigation.

The site is underlain by artificial fill placed for the existing development. Fill consisting of silty sand was encountered in one of our borings. The composition and thickness of the fill may vary across the site. To our knowledge, the fill was not observed or tested during placement and should be considered to be uncertified fill. The fill is underlain by beach and estuary deposits consisting of poorly graded sand with silty sand, sandy silt, and silty clay. Ground water was encountered in our borings at depths of 6½ to 9 feet (Elevation +½ to +4 feet). Approximately the upper 20 feet of the soils consist of loose to medium dense sandy soils that are susceptible to liquefaction. The potential seismically induced settlement of the on-site soils ranges from approximately 1 to 3 inches.

Plans provided to us show that the structure was proposed to be supported on timber piles. Verification of the foundation type and condition would require an invasive and destructive investigation and was not within the scope of our investigation. The capacity of the existing piles is sufficient for the design static loads but not for additional seismic loads. Furthermore, in the event of liquefaction occurring at the site, the capacity of the piles would no longer be sufficient to support even the static design loads. This would be expected to result in appreciable settlement of the building(s), probably with permanent damage to the foundations and structure. There is also a potential for several feet of lateral spreading that could damage the foundations and structure.

Renovation of the existing building(s)should include new piling to replace the existing piles. Mini-piles are expected to be the most feasible type of piling for installation within the existing buildings. The new piling would be to provide vertical and lateral capacity for the foundations. In addition to the new piling, the potential lateral spreading should be mitigated. Mitigation for lateral spreading could consist of ground improvement between the existing buildings and the ocean (as close as possible to the buildings). The ground improvement should wrap around the existing buildings as much as possible. Ground improvement of the on-site soils can be accomplished using Vibro-replacement or deep soil mixing.

If the buildings are to be replaced, ground improvement to mitigate the liquefaction potential, liquefaction-induced settlement, and lateral spreading potential should be performed. The ground improvement can be accomplished using Vibro-replacement or deep soil mixing. The ground improvement should be sufficient to allow support of the replacement buildings, and swimming pool(s) on shallow foundations (spread footings).

1.0 SCOPE

This report provides the results of our geotechnical investigation for the proposed Belmont Plaza Olympic Pool Revitalization project in the City of Long Beach, California. The locations of the existing buildings and our exploration borings are shown on Figure 1, Plot Plan. The results of our current field explorations and laboratory tests are presented in Appendix A with the results of cone penetration test (CPT) soundings and soil corrosivity testing presented in Appendices B and C, respectively.

We previously performed a foundation investigation for the subject site and existing development and submitted the results in a report dated August 15, 1966 (the report was issued under the name of a MACTEC legacy company, LeRoy Crandall & Associates, Job No. A-66102). A copy of our prior report is presented in Appendix D. The locations of our previous exploration borings are shown on Figure 1. We have reviewed and accept the field and laboratory test data presented in that report. However, with advances in knowledge of the behavior of soils since the report was issued, many of the recommendations contained in the report are no longer considered to be applicable.

This investigation was authorized to perform subsurface explorations, laboratory testing, and geologic and engineering analyses to assess the geologic-seismic hazards that might affect the site. We were also to provide preliminary geotechnical recommendations for use in evaluating the existing development and to provide information for use in planning and preliminary design for revitalization of the existing development or for replacement development. Our services were to consist of the following main tasks:

- Review of prior data at the site that has been provided to us.
- Subsurface explorations to determine the nature and stratigraphy of the subsurface soils, and to obtain undisturbed and bulk samples for laboratory observation and testing.
- A geologic-seismic hazards evaluation including an evaluation of liquefaction, slope instability and surface rupture due to faulting or lateral spreading.
- Laboratory testing of soil samples for determination of the static physical soil properties.

- Corrosion studies to determine the presence of potentially corrosive soils.
- Engineering evaluation of the geotechnical data to develop preliminary recommendations for use in planning and preliminary design of the proposed revitalization or for a replacement development.

Our investigation was not intended to be sufficient to provide final geotechnical design recommendations for use in design of structures at the site. A comprehensive investigation should be performed to develop final recommendations once the structural features of the future work have been established. Depending on the features of the future work, additional field and laboratory testing may or may not be required.

Our recommendations are based on the results of our current and previous field explorations, laboratory tests, and appropriate engineering analyses. The results of the current field explorations and laboratory tests, which form the basis of our recommendations, are presented in Appendices A, B, and C.

2.0 PROJECT CONSIDERATIONS

The Belmont Plaza Pool facility site is bounded on the north by East Olympic Plaza, Termino Avenue (and its extension) on the west, Bennett Avenue on the east, and the beach (Pacific Ocean) on the south. The northern (landward side) portion of the site is occupied by a park; the park is not a part of the project. The site grades were raised by several feet for the existing development by the placement of fill. There is a retaining wall up to 3 feet in height between the walk surrounding the facility and the beach.

The existing building complex is roughly 440 feet by 220 feet in plan. Based on plans provided to us, the existing building is supported on timber piles with tip diameters of at least 8 inches and minimum lengths of 32 feet. The plans show the pile caps at various elevations. The pool walls and diving towers are believed to be supported on shallow foundations. The structural features and details are being evaluated by Miyamoto International, Inc. (2009).

It is proposed to evaluate the existing structure and foundations of the existing Belmont Plaza Pool and to develop plans and recommendations for revitalization or replacement of the facility .

3.0 EXPLORATIONS AND TESTING

The soil conditions beneath the site were explored by drilling two borings to depths of about 76½ feet below the existing grade at the locations shown on Figure 1. Details of the current explorations and the logs of the borings are presented in the Appendix A.

Laboratory tests were performed on selected samples obtained from the current borings to aid in the classification of the soils and to determine the pertinent engineering properties of the foundation soils. The following tests were performed:

- Moisture content and dry density determinations.
- Direct shear.
- Consolidation.
- Sieve analyses.
- Atterberg limits.

All testing was done in general accordance with applicable ASTM specifications. Details of the laboratory testing program and test results are presented in the Appendix A.

To supplement the data from the borings and laboratory tests, we retained Kehoe Testing & Engineering (Kehoe) to perform Cone Penetration Test (CPT) soundings. The soundings were performed at five locations selected by us to depths of approximately 60 feet each. Two of the soundings were performed near the borings to provide correlation data. Shear wave measurements were performed in one of the soundings (CPT-3). The results of Kehoe's testing are presented in Appendix B.

Soil Corrosivity tests were performed on two samples of the upper on-site soils by Schiff Associates (Schiff). The results of the soil corrosivity study are presented in Appendix C.

Data were also available from our previous investigation for the subject site (our Job No. A-66102). Our report for the previous investigation, including details of the prior explorations and results of laboratory testing, is presented in Appendix D.

4.0 GEOLOGY

The site is located in the northern portion of the Peninsular Ranges geomorphic province. This province extends northwesterly from Baja California into the Los Angeles Basin and westerly into the offshore area, including Santa Catalina, Santa Barbara, San Clemente and San Nicolas islands. The northern boundary of the province is the Transverse Ranges along the Malibu Coast, Santa Monica, Hollywood, Raymond, Sierra Madre, and Cucamonga faults. The eastern boundary of the province is the Colorado Desert geomorphic province along the San Jacinto fault system. The Peninsular Ranges province is characterized by northwest/southeast trending alignments of mountains and hills and intervening basins, reflecting the influence of northwest trending major faults and folds, such as the nearby Newport Inglewood fault zone, controlling the general geologic structural fabric of the region.

Most of the site is underlain by artificial fill. Fill consisting of silty sand was encountered in one of our borings. The composition of the fill may vary across the site. While correspondence in our files indicates that there was intent to compact the fill, we have no records that the fill was actually compacted.

The fill is underlain by beach and estuary deposits consisting of poorly graded sand with silty sand, sandy silt, and silty clay. The general geology of the area is shown on Figure 2, Regional Geology.

Ground water was encountered in our borings (current and previous) at depths of 5 to 9 feet below the existing grade (Elevation +½ to +4 feet).

5.0 LIMITED GEOLOGIC-SEISMIC HAZARDS EVALUATION

Based on the available geologic data, active or potentially active faults with the potential for surface fault rupture are not known to be located beneath or projecting towards the site. The closest active fault to the site is the Newport Inglewood fault zone located approximately 1.5 miles to the northeast. The Palos Verdes fault is located approximately 7 miles to the southwest. The site is not in an Alquist-Priolo Earthquake Fault Zone. In our opinion, the potential for surface rupture at the site due to fault plane displacement propagating to the ground surface during the design life of the project is considered low.

Figure 3 shows the location of the site in relation to active faults and significant historic earthquakes in the region. Although the site could be subjected to strong ground shaking in the event of an earthquake, this hazard is common in Southern California and the effects of ground shaking on structures can be mitigated by proper engineering design and construction in conformance with current building codes and engineering practices.

The site is along the coastline and is susceptible to damage from a tsunami. Government agencies are currently upgrading the region's tsunami preparedness, warning, and evacuation systems. The site is in a FEMA Special Flood Hazard Area subject to inundation by the 1% annual chance flood. According to the City of Long Beach Safety Element of the General Plan, the site is not located downslope of any large bodies of water that could adversely affect the site in the event of earthquake-induced dam failures or seiches (wave oscillations in an enclosed or semi-enclosed body of water).

The site is located between the Seal Beach and Wilmington oil fields. There are no known oil wells on the site. The site is along the margins of the area impacted by ground subsidence due to oil extraction in the Wilmington oil field. Water injection was begun in 1958 to repressurize the oil field and the area has been stabilized.

The site is relatively level and the absence of nearby slopes precludes any slope stability hazards. The site is not in a state of California Earthquake-Induced Landslide Hazard Zone (California Division of Mines and Geology, CDMG, 1999).

6.0 LIQUEFACTION EVALUATION

Liquefaction potential is greatest where the ground water level is shallow, and submerged loose, fine sands occur within a depth of about 15 meters (50 feet) or less. Liquefaction potential decreases as grain size and clay and gravel content increase. As ground acceleration and shaking duration increase during an earthquake, liquefaction potential increases. The site is within a state of California designated Liquefaction Hazard Zone (CDMG, 1999).

To evaluate the site-specific liquefaction potential, we computed the peak ground acceleration (PGA) for the maximum credible earthquake ground motion with a 2% probability of being exceeded in 50 years using EZ-FRISK, Version 7.32. This ground motion, which has a return period of 2,475 years, was adjusted to be compatible with a Magnitude 7.5 earthquake. The next generation ground motion attenuation relationships (NGA) of Abrahamson & Silva (2008), Boore & Atkinson (2008), Campbell & Bozorgnia (2008) and Chiou & Young, (2008) were used, with equal weight, in the analysis. Based on the shear wave velocity measurements in CPT-3, a shear wave velocity of 267 meters per second was used for the upper 30 meters. The depth which the shear wave velocity is at least 1,000 meters per second was assumed to be 2 kilometers while the depth at which the velocity is at least 2,500 meters per second was assumed to be 4 kilometers. The details of the CPT soundings and shear wave velocity measurements are presented in Appendix B.

We used a PGA for our liquefaction analyses that is two-thirds of the Magnitude-7.5 compatible PGA for equivalence with the design level earthquake as defined in the 2007 California Building Code and ASCE 7-05, and as referenced in California Geological Survey Note 48 dated October 2007. The Magnitude 7.5-compatible PGA computed in this manner for the subject site is 0.40g. (The Magnitude-7.5 compatible PGA is not the same as the one used in the evaluation of structures. This latter PGA is 0.49g).

The liquefaction potential of the soils underlying the site was evaluated using the Magnitude-7.5 compatible PGA, as described above and the results of our current explorations at the project site. The ground-water level for our liquefaction analysis was assumed to be 7 feet below the existing grade based on our measurements of the ground water level; the historical high ground-water level has not been established for the site. The liquefaction potential was computed according to procedures described in Youd et al. (2001).

Seismically-induced settlement is often caused by loose to medium-dense granular soils densified during ground shaking. Dry and partially saturated soils as well as saturated granular soils are subject to seismically-induced settlement. The medium dense granular soils encountered in our exploratory borings are considered to be susceptible to liquefaction and seismically-induced settlement. We evaluated the seismically-induced settlement based on the procedures outlined by Tokimatsu and Seed (1987) and Ishihara and Yoshimine (1992). We estimate the seismically-induced settlement to be about ¾ to 2¾ inches.

Some, but not all, liquefiable soils are susceptible to lateral spreading. Methods to calculate the extent and magnitude of lateral spreading are few and only provide a rough order of magnitude estimates of the amount of lateral spreading. Assuming that the soils between the site and the Pacific Ocean are similar to those beneath the site, several feet of lateral spreading towards the Pacific Ocean could occur in the event of design earthquake ground motions. The movement of the soils due to lateral spreading would not be expected to be uniform. Therefore, differential lateral spreading should be expected in the building area. We evaluated the lateral spreading potential based on the procedures outlined by Youd et al (2002).

7.0 RECOMMENDATIONS

7.1 GENERAL

The existing Belmont Plaza buildings, especially the pool building, are being considered for structural rehabilitation or replacement. The plans for the existing buildings indicate that they are supported on timber piles. While the foundation recommendations presented in our 1966 report remain applicable for static loading, the upper soils are subject to liquefaction in the event of design earthquake level ground motion at the site. Liquefaction of the on-site soils would result in significant reductions in the capacities of the existing foundations. In the event of strong ground motion at the site, settlement and damage to the existing structures' foundations and structural elements should be anticipated.

If renovation of the existing buildings is to be performed, piling is expected to be the most feasible means of replacing or strengthening the foundations. Because of expected caving in the granular soils below the relatively shallow ground water level beneath the site, the installation of conventional drilled cast-in-place concrete piling would be difficult. Since the installation of new piling would for the most part be performed within the existing structures, micro-piles or auger-cast piles are recommend. For preliminary planning purposes, 10-inch-diameter mini-piles or 24-inch-diameter auger cast piles should extend approximately 50 feet below the existing grade to develop a downward capacity of 160 kips (sufficient capacity to replace two of the existing timber piles). In addition to replacing and/or strengthening the foundations, some means of mitigating the potential lateral spreading should be used. This mitigation would most likely consist of ground improvement between the existing buildings and the Pacific Ocean; the improvements should be constructed as close to the buildings as possible.

If the buildings are to be replaced, the liquefaction induced settlement and lateral spreading should be mitigated by ground improvement of the site. The replacement buildings could then be supported on shallow foundations, most likely spread footings.

7.2 FOUNDATIONS

Existing Timber Piles

The existing building is reportedly supported on timber piles. The tip diameter of thee piles was to be 8 inches in diameter and the piles were to be a minimum of 32 feet long. Based on our previous report (1966) for the site, these piles have an allowable downward design capacity of 80 kips. As was usual at the time the analyses were performed, this capacity does not consider liquefaction. (The Niigata and Alaska earthquakes of 1964 were the start of liquefaction becoming a concern. The early versions of the current procedures to evaluate liquefaction were not published for use until the 1980s and only nominal peak ground accelerations were usually used in the analyses until the 1994 Northridge Earthquake.)

In the event of liquefaction affecting the upper 20 feet soils, as could potentially occur based on our current analyses, the existing piles would be overloaded with the ultimate downward axial capacity of the piles being temporarily less than the structural and downdrag forces imposed on the piles. The extent of the consequences are this is difficult to project since to a large degree they are influenced by the structure and will be locally variable. However, readily perceptible settlement of the structure, probably on the order of several inches, is expected with probable permanent damage to the timber piles and the structure. There would also be a loss of lateral capacity of the piles probably resulting in some lateral movement of the building columns. Slabs supported on grade supported will settle and voids may develop beneath the slabs.

New Foundations

Provided that ground improvement is performed at the site in accordance with the recommendations in the Grading section of this report, the replacement buildings may be supported on spread footings. Spread footings carried at least 1 foot into properly compacted fill and at least 2 feet below the lowest adjacent grade or floor level can be designed to impose a net dead-plus-live load pressure of 3,000 pounds per square foot. The excavations should be deepened as necessary to extend into satisfactory soils. A one-third increase can be used for wind or seismic loads. The recommended bearing value is a net value, and the weight of concrete in the footings can be taken as 50 pounds per cubic foot; the weight of soil backfill can be neglected when determining the downward loads.

Lateral loads can be resisted by soil friction and by the passive resistance of the soils. A coefficient of friction of 0.4 may be used between the footings and the floor slab and the supporting soils. The passive resistance of natural soils or properly compacted fill soils can be assumed to be equal to the pressure developed by a fluid with a density of 250 pounds per cubic foot. A one-third increase in the passive value can be used for wind or seismic loads. The frictional resistance and the passive resistance of the soils can be combined without reduction in determining the total lateral resistance.

7.3 SITE COEFFICIENT AND SEISMIC ZONATION

We have determined the seismic parameters in accordance with the Section 1613A of the 2007 edition of the California Building Code (2007 CBC) and Section 11.4 of ASCE 7-05 Standard (ASCE, 2005) using the United States Geological Survey (USGS, 2007) program, Earthquake Ground Motion Parameters, Version 5.0.8. The site location used was Latitude 33.7581° and Longitude -118.1456° with a Site Class "D." If the proposed buildings are to be designed using the provisions of the 2007 CBC, the seismic site coefficients may be taken as presented below:

Site Coefficient	Value
S_S (0.2 second period, Site Class B)	1.74g
S_1 (1.0 second period, Site Class B)	0.67g
Site Class	D
F_a	1.00
F_v	1.50
$S_{MS} = F_aS_S$ (0.2 second period, Site Class D)	1.74g
$S_{M1} = F_vS_1$ (1.0 second period, Site Class D)	1.00g
$S_{DS} = 2/3 \times S_{MS}$ (0.2 second period, Site Class D)	1.16g
$S_{D1} = 2/3 \times S_{M1}$ (1.0 second period, Site Class D)	0.67g

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7.4 FLOOR SLAB SUPPORT

For pile supported buildings, the floor slabs should be structurally supported. If the subgrade is prepared as recommended in a following section on grading and some distress in the on-grade concrete walks and slabs as a result of liquefaction in the event of strong ground motion is acceptable, the concrete walks and slabs adjacent to the buildings may be supported on grade.

If the upper soils are improved so that the buildings can be supported on spread footings, the floor slabs may be supported on grade on the improved soils. Construction activities and exposure to the environment can cause deterioration of the prepared subgrade. Therefore, we recommend that our

field representative observe the condition of the final subgrade soils immediately prior to slab-on-grade construction, and, if necessary, perform further density and moisture content tests to determine the suitability of the final prepared subgrade.

In areas where vinyl or other moisture-sensitive floor covering is planned, we recommend that the floor slab in those areas be underlain by a vapor retarder/barrier consisting of a vapor-retarding membrane. If the interior of the structure directly above the slab-on-grade is a humidity controlled area, then the same recommendations apply. The membrane should conform to the requirements of ASTM E 1745, " Standard Specifications for Water Vapor Retarders in Contact with Soil or Granular Fill under Concrete Slabs." The installation of the membrane applies to both the structurally supported floor slabs and slabs supported on grade.

In addition, measures will be required to prevent slab curling due to uneven curing between the top and the bottom of the slab. These measures should consist of one or more of the following:

- Reduced joint spacing
- A low shrinkage concrete mix design such as a low water/cement ratio mix or water-reducing admixtures
- Use of a 2-inch thick "blotter" layer

If a blotter layer is used, it should consist of trimmable, free-draining granular fill between the membrane and the slab. The blotter layer fill should have a Sand Equivalent of 30. The blotter layer should be placed with a moisture content of less than 5% percent. Note that if a blotter layer is used, then the layer should not be allowed to become wet (due to rain, wet-curing, wet-grinding or cutting, and cleaning where water enters prior to slab placement or after slab placement through openings such as column block-outs). Also, the blotter layer should be cut off from sources of water (for instance, the blotter layer should not be continuous to the exterior of the building).

Care should be taken to prevent any tears or discontinuities in the membrane. The membrane should be inspected prior to placement of the slab or installation of a blotter layer and any holes or discontinuities (e.g., around penetrations) properly sealed.

Where vinyl or other moisture sensitive floor covering or storage of moisture sensitive materials is not planned, the floor slab and other concrete slabs may be supported directly on the final prepared subgrade.

7.5 RETAINING WALLS AND WALLS BELOW GRADE

Lateral Earth Pressure

For design of cantilevered retaining walls, where the surface of the backfill is level, it may be assumed that drained soils will exert a lateral pressure (active earth pressure) equal to that developed by a fluid with a density of 30 pounds per cubic foot. In addition to the recommended earth pressure, the walls should be designed to resist any applicable surcharges due to storage or traffic loads. If the soils are not drained, they should also resist hydrostatic pressures.

For the design of braced basement walls, it may be assumed that drained soils will exert a lateral pressure (at-rest earth pressure) equal to that developed by a fluid with a density of 52 pounds per cubic foot. In addition to the recommended earth pressure, the walls should be designed to resist any applicable surcharges due to foundation, storage, or traffic loads. If the soils are not drained, they should also resist hydrostatic pressures.

In addition to the recommended earth pressure, retaining walls adjacent to areas subject to vehicular traffic should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pounds per square foot surcharge behind the walls due to normal vehicular traffic. If the traffic is kept back at least 10 feet from the walls, the traffic surcharge may be neglected.

Seismic Lateral Earth Pressure

In addition to the above-mentioned lateral earth pressures, basement walls with more than 6 feet of unbalanced earth (where the difference in height of retained soil from one side of the basement to the other is greater than 6 feet) and cantilever retaining walls greater than 12 feet in height should be designed to support a seismic active pressure. The seismic active pressure for use in design should be applied uniformly to the back of the wall and should taken as being 5H pounds per square foot, where H is the height of the retained soils in feet or the difference in height of the retained soils between the opposing basement walls.

Drainage

Walls, or at least the portions of walls, that extend below Elevation +8 feet should be designed to resist hydrostatic pressure in addition to the lateral earth, seismic, traffic, and other surcharge pressures. The portions of walls not designed to resist hydrostatic pressures should be provided with a drainage system. However, walls that are provided with a full height drainage system and use weepholes at the base of the wall (such as retaining walls that are separate from the buildings) for removal of the water do not need to be designed to resist hydrostatic pressure even if they extend below Elevation +8. For design, the hydrostatic pressure may be taken as being 50 pounds per cubic foot (this pressure considers buoyancy of the soils and the unit weight of salt water).

Walls below grade that are not designed to resist hydrostatic pressures should be provided with a drainage system placed behind the walls below grade to help dissipate the hydrostatic forces that may develop behind the walls. The drainage system may consist of a 4-inch-diameter perforated pipe placed with the perforations down and surrounded by at least 4 inches of granular filter gravel. The pipe should be sloped at least 2 inches in 100 feet. The granular filter material should be separated from the adjacent soils by a filter fabric. The perforated pipes should be placed at the bases of the walls below grade. In addition, a 1-foot wide zone of granular filter material, or continuous Miradrain collector panels, should be placed behind each wall. The strip of granular filter material or the Miradrain (Miradrain 6000 or equivalent) panels should extend down to the drainage system, and should be terminated at 2 feet below the ground surface.

The installed drainage system should be observed by personnel from our firm prior to being backfilled. Inspection of the drainage system may also be required by the reviewing governmental agencies.

It should be realized that a permit from the State of California Regional Water Quality Control Board would have to be obtained to discharge the water from the drainage system into the storm drain. To obtain such a permit, chemical tests will have to be performed on water samples obtained at the site to verify that chemicals or pollutants within the water do not exceed the allowable limits for discharging into the storm drain.

7.6 GRADING

Site Improvement

If the upper potentially liquefiable soils are sufficiently improved, the potential liquefaction hazard at the site would be mitigated and the replacement buildings could be supported on spread footings, unless other considerations require the use of piling. If the existing buildings and/or pool are proposed to remain, ground improvement probably will not be a feasible alternative to piling due to possibility of damaging the buildings and/or pool.

We expect that a ground improvement procedure such as stone columns or Vibro-Replacement would provide the best outcome for the project site. These procedures consist of densifying the on-site soils and the addition coarse grained materials. The presence of silts and clays in the upper soils excludes the possibility of using techniques that would just densify the on-site soils. The ground improvement should extend at least 25 feet below the existing surrounding grade to approximately Elevation -18 feet (18 feet below sea level). The ground improvement procedure is performed by a specialty contractor and such a contractor should be consulted early on in the planning process to aid in determining if proceeding with ground improvement alternative is economically desirable. We would be pleased to develop recommendations for planning and verification of sufficient improvement if it is decided to proceed with ground improvement.

After the on-site soils are improved, the surface of the ground is expect to be lower and the upper materials will be disturbed. The disturbed materials should be excavated as recommended in the following section on Site Preparation and Compaction.

Site Preparation and Compaction

To provide support for at-grade concrete walks and slabs adjacent to the new buildings, all the existing uncertified fill (those fills for which a record of compaction during placement is unavailable) should be excavated. To our knowledge, the existing fill soils at the site were not observed and tested during placement. Further excavation should be performed to remove disturbed natural soils within the construction area and for at least 2 feet beyond any proposed paving and 5 feet beyond any proposed footings in plan. Where there is insufficient room for the recommended overexcavation, we can provide case specific recommendations. The excavated soils should be replaced as properly compacted fill. All planned additional fill should be properly compacted.

Where excavations are deeper than about 4 feet, the sides of the excavations should be sloped back at 1:1 (horizontal to vertical) or shored for safety. Adjacent to an existing building, the excavations should not extend below a plane drawn downward at 1½: (horizontal to vertical) from the bottoms of the exterior footings (pile caps and/or grade beams) of the existing buildings. If the existing pool is to remain, the excavation should not extend below 1½:1 1 (horizontal to vertical) plane extending downward from the top edge of the pool.

All applicable requirements of the 2009 State of California Construction and General Industry Safety Orders, the Occupational Safety and Health Act of 1970, and the Construction Safety Act should be met.

After excavating the upper soils as recommended, the exposed natural soils should be carefully observed for the removal of all unsuitable deposits. Next, the exposed soils should be rolled with heavy compaction equipment. The upper 6 inches of exposed soils should be compacted to at least 90% of the maximum density obtainable by the ASTM Designation D1557-07 method of compaction. For soils with less than 5% of the particles by weight passing the No. 200 sieve, the soils should be compacted to at least 95%.

After compacting the exposed soils, all required fill should be placed in loose lifts not more than 8 inches thick and compacted to at least 90% (95% if less than 5% of the particles pass the No. 200 sieve). The moisture content of the on-site granular soils at the time of compaction should vary from zero to no more than 4% above optimum moisture content. The moisture content of any on-site cohesive soils at the time of compaction should be brought to about 4% over optimum moisture content.

Material for Fill

The on-site soils, less any debris or organic matter, may be used in required fills. The on-site clayey soils should not be placed with 2 feet of proposed floor slabs, pool decks, or other portland cement concrete paved areas. Any required imported material should consist of relatively non-expansive soils with an Expansion Index of less than 35. The imported materials should contain sufficient fines (binder material) so as to be relatively impermeable and result in a stable subgrade

when compacted. All proposed import materials should be approved by our personnel prior to being placed at the site.

7.7 GEOTECHNICAL OBSERVATION

The reworking of the upper soils and the compaction of all required fill should be observed and tested during placement by a representative of our firm. This representative should perform at least the following duties:

- Observe the clearing and grubbing operations for proper removal of all unsuitable materials.
- Observe ground improvement procedures if they are used.
- Observe the exposed subgrade in areas to receive fill and in areas where excavation has resulted in the desired finished subgrade. The representative should also observe proofrolling and delineation of areas requiring overexcavation.
- Evaluate the suitability of on-site and import soils for fill placement; collect and submit soil samples for required or recommended laboratory testing where necessary.
- Observe the fill and backfill for uniformity during placement.
- Test backfill for field density and compaction to determine the percentage of compaction achieved during backfill placement.
- Observe and probe foundation materials to confirm that suitable bearing materials are present at the design foundation depths.
- Observe the installation of piling and any testing of the piling that is required.

The governmental agencies having jurisdiction over the project should be notified prior to commencement of grading so that the necessary grading permits can be obtained and arrangements can be made for required inspection(s). The contractor should be familiar with the inspection requirements of the reviewing agencies.

8.0 GENERAL LIMITATIONS AND BASIS FOR RECOMMENDATIONS

Our professional services have been performed using that degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants practicing in this or similar localities. No other warranty, expressed or implied, is made as to the professional advice included in this report. This report has been prepared for MDM Architects, LLP, their client, the City of Long Beach, and their design consultants to be used solely in the evaluation, planning and design of the proposed Belmont Plaza Olympic Pool Revitalization Project. The report has not been prepared for use by other parties, and may not contain sufficient information for purpose of other parties or other uses.

The recommendations provided in this report are based upon our understanding of the described project information and on our interpretation of the data collected during our current and previous subsurface explorations. We have made our recommendations based upon experience with similar subsurface conditions under similar loading conditions. The recommendations apply to the specific project discussed in this report; therefore, any change in the structure configuration, loads, location, or the site grades should be provided to us so that we can review our conclusions and recommendations and make any necessary modifications.

The recommendations provided in this report are also based upon the assumption that the necessary geotechnical observations and testing during construction will be performed by representatives of our firm. The field observation services are considered a continuation of the geotechnical investigation and essential to verify that the actual soil conditions are as expected. This also provides for the procedure whereby the client can be advised of unexpected or changed conditions that would require modifications of our original recommendations. If another firm is retained for the geotechnical observation services, our professional responsibility and liability would be limited to the extent that we would not be the geotechnical engineer of record.

9.0 REFERENCES

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FIGURES

Explanation

Boring 2 — Current Boring
3 — Previous Boring (Job No. A-66102)
CPT-5 — Current CPT Sounding
Location & Number

0 30' 60' 120'
SCALE: 1" = 60'-0"

N

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PHONE (323) 889-5300 FAX (323) 889-5398

JOB:	4953-09-0301
DATE:	4/03/09
SCALE:	1' = 60'-0"
DRAWN:	L. Morley
REVD:	
CHKD:	B. Korin
DATE:	3/31/09

Belmont Plaza Olympic Pool Revitalization
4000 East Olympic Plaza, Long Beach, California 90803

PLOT PLAN

FIGURE NO. 1
PROJECT NO. 4953-09-0301

Path: P:\4953 Geotech\2008-proj\90301 Belmont Pool\4.0 Project Deliverables\4.3 Drawings\4953-09-0301.dwg [B60]
Date: March 27, 2009 - 12:00pm By: LMorley

ABBREVIATED EXPLANATION

Approximate stratigraphic relationships only; see accompanying Sheet 2 for correlation of map units and more detailed descriptions.

CENOZOIC

QUATERNARY – HOLOCENE

- af — Artificial fill
- Qw — Active channel and wash deposits
- Qa — Alluvial flood plain deposits
- Qls — Landslide deposits
- Qb — Beach deposits
- Qe — Eolian deposits
- Qps — Paralic estuarine deposits
- Qyf — Young alluvial fan and valley deposits, undivided a = sand, s = silt, c = clay
- Qyf2 — Young alluvial fan deposits, unit 2
- Qyf1 — Young alluvial fan deposits, unit 1
- Qya — Young alluvial flood plain deposits, unit 1
- Qye — Young eolian deposits
- Qyps — Young paralic estuarine deposits
- Qof — Old alluvial fan and valley deposits, undivided a = sand, s = silt, c = clay
- Qoa — Old alluvial flood plain deposits, undivided

QUATERNARY – PLEISTOCENE

- Qoe — Old eolian deposits
- Qom — Old marine deposits, undivided
- Qop — Old paralic deposits, undivided, a = sand, s = silt, c = clay
- Qlh — La Habra Formation
- San Pedro Formation
  - Qsp — San Pedro Formation, undivided
  - Qspt — Timms Point Silt Member
  - Qspl — Lomita Marl Member
- Qi — Inglewood Formation

TERTIARY – PLIOCENE

- Fernando Formation
  - Tfu — Upper Member
  - Tfl — Lower Member, Tflc = conglomerate
- Pico Formation
  - Tpsc — Sycamore Canyon Member, Tpsco = conglomerate
  - Tpy — Yorba Member
  - Tps — Sespe Member
  - Tplv — La Vida Member

TERTIARY – MIOCENE

- Monterey Formation
  - Tmm — Malaga Mudstone Member
  - Tmvd — Valmonte Diatomite Member
  - Tmv — Volcanic rocks within the Monterey Formation
  - Tma — Altamira Shale Member

MESOZOIC – CRETACEOUS JURASSIC

- mzs — Catalina Schist

Offshore Region

- Qms — Unconsolidated shelf sediment
- Qmf — Unconsolidated flank sediment
- Qmb — Unconsolidated basin sediment
- Qmr — Unconsolidated ridge sediment
- Qmc — Unconsolidated canyon sediment
- Qct — Canyon terrace
- Qcf — Canyon fill
- Qgf — Gully fill
- Qf — Fan deposits
- Qls — Landslide deposits
- Qp — Pleistocene sedimentary deposits, undivided
- QTt — Plio-Pleistocene terrace deposits
- Tp — Pliocene sedimentary rocks, undivided*
- Tmp — Mio-Pliocene sedimentary rocks, undivided*
- Tu — Tertiary sedimentary and volcanic rocks undivided*
- Tmu — Miocene sedimentary rocks, undivided*
- Tmv — Miocene volcanic rocks*
- ms — Metamorphic rocks of pre- Late Cretaceous age*

* Qf = Map unit overlain by more than 3 meters of unconsolidated Quaternary sediment.

MAP SYMBOLS

Contact - accuracy of location ranges from well located to inferred. All offshore contacts are considered approximately located.

Fault - solid where well located; dashed where approximately located or inferred; dotted where concealed; queried where continuation or existance is uncertain. Where age was determined in offshore area, age symbol is shown astride fault and relative offset is shown by U, upthrown side; D, downthrown side (relative or apparent). Age of faults are indicated as follows:

- cuts strata of Holocene age
- cuts strata of Pleistocene age
- cuts strata of Quaternary age
- cuts strata of Pliocene age
- cuts Miocene or older strata

Anticlinal fold - solid where well located; dashed where approximately located or inferred; dotted where concealed. Plunge direction indicated by arrowhead on fold axis.

Synclinal fold - solid where well located; dashed where approximately located or inferred; dotted where concealed. Plunge direction indicated by arrowhead on fold axis.

Strike and dip of stratified rocks. Number indicates dip angle in degrees when known.

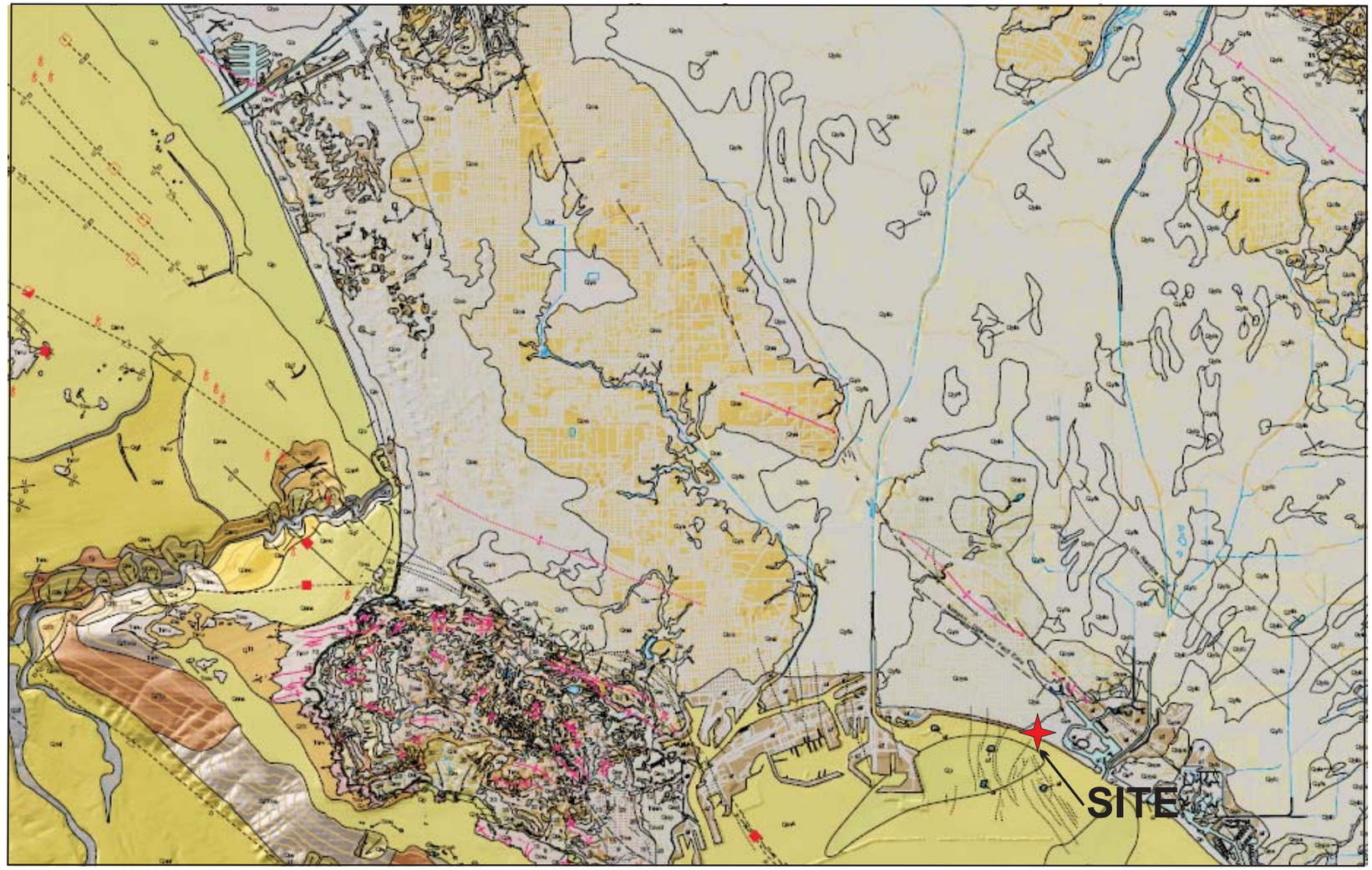
- 25 — Inclined beds
- 80 — Overturned beds
- Horizontal beds

Strike and dip of metamorphic and igneous foliation.

- Vertical foliation

Arrows on landslides indicate direction of movement. Hachured where headscarp is mappable.

33° 30' 7' — Oil and/or gas seep.



Base: California Geological Survey, 2003, Geologic Map of the Long Beach 30'x 60' Quadrangle, California, Map No. 5

SCALE 1:100,000

MILES

KILOMETERS

THOUSAND FEET

CONTOUR INTERVAL 40 METERS
BATHYMETRIC CONTOUR INTERVAL 100 METERS

N

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Figure 2 - Regional Geology

Belmont Plaza Olympic Pool Revitalization
Long Beach, California

JOB NO.:	4953-09-0301	REVISIONS:
DATE:	03/30/2009	
SCALE:	As shown	
DRAWN BY:	DGW	
CHECKED BY:	PER	

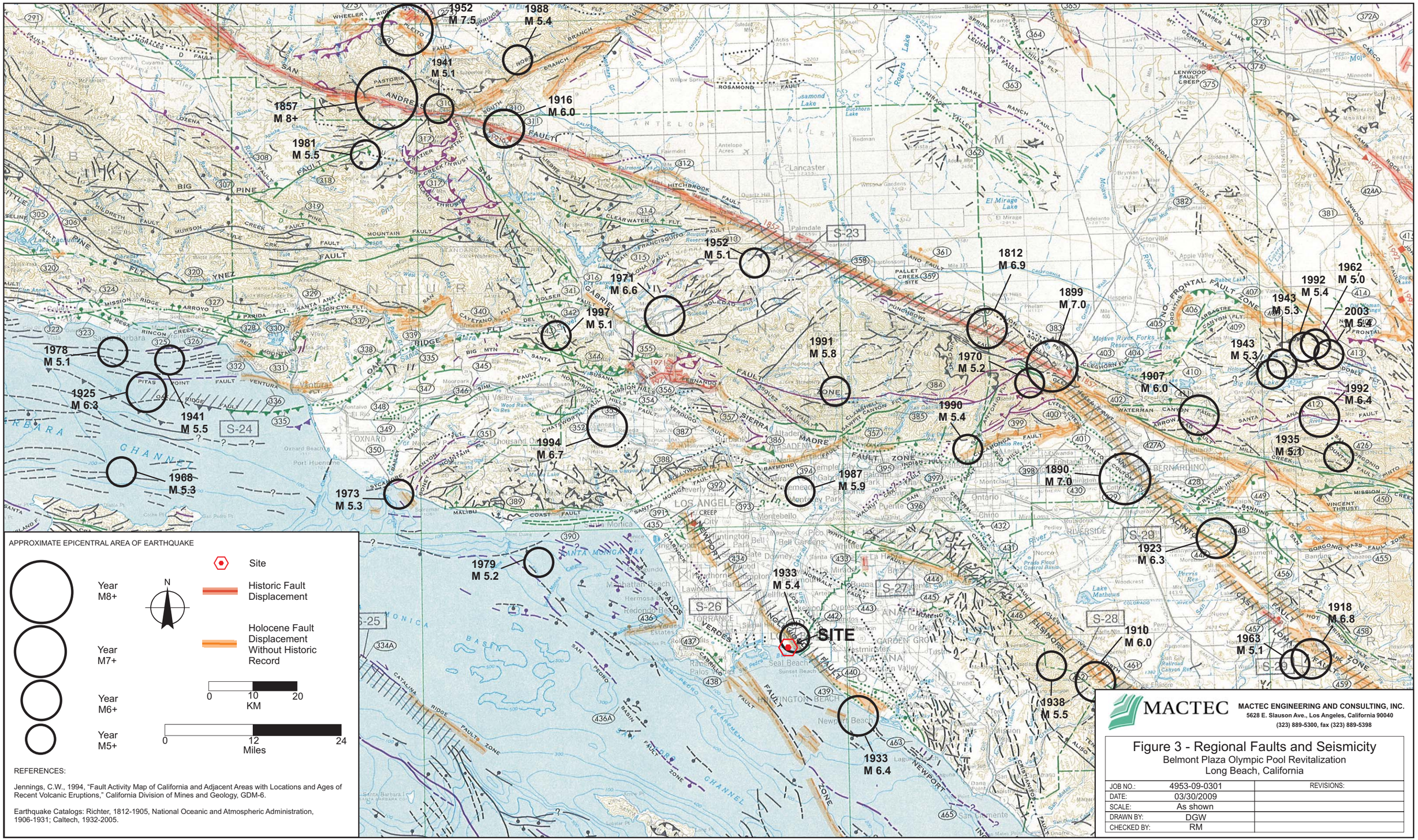

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Figure 3 - Regional Faults and Seismicity

Belmont Plaza Olympic Pool Revitalization
Long Beach, California

		REVISIONS:
JOB NO.:	4953-09-0301	
DATE:	03/30/2009	
SCALE:	As shown	
DRAWN BY:	DGW	
CHECKED BY:	RM	

APPENDIX A

CURRENT EXPLORATIONS AND LABORATORY TESTS

APPENDIX A

CURRENT EXPLORATIONS AND LABORATORY TESTS

EXPLORATIONS

The soil conditions beneath the site were explored by drilling two borings. In addition, data were available from our prior investigation at the site (our Job No. A-66102). The locations of our current and prior borings are shown on Figure 1. The current borings were drilled to depths of about 76½ feet below the existing grade using 5-inch-diameter rotary-wash-type drilling equipment. Drilling mud was used to prevent caving. The mud was removed following completion of the drilling to permit future measurements of the water level.

The soils encountered were logged by our field technician, and undisturbed and bulk samples were obtained for laboratory inspection and testing. The logs of the current borings are presented on Figures A-1.1 through A-1.2; the logs from our prior borings are presented in our previous report in Appendix D. The depths at which the undisturbed samples were obtained are indicated to the left of the boring logs. The number of blows required to drive the Crandall sampler 12 inches using a 300 pound hammer falling 18 inches is indicated on the logs. The soils are classified in accordance with the Unified Soil Classification System described on Figure A-2.

LABORATORY TESTS

Laboratory tests were performed on selected samples obtained from the borings to aid in the classification of the soils and to determine their engineering properties.

The field moisture content and dry density of the soils encountered were determined by performing tests on the undisturbed samples. The results of the tests are shown to the left on the boring logs.

Direct shear tests were performed on selected undisturbed samples to determine the strength of the soils. The tests were performed after soaking to near-saturated moisture content and at various surcharge pressures. The yield-point values determined from the direct shear tests are presented on Figure A-3, Direct Shear Test Data.

Confined consolidation tests were performed on one undisturbed sample to determine the compressibility of the soils. Water was added to the sample during the tests to illustrate the effect

of moisture on the compressibility. The results of the tests are presented on Figure A-4, Consolidation Test Data.

To determine the particle size distribution of the soils and to aid in classifying the soils, mechanical analyses were performed on seven samples. The results of the mechanical analyses are presented on Figures A-5.1 through A-5.4, Particle Size Distribution.

Soil corrosivity studies were performed on samples of the on-site soils. The results of the study and recommendations for mitigating procedures are presented on Appendix D.

APPENDIX B

CONE PENETRATION TEST DATA

APPENDIX C

CORROSION STUDY

APPENDIX D

PRIOR REPORT OF GEOTECHNICAL INVESTIGATION

FIGURES

APPENDIX A

CURRENT EXPLORATIONS AND LABORATORY TESTS

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BORING 1

DATE DRILLED: March 5, 2009
EQUIPMENT USED: Rotary Wash
HOLE DIAMETER (in.): 4.9
ELEVATION: 10.0**

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. LATITUDE AND LONGITUDE OF BORING LOCATION SHOWN ON LOGS ARE APPROXIMATE; REFER TO PLOT PLAN FOR MORE ACCURATE LOCATION INFORMATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.		
							SM	3-inch thick Asphalt Concrete FILL - SILTY SAND - moist, brown
							SP	POORLY GRADED SAND - medium dense, moist, light brown, trace Silt, medium to coarse grained, some shell fragments
			14.5	97	40			
5	5		12.1	100	20			
			21.8	106	14		CL	SANDY CLAY - stiff, wet, olive gray
							SP	POORLY GRADED SAND - medium dense, wet, grayish brown, fine grained with trace fine gravel
0	10	28						5.6% Passing No. 200 Sieve
			17.9	115	27		CL	SANDY CLAY - very stiff to hard, wet, reddish-brown (LL=31, PI=13)
-5	15	33						
							ML	SANDY SILT - hard, moist, olive gray
-10	20		27.5	98	60		SM	SILTY SAND - dense, moist, olive gray, fine grained
-15	25	46					SP-SM	POORLY GRADED SAND with SILT - dense, wet, gray, fine grained, trace silt
-20	30	44						Becomes bluish gray
-25	35	53						Becomes very dense
	40							

Field Tech: DW
Prepared By: MKT
Checked By: LT 4/14/09

(CONTINUED ON FOLLOWING FIGURE)

Proposed Belmont Plaza
Olympic Pool Project
Long Beach, California

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LOG OF BORING
Project: 4953-09-0301 Figure: A-1.1a

B12SOIL_CRANDALL(DECIMAL_ELE) 90301.GPJ LAW_CRAN.GDT 4/14/09

BORING 1 (Continued)

DATE DRILLED: March 5, 2009
EQUIPMENT USED: Rotary Wash
HOLE DIAMETER (in.): 4.9
ELEVATION: 10.0**

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.	
		73					8.6% Passing No. 200 Sieve Thin layer of Clayey Silt
-35	45		19.7	99	83/9"		Some Clay
-40	50	69					
-45	55	30					Dense, some coarse grained with fine gravel up to ¼ inch in size
-50	60	54					* Number of blows required to drive Crandall sampler 12 inches using a 300 pound hammer falling 18 inches. ** Elevation based on Aerial Topographic Survey provided by PSOMAS.
-55	65	58					9.2% Passing No. 200 Sieve Thin layer of Clayey Silt
-60	70		20.7	96	50/3"		SP POORLY GRADED SAND - very dense, wet, gray, fine to medium grained
-65	75	68					CL-ML SILTY CLAY - hard, moist, gray (LL=36, PI=12)
	80						END OF BORING AT 76.5 FEET NOTES: Boring bailed to 30 feet. Ground water measured at 9 feet. Boring backfilled with bentonite cement grout from bottom up and patched with asphalt concrete.

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. LATITUDE AND LONGITUDE OF BORING LOCATION SHOWN ON LOGS ARE APPROXIMATE; REFER TO PLOT PLAN FOR MORE ACCURATE LOCATION INFORMATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

Field Tech: DW
Prepared By: MKT
Checked By: LT 4/14/09

Proposed Belmont Plaza
Olympic Pool Project
Long Beach, California

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LOG OF BORING
Project: 4953-09-0301 Figure: A-1.1b

BORING 2

DATE DRILLED: March 6, 2009
EQUIPMENT USED: Rotary Wash
HOLE DIAMETER (in.): 4.9
ELEVATION: 9.5**

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. LATITUDE AND LONGITUDE OF BORING LOCATION SHOWN ON LOGS ARE APPROXIMATE; REFER TO PLOT PLAN FOR MORE ACCURATE LOCATION INFORMATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.		
							SP	FILL - POORLY GRADED SAND - moist, brown, fine grained, some debris fragments
			-	-	28		SP	POORLY GRADED SAND - medium dense, moist, brown, fine grained, some shell fragments
5	5		25.6	96	25		SP-SM	POORLY GRADED SAND with SILT - medium dense, moist, brownish-gray, fine grained
			18.2	110	21		SP	POORLY GRADED SAND - medium dense, wet, grayish-brown, fine to medium grained
0	10	17					SM	SILTY SAND - medium dense, wet, olive gray, fine grained
			24.7	99	25		SP	POORLY GRADED SAND - medium dense, wet, gray, fine grained
							SM	SILTY SAND - medium dense, wet, olive gray, fine grained
-5	15	9					ML	CLAYEY SILT - stiff, wet, olive brown (LL=40, PI=11)
							SM	SILTY SAND - medium dense, wet, olive brown, fine grained
-10	20	28						27.3% Passing No. 200 Sieve
-15	25	35					SP-SM	POORLY GRADED SAND with SILT - dense, wet, brown, fine grained 6.3% Passing No. 200 Sieve
-20	30	51						Some fine gravel up to ¼ inch in size, FeO2 staining 5.5% Passing No. 200 Sieve
-25	35	43						
-30	40							

(CONTINUED ON FOLLOWING FIGURE)

Field Tech: DW
Prepared By: MKT
Checked By: LT 4/15/09

Proposed Belmont Plaza
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Long Beach, California

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LOG OF BORING
Project: 4953-09-0301 Figure: A-1.2a

B12SOIL_CRANDALL(DECIMAL_ELE) 90301.GPJ LAW_CRAN.GDT 4/14/09

BORING 2 (Continued)

DATE DRILLED: March 6, 2009
EQUIPMENT USED: Rotary Wash
HOLE DIAMETER (in.): 4.9
ELEVATION: 9.5**

THIS RECORD IS A REASONABLE INTERPRETATION OF SUBSURFACE CONDITIONS AT THE EXPLORATION LOCATION. LATITUDE AND LONGITUDE OF BORING LOCATION SHOWN ON LOGS ARE APPROXIMATE; REFER TO PLOT PLAN FOR MORE ACCURATE LOCATION INFORMATION. SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND AT OTHER TIMES MAY DIFFER. INTERFACES BETWEEN STRATA ARE APPROXIMATE. TRANSITIONS BETWEEN STRATA MAY BE GRADUAL.

ELEVATION (ft)	DEPTH (ft)	"N" VALUE STD.PEN.TEST	MOISTURE (% of dry wt.)	DRY DENSITY (pcf)	BLOW COUNT* (blows/ft)	SAMPLE LOC.		
			19.8	105	64/9"	⊠		Thin layer of Sandy Silt
-35	45	51				⊠	CL	SANDY CLAY - hard, wet, gray
							SP	POORLY GRADED SAND - very dense, wet, brown, fine grained, some fine gravel up to ¼ inch in size
-40	50	63				⊠		
-45	55	92/11"				⊠		
-50	60	50/5"				⊠		Becomes gray and orangish brown
-55	65	90				⊠		
-60	70		26.0	101	80/8"	⊠	ML	SANDY SILT - hard, wet, gray 82.3% Passing No. 200 Sieve
							SP-SM	POORLY GRADED SAND with SILT - very dense, wet, gray, fine grained, trace Clay
-65	75	97/9"				⊠		
-70	80							END OF BORING AT 76.5 FEET NOTES: Boring bailed to 30 feet. Ground water measured at 6.4 feet. Boring backfilled with bentonite cement grout from bottom up and patched with asphalt concrete.

Field Tech: DW
Prepared By: MKT
Checked By: LT (4/14/09)

Proposed Belmont Plaza
Olympic Pool Project
Long Beach, California

MACTEC

LOG OF BORING
Project: 4953-09-0301 Figure: A-1.2b

B12SOIL CRANDALL(DECIMAL ELE) 90301.GPJ LAW CRAN.GDT 4/14/09

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOILS (More than 50% of material is LARGER than No. 200 sieve size)	GRAVELS (More than 50% of coarse fraction is LARGER than the No. 4 sieve size)	CLEAN GRAVELS (Little or no fines)	GW	Well graded gravels, gravel - sand mixtures, little or no fines.
			GP	Poorly graded gravels or grave - sand mixtures, little or no fines.
		GRAVELS WITH FINES (Appreciable amount of fines)	GM	Silty gravels, gravel - sand - silt mixtures.
			GC	Clayey gravels, gravel - sand - clay mixtures.
	SANDS (More than 50% of coarse fraction is SMALLER than the No. 4 Sieve Size)	CLEAN SANDS (Little or no fines)	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		SANDS WITH FINES (Appreciable amount of fines)	SM	Silty sands, sand - silt mixtures
			SC	Clayey sands, sand - clay mixtures.
FINE GRAINED SOILS (More than 50% of material is SMALLER than No. 200 sieve size)	SILTS AND CLAYS (Liquid limit LESS than 50)		ML	Inorganic silts and very fine sands, rock flour, silty of clayey fine sands or clayey silts and with slight plasticity.
			CL	Inorganic lays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS (Liquid limit GREATER than 50)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS			PT	Peat and other highly organic soils.

	Undisturbed Sample		Auger Cuttings
	Split Spoon Sample		Bulk Sample
	Rock Core		Crandall Sampler
	Dilatometer		Pressure Meter
	Packer		No Recovery
	Water Table at time of drilling		Water Table after drilling

Correlation of Penetration Resistance with Relative Density and Consistency

SAND & GRAVEL		SILT & CLAY	
No. of Blows	Relative Density	No. of Blows	Consistency
0 - 4	Very Loose	0 - 1	Very Soft
5 - 10	Loose	2 - 4	Soft
11 - 30	Medium Dense	5 - 8	Medium Stiff
31 - 50	Dense	9 - 15	Stiff
Over 50	Very Dense	16 - 30	Very Stiff
		Over 30	Hard

BOUNDARY CLASSIFICATIONS: Soils possessing characteristics of two groups are designated by combinations of group symbols.

SILT OR CLAY	SAND			GRAVEL		Cobbles	Boulders
	Fine	Medium	Coarse	Fine	Coarse		

No.200 No.40 No.10 No.4 3/4" 3" 12"

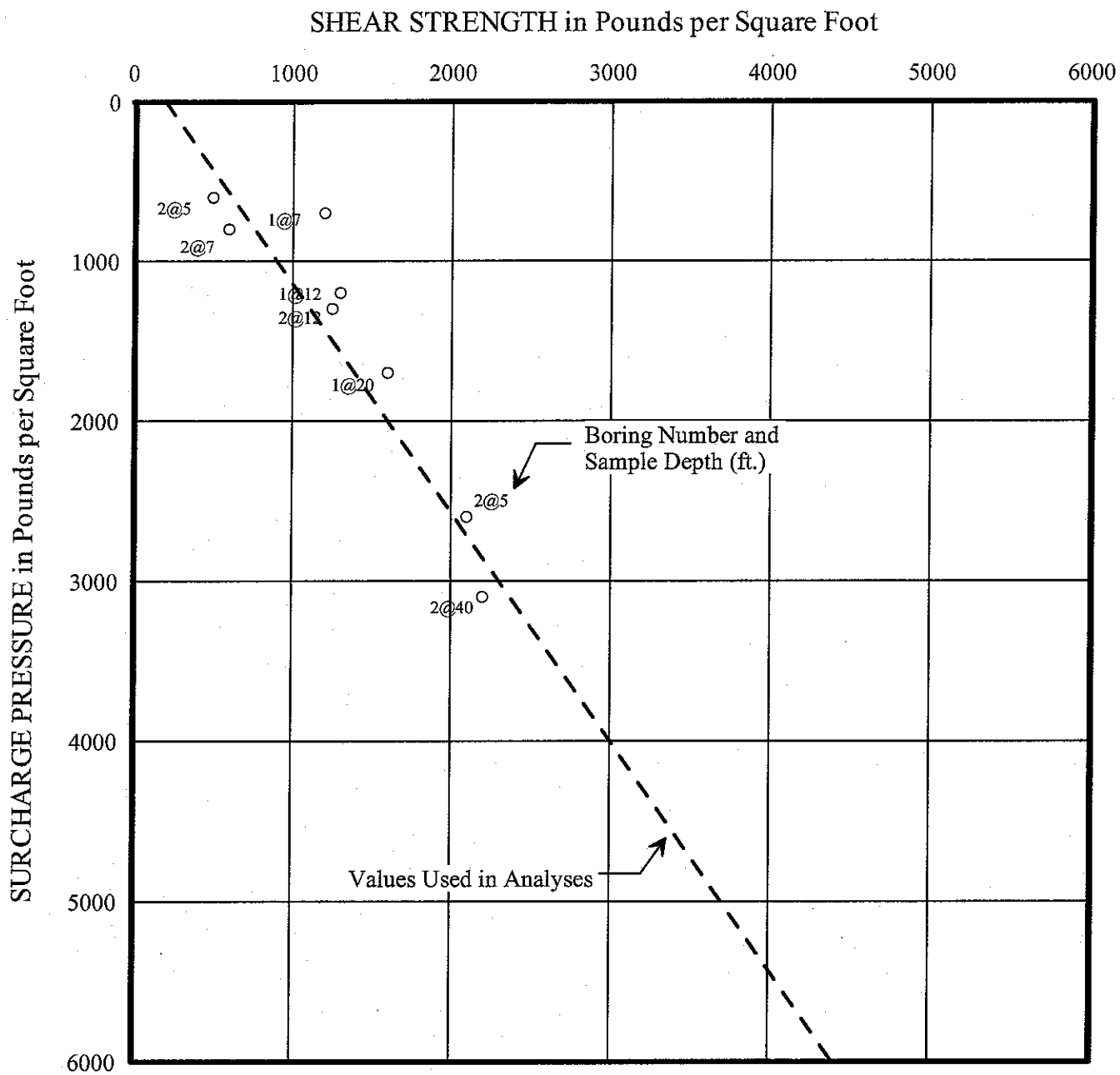
U.S. STANDARD SIEVE SIZE

Reference: The Unified Soil Classification System, Corps of Engineers, U.S. Army Technical Memorandum No. 3-357, Vol. 1, March, 1953 (Revised April, 1960)

KEY TO SYMBOLS AND DESCRIPTIONS

MACTEC

Figure A-2

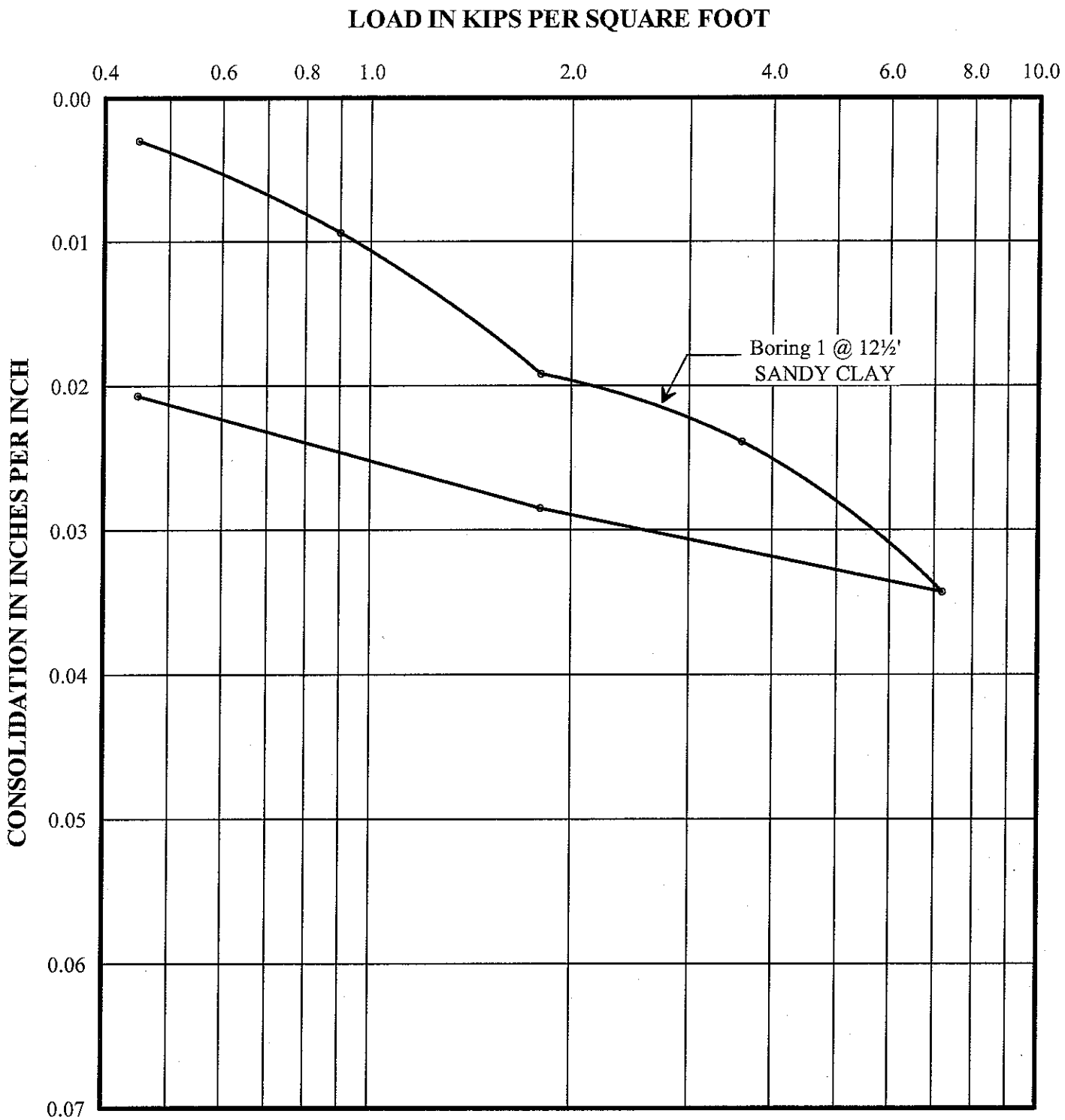


KEY: ○ Samples tested after soaking to a moisture content near saturation.

└ Natural Soils

Prepared/Date: MKT 3/25/09
Checked/Date: LT 4/7/09

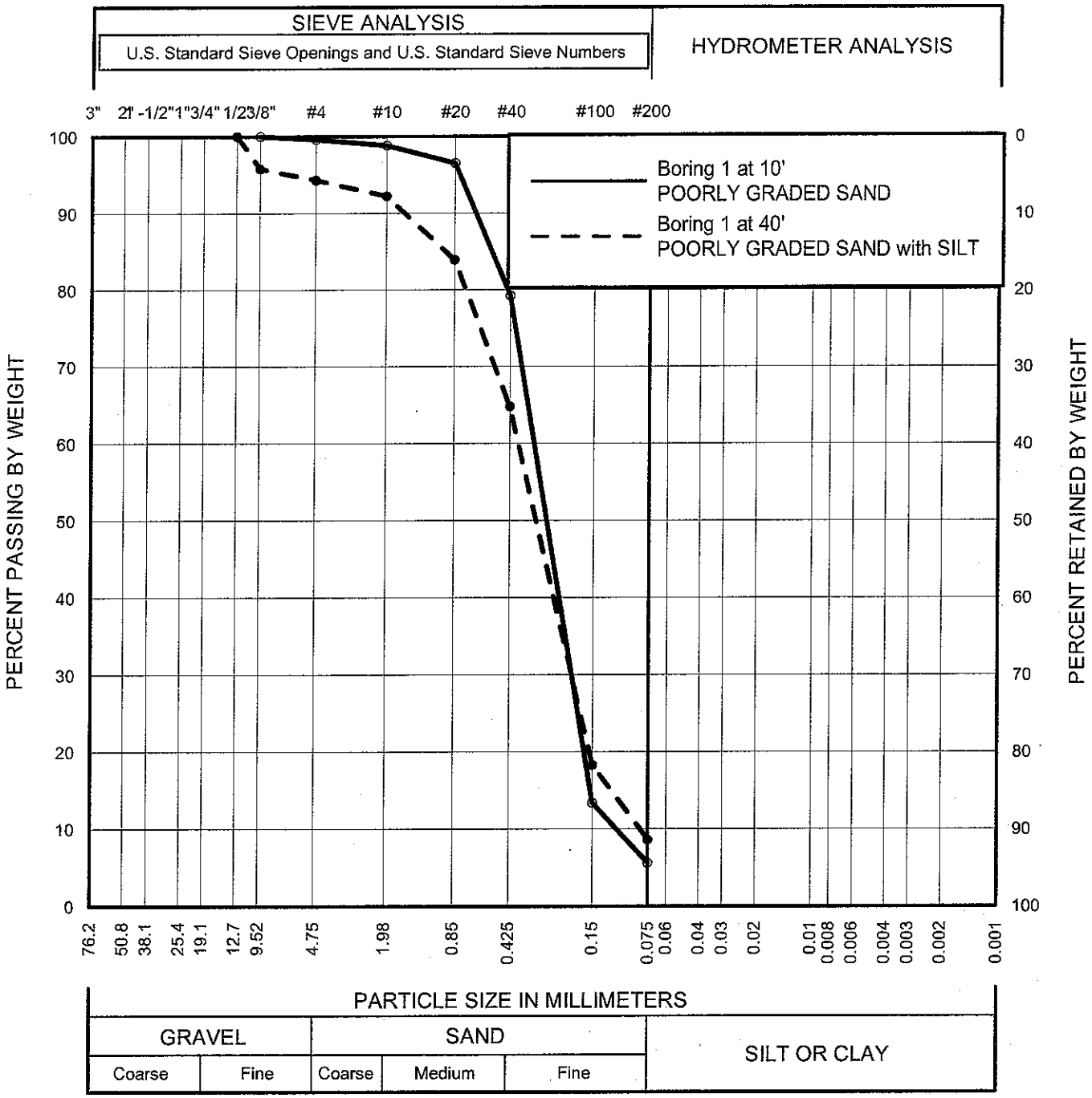
Proposed Belmont Plaza Olympic Pool Project Long Beach, California	MACTEC	DIRECT SHEAR TEST DATA Project No. 4953-09-0301 Figure A-3



Note: Water added to a sample after consolidation under a load of 1.8 kips per square foot.

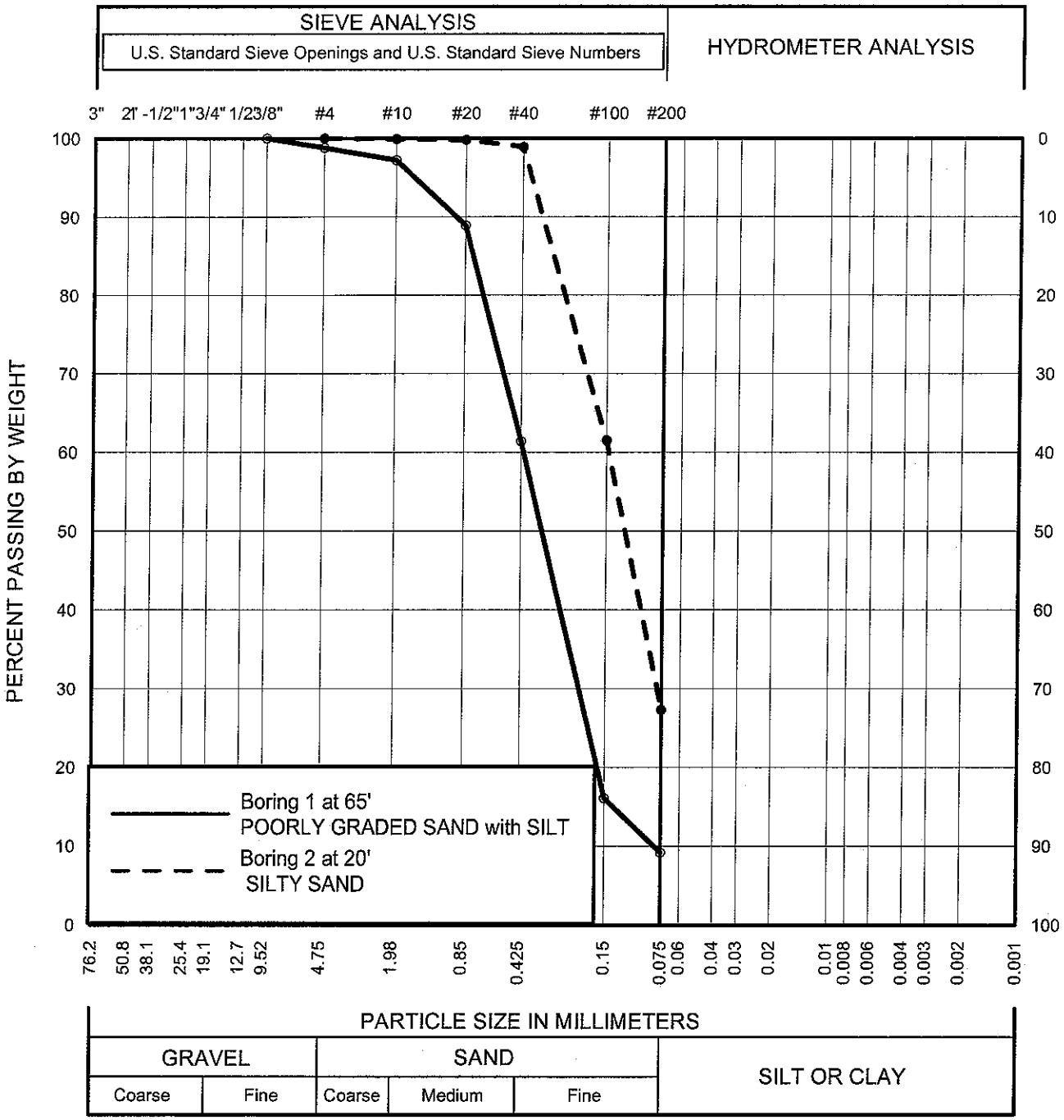
Prepared/Date: MKT 4/6/09
Checked/Date: BOK 4/15/09

Proposed Belmont Plaza Olympic Pool Project Long Beach, California	MACTEC	CONSOLIDATION TEST DATA Project 4953-09-0301 Figure A-4



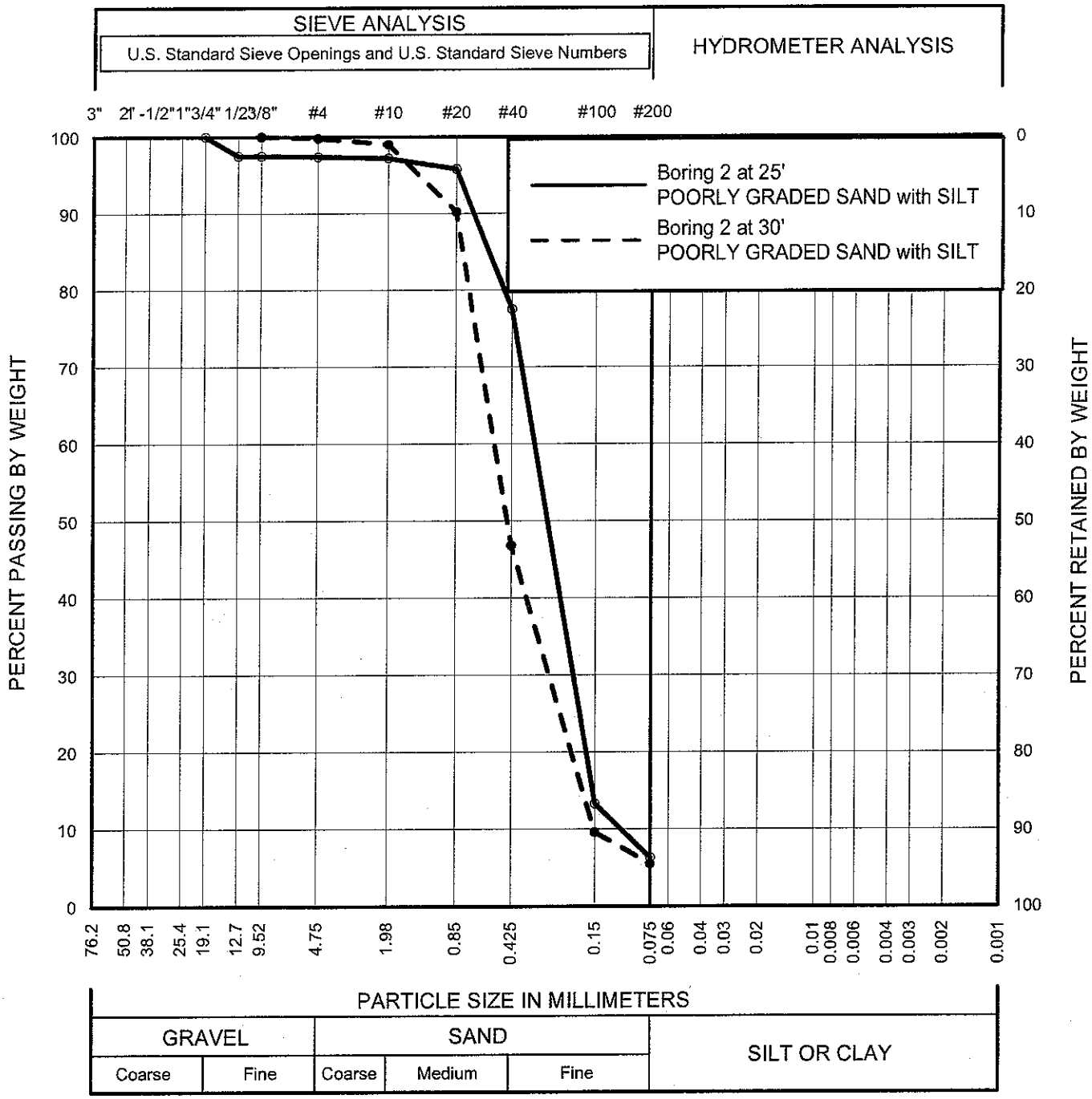
Prepared/Date: MKT 3/25/09
Checked/Date: LT 4/9/09

Proposed Belmont Plaza Olympic Pool Project Long Beach, California	MACTEC	PARTICLE SIZE DISTRIBUTION Project No. 4953-09-0301 Figure A-5.1



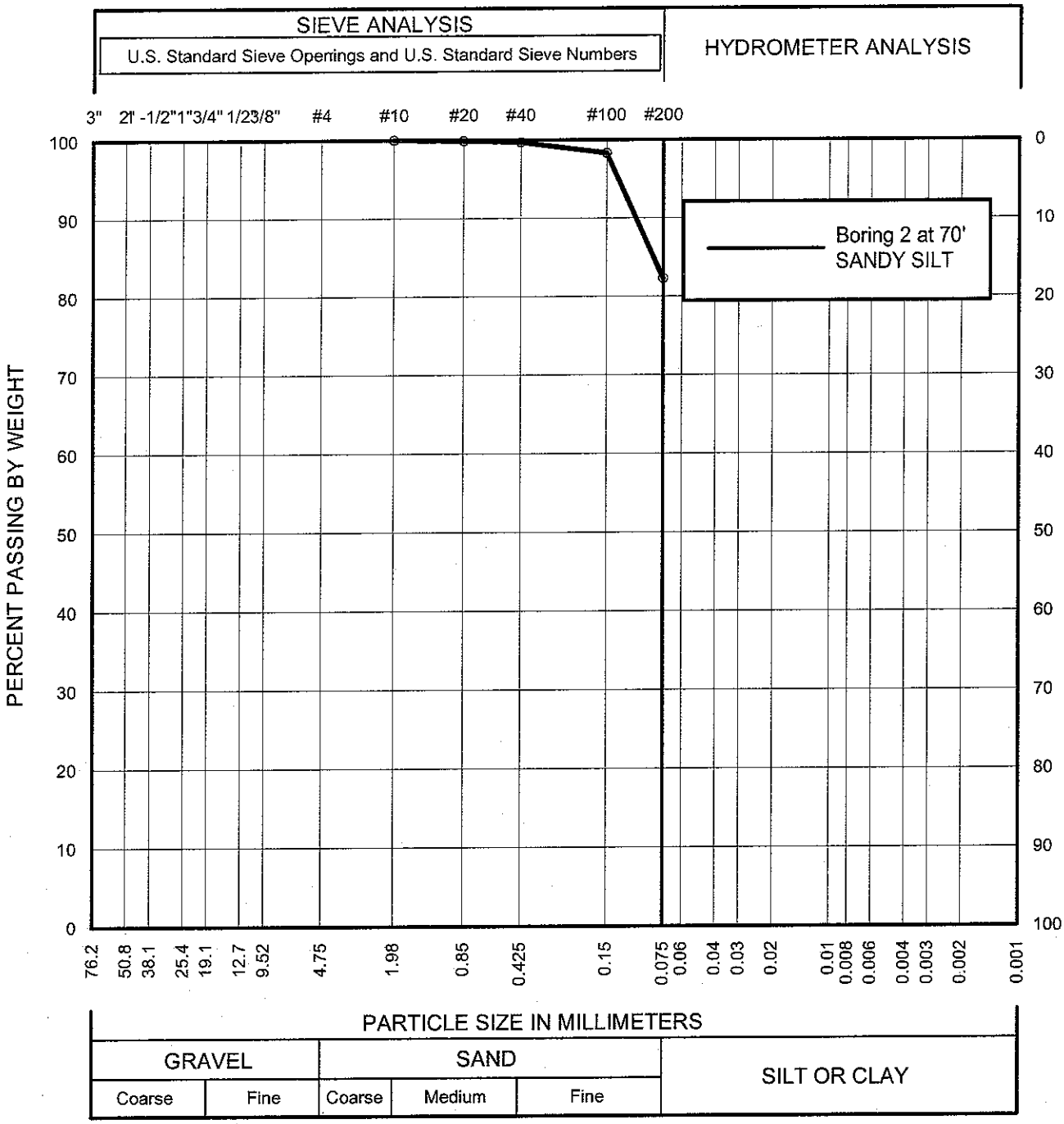
Prepared/Date: MKT 3/25/09
Checked/Date: BOK 4/15/09

Proposed Belmont Plaza Olympic Pool Project Long Beach, California	MACTEC	PARTICLE SIZE DISTRIBUTION Project No. 4953-09-0301 Figure A-5.2



Prepared/Date: MKT 3/25/09
Checked/Date: LT 4/9/09

Proposed Belmont Plaza Olympic Pool Project Long Beach, California	MACTEC	PARTICLE SIZE DISTRIBUTION Project No. 4953-09-0301 Figure A-5.3



Prepared/Date: MKT 3/25/09
Checked/Date: LT (4/9/09)

Proposed Belmont Plaza Olympic Pool Project Long Beach, California	MACTEC	PARTICLE SIZE DISTRIBUTION Project No. 4953-09-0301 Figure A-5.4

APPENDIX B

CONE PENETRATION TEST DATA

Reviewed and acceptable for use

NAME: _____ DATE: 4/14/2009

MACTEC Engineering and Consulting, Inc.

SUMMARY

OF

CONE PENETRATION TEST DATA

Project:

Belmont Plaza Pool
4000 E. Olympic Plaza
Long Beach, CA
March 6, 2009

Prepared for:

Mr. Angel Recio
MACTEC Engineering & Consulting, Inc.
5628 E. Slauson Avenue
Los Angeles, CA 90040-2922
Office (323) 889-5300 / Fax (323) 721-6700

Prepared by:

KEHOE TESTING & ENGINEERING
5415 Industrial Drive
Huntington Beach, CA 92649-1518
Office (714) 901-7270 / Fax (714) 901-7289
www.kehoetesting.com

TABLE OF CONTENTS

1. **INTRODUCTION**

2. **SUMMARY OF FIELD WORK**

3. **FIELD EQUIPMENT & PROCEDURES**

4. **CONE PENETRATION TEST DATA & INTERPRETATION**

APPENDIX

- CPT Plots
- CPT Classification/Soil Behavior Chart
- Interpretation Output (CPTINT)
- Summary of Shear Wave Velocities
- CPTINT Correlation Table

SUMMARY

OF

CONE PENETRATION TEST DATA

1. INTRODUCTION

This report presents the results of a Cone Penetration Test (CPT) program carried out for the Belmont Plaza Pool project located at 4000 E. Olympic Plaza in Long Beach, California. The work was performed by Kehoe Testing & Engineering (KTE) on March 6, 2009. The scope of work was performed as directed by MACTEC Engineering & Consulting, Inc. personnel.

2. SUMMARY OF FIELD WORK

The fieldwork consisted of performing CPT soundings at five locations to determine the soil lithology. The groundwater measurements were taken in the open CPT hole approximately 10 minutes after completion of CPT. The following **TABLE 2.1** summarizes the CPT soundings performed:

LOCATION	DEPTH OF CPT (ft)	COMMENTS/NOTES:
CPT-1	60	Groundwater @ 8 ft
CPT-2	60	Groundwater @ 11 ft
CPT-3	60	Groundwater @ 8 ft
CPT-4	60	Hole open to 5 ft (dry)
CPT-5	60	Hole open to 2 ft (dry)

TABLE 2.1 - Summary of CPT Soundings

3. FIELD EQUIPMENT & PROCEDURES

The CPT soundings were carried out by **KTE** using an integrated electronic cone system manufactured by Vertek. The CPT soundings were performed in accordance with ASTM standards (D5778). The cone penetrometers were pushed using a 30-ton CPT rig. The cone used during the program was a 15 cm^2 cone and recorded the following parameters at approximately 2.5 cm depth intervals:

- Cone Resistance (qc)
- Sleeve Friction (fs)
- Dynamic Pore Pressure (u)
- Inclination
- Penetration Speed
- Pore Pressure Dissipation (at selected depths)

At location CPT-3, shear wave measurements were obtained at approximately 5-foot intervals. The shear wave is generated using an air-actuated hammer, which is located inside the front jack of the CPT rig. The cone has a triaxial geophone, which recorded the shear wave signal generated by the air hammer.

The above parameters were recorded and viewed in real time using a portable computer and stored on a diskette for future analysis and reference. A complete set of baseline readings was taken prior to each sounding to determine temperature shifts and any zero load offsets. Monitoring base line readings ensures that the cone electronics are operating properly.

4. CONE PENETRATION TEST DATA & INTERPRETATION

The Cone Penetration Test data is presented in graphical form in the attached Appendix. Penetration depths are referenced to ground surface. The soil classification on the CPT plots is derived from the CPT Classification Chart (Robertson, 1986) and presents major soil lithologic changes. The stratigraphic interpretation is based on relationships between cone resistance (qc), sleeve friction (fs), and penetration pore pressure (u). The friction ratio (Rf), which is sleeve friction divided by cone resistance, is a calculated parameter that is used to infer soil behavior type. Generally, cohesive soils (clays) have high friction ratios, low cone resistance and generate excess pore water pressures. Cohesionless soils (sands) have lower friction ratios, high cone bearing and generate little (or negative) excess pore water pressures.

Output from the interpretation program CPTINT provides averaged CPT data over one-foot intervals. The CPTINT output includes Soil Classification Zones, SPT N Values and Undrained Shear Strength (Su). A summary of the equations used for the tabulated parameters is provided in the CPTINT Correlation Table in the Appendix.

The interpretation of soils encountered on this project was carried out using correlations developed by Robertson et al, 1986. It should be noted that it is not always possible to clearly identify a soil type based on qc, fs and u. In these situations, experience, judgment and an assessment of the pore pressure data should be used to infer the soil behavior type.

If you have any questions regarding this information, please do not hesitate to call our office at (714) 901-7270.

Sincerely,

KEHOE TESTING & ENGINEERING

Richard W. Koester, Jr.
General Manager

03/09/09-at-83-9492

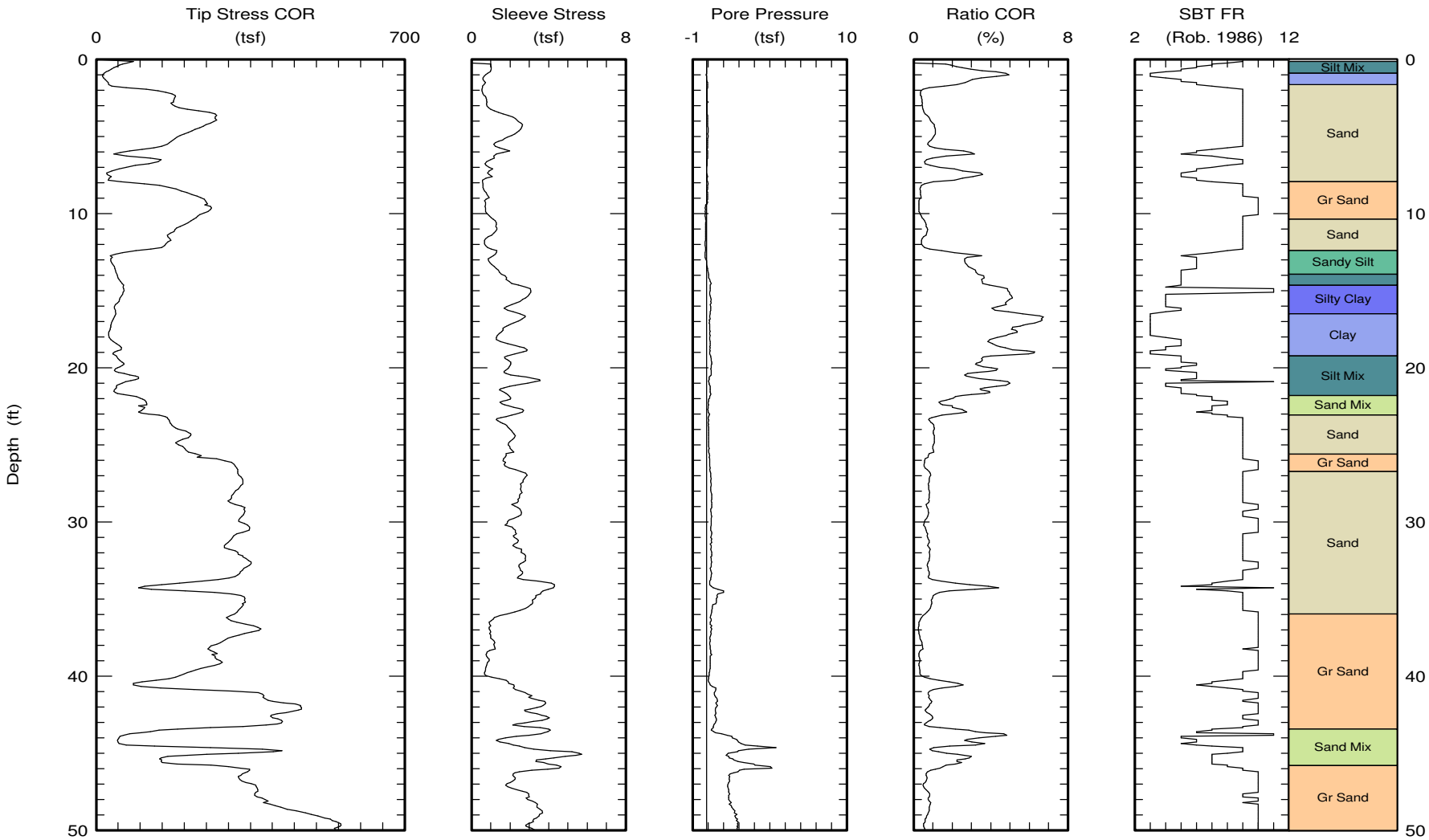
APPENDIX

Kehoe Testing & Engineering
Office: (714) 901-7270
Fax: (714) 901-7289
rich@kehoetesting.com
skehoe@msn.com

CPT Data
30 ton rig

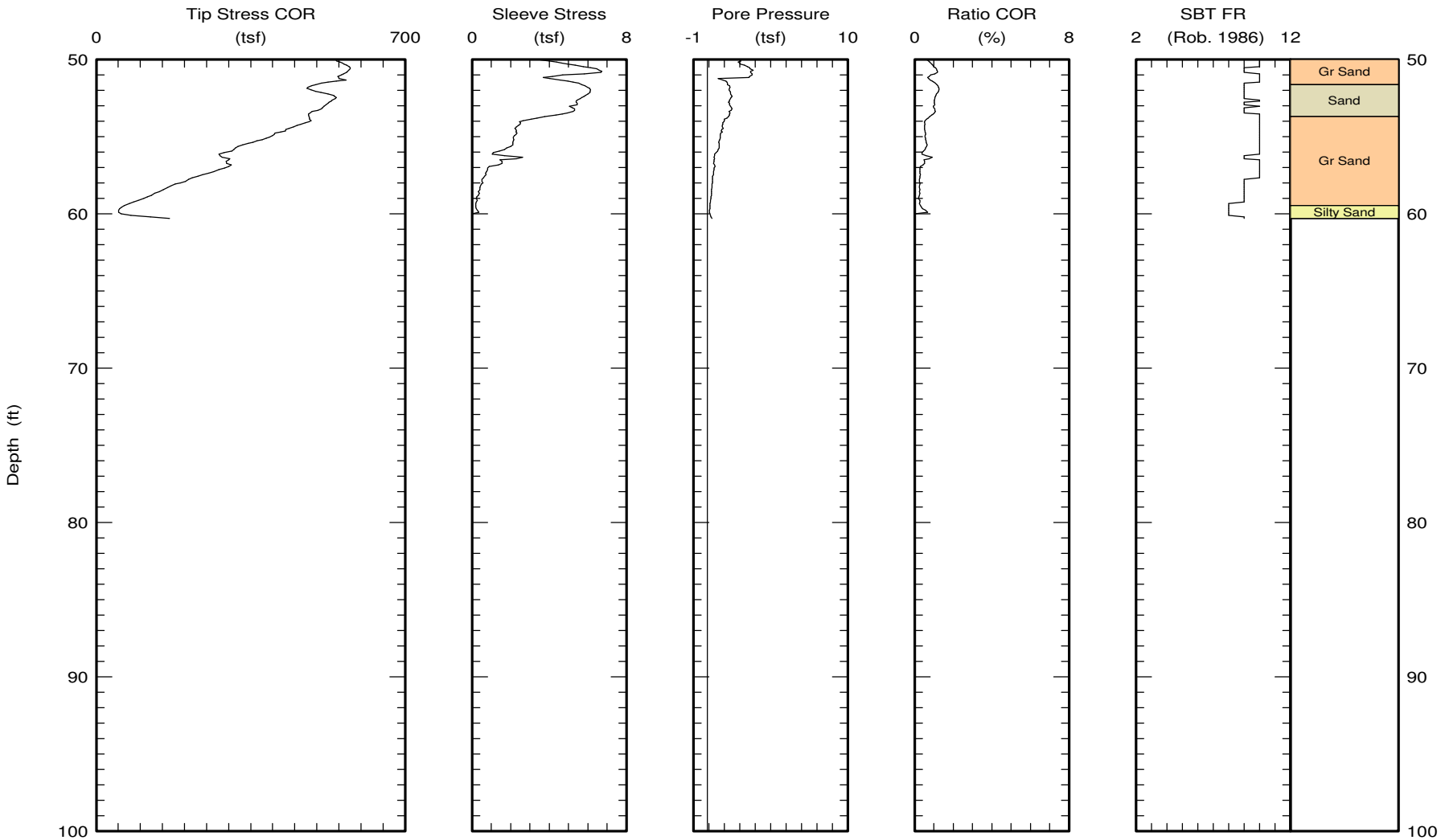
Date: 06/Mar/2009
Test ID: CPT-1
Project: LongBeach

Customer: MACTEC
Job Site: Belmont Plaza Pool



Maximum depth: 60.33 (ft)

Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 rich@kehoetesting.com skehoe@msn.com	CPT Data 30 ton rig	Date: 06/Mar/2009 Test ID: CPT-1 Project: LongBeach
	Customer: MACTEC Job Site: Belmont Plaza Pool	



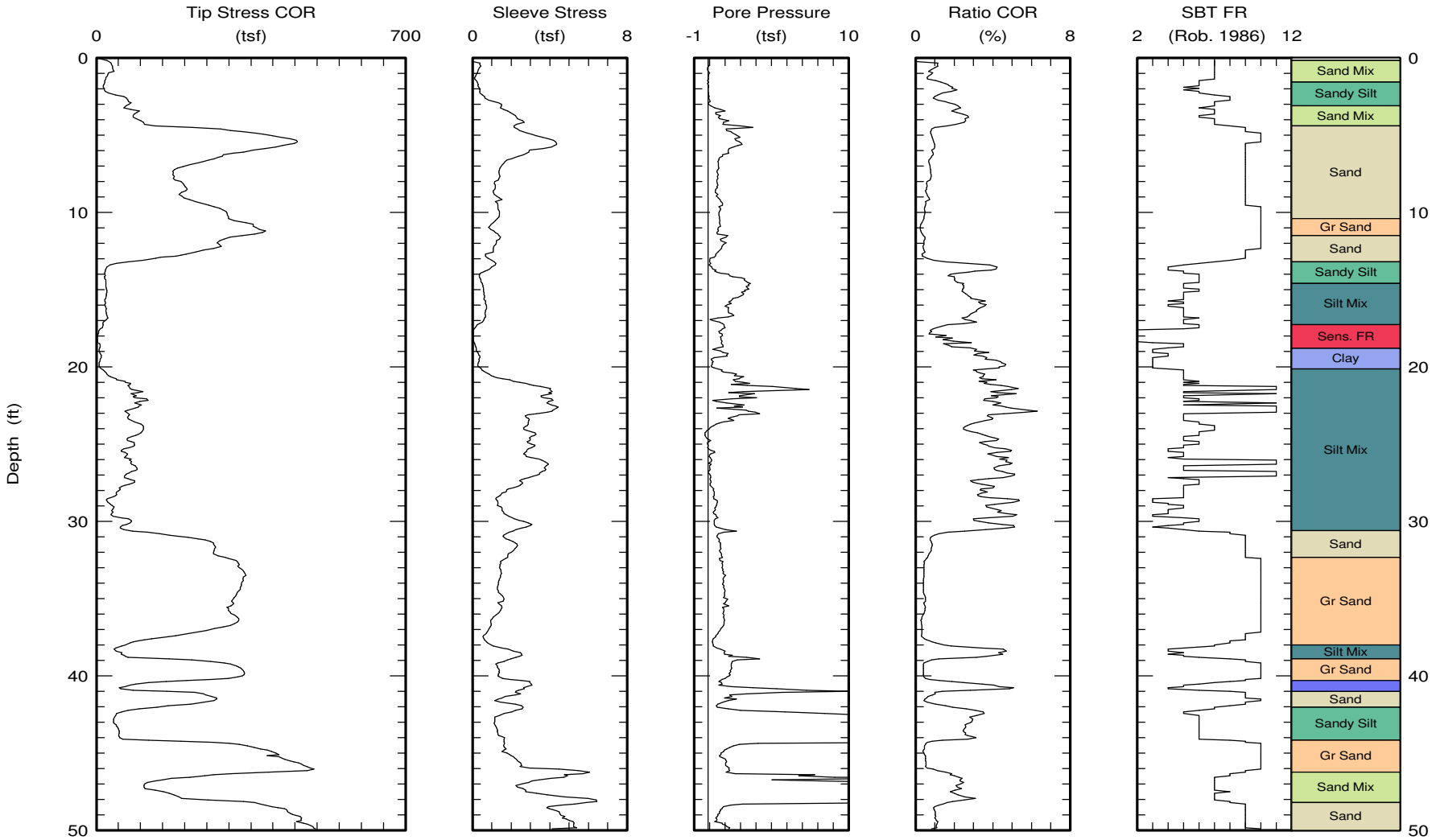
Maximum depth: 60.33 (ft)

Kehoe Testing & Engineering
Office: (714) 901-7270
Fax: (714) 901-7289
rich@kehoetesting.com
skehoe@msn.com

CPT Data
30 ton rig

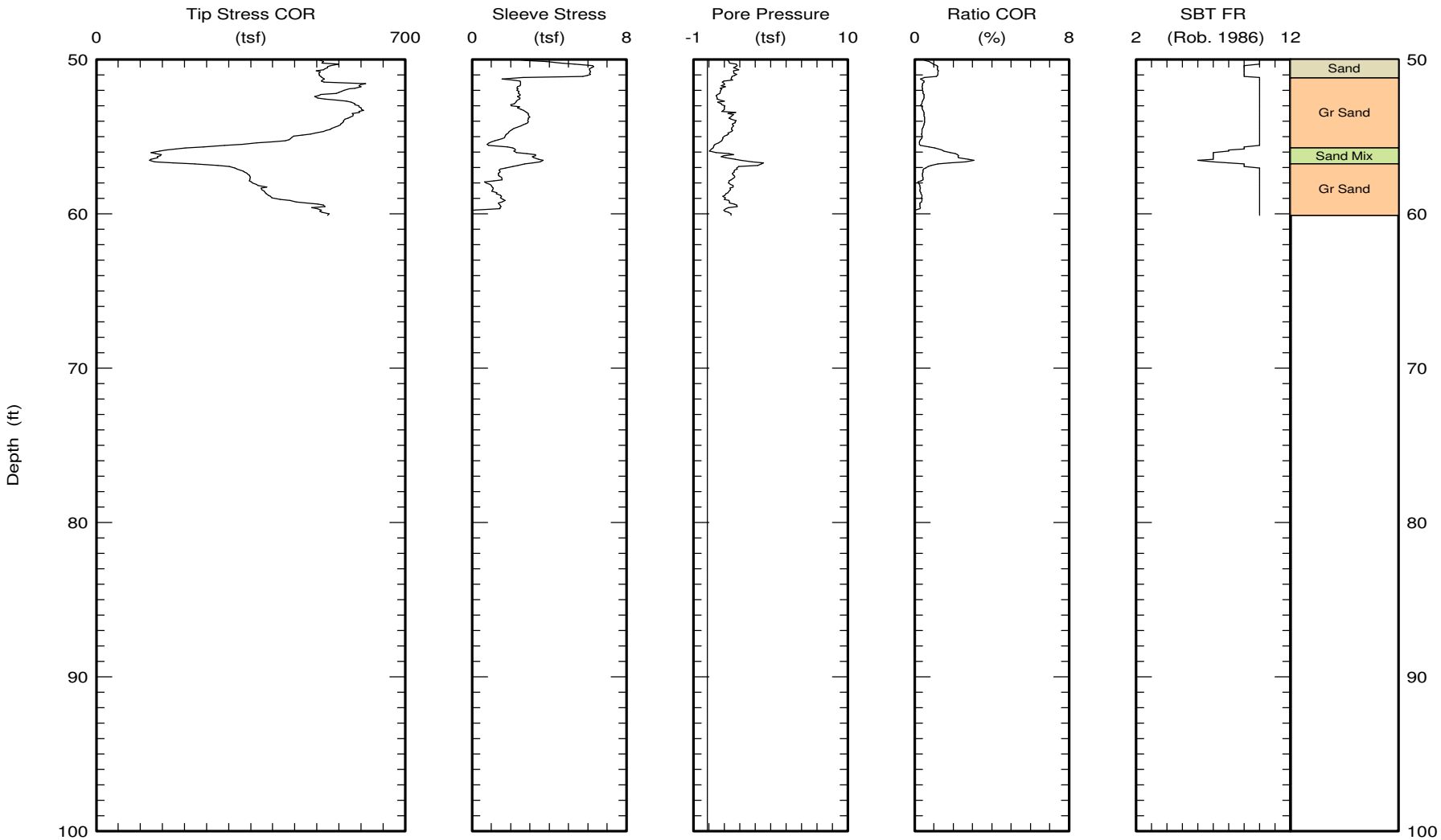
Date: 06/Mar/2009
Test ID: CPT-2
Project: LongBeach

Customer: MACTEC
Job Site: Belmont Plaza Pool



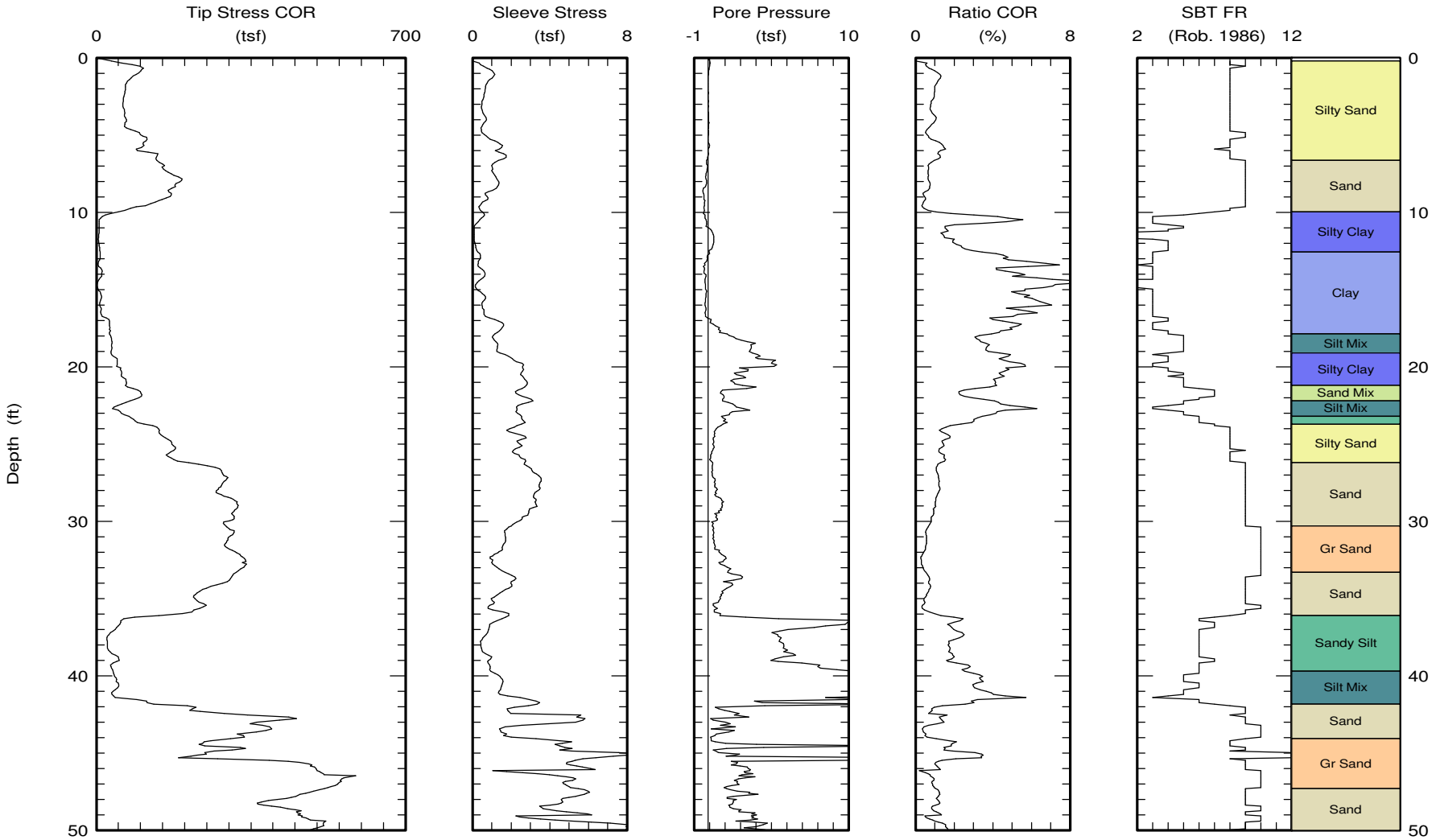
Maximum depth: 60.11 (ft)

Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 rich@kehoetesting.com skehoe@msn.com	CPT Data 30 ton rig	Date: 06/Mar/2009 Test ID: CPT-2 Project: LongBeach
	Customer: MACTEC Job Site: Belmont Plaza Pool	



Maximum depth: 60.11 (ft)

Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 rich@kehoetesting.com skehoe@msn.com	CPT Data 30 ton rig	Date: 06/Mar/2009 Test ID: CPT-3 Project: LongBeach
	Customer: MACTEC Job Site: Belmont Plaza Pool	



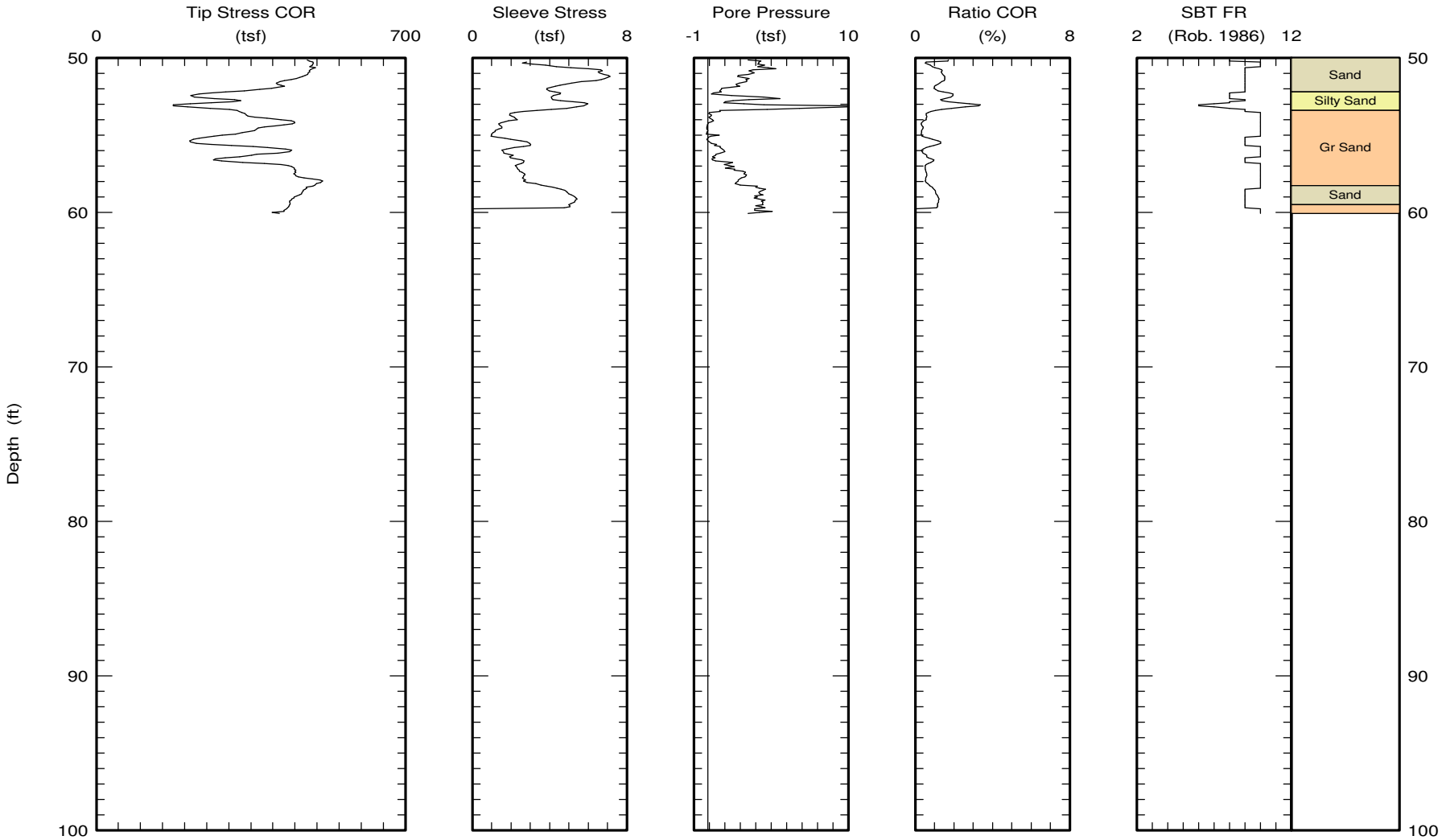
Maximum depth: 60.09 (ft)

Kehoe Testing & Engineering
Office: (714) 901-7270
Fax: (714) 901-7289
rich@kehoetesting.com
skehoe@msn.com

CPT Data
30 ton rig

Date: 06/Mar/2009
Test ID: CPT-3
Project: LongBeach

Customer: MACTEC
Job Site: Belmont Plaza Pool



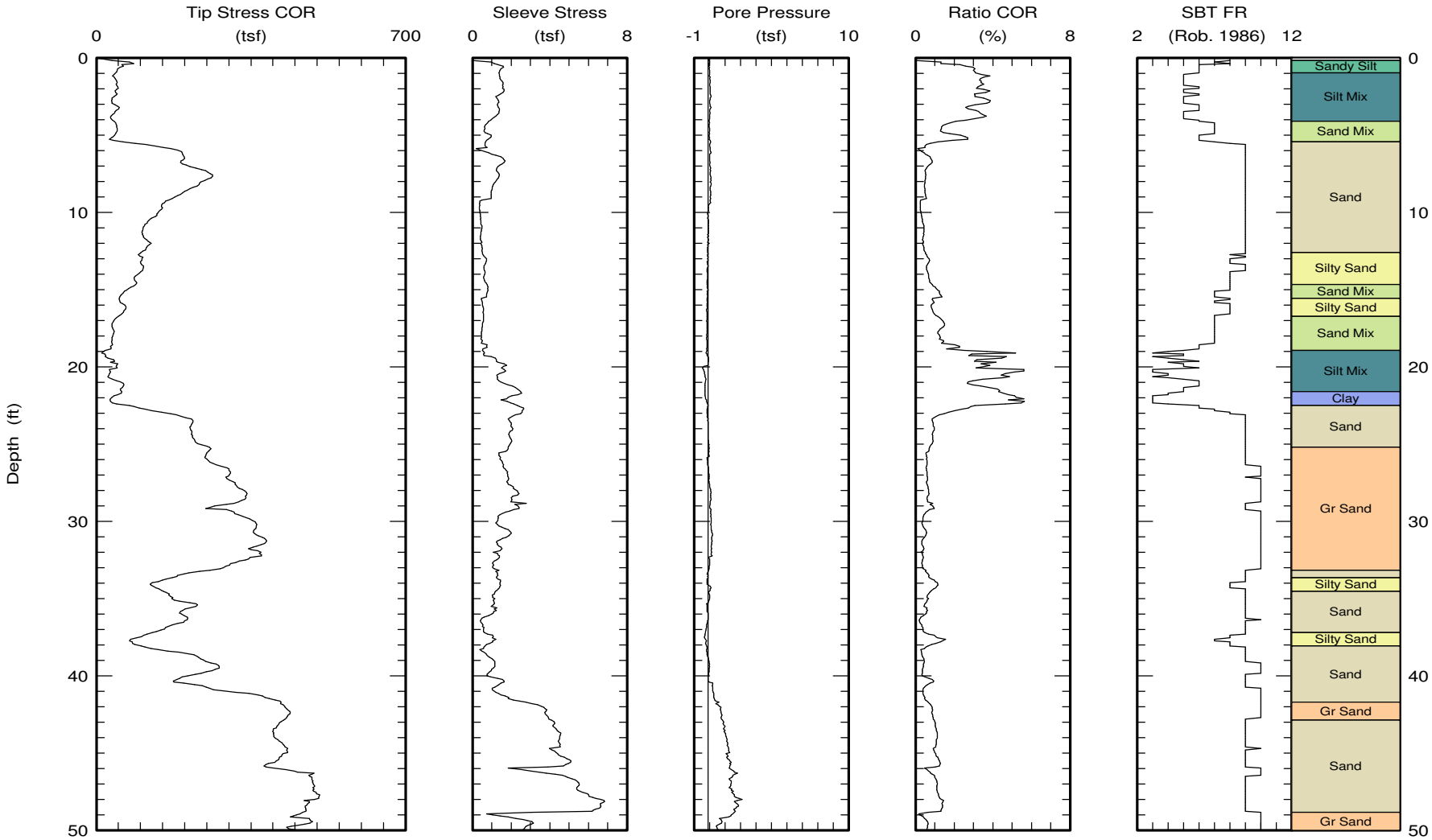
Maximum depth: 60.09 (ft)

Kehoe Testing & Engineering
Office: (714) 901-7270
Fax: (714) 901-7289
rich@kehoetesting.com
skehoe@msn.com

CPT Data
30 ton rig

Date: 06/Mar/2009
Test ID: CPT-4
Project: LongBeach

Customer: MACTEC
Job Site: Belmont Plaza Pool



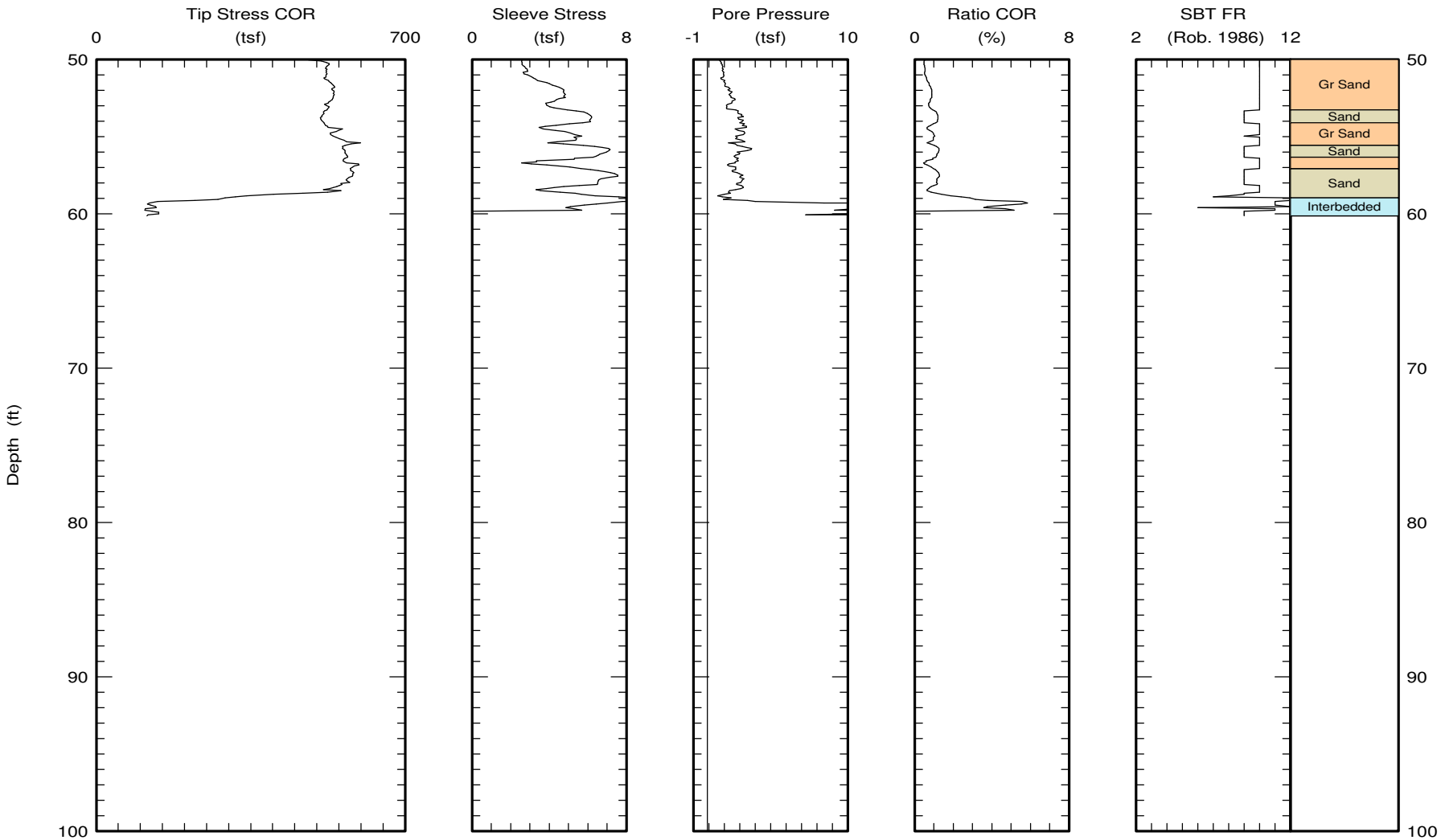
Maximum depth: 60.15 (ft)

Kehoe Testing & Engineering
Office: (714) 901-7270
Fax: (714) 901-7289
rich@kehoetesting.com
skehoe@msn.com

CPT Data
30 ton rig

Date: 06/Mar/2009
Test ID: CPT-4
Project: LongBeach

Customer: MACTEC
Job Site: Belmont Plaza Pool



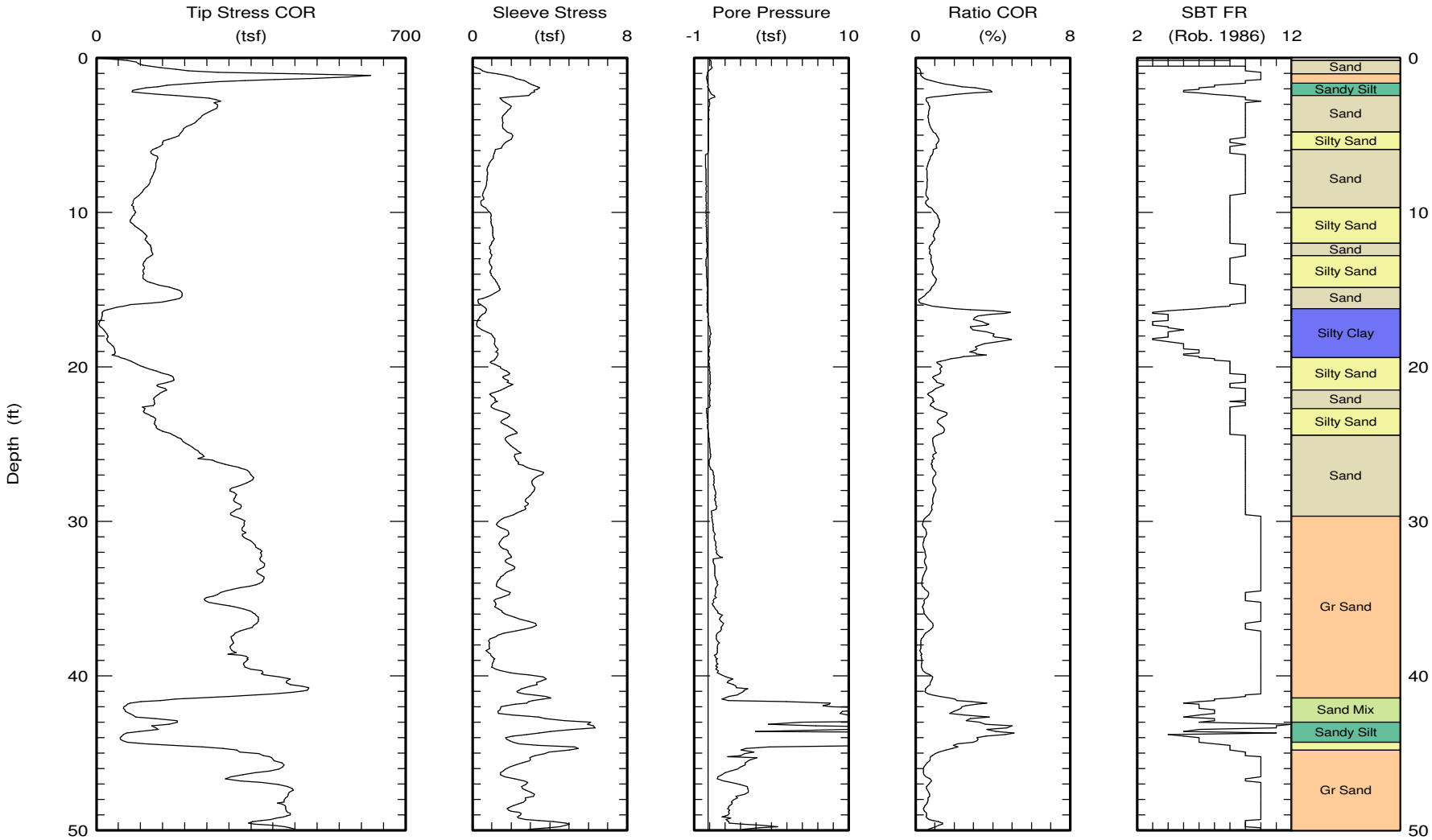
Maximum depth: 60.15 (ft)

Kehoe Testing & Engineering
Office: (714) 901-7270
Fax: (714) 901-7289
rich@kehoetesting.com
skehoe@msn.com

CPT Data
30 ton rig

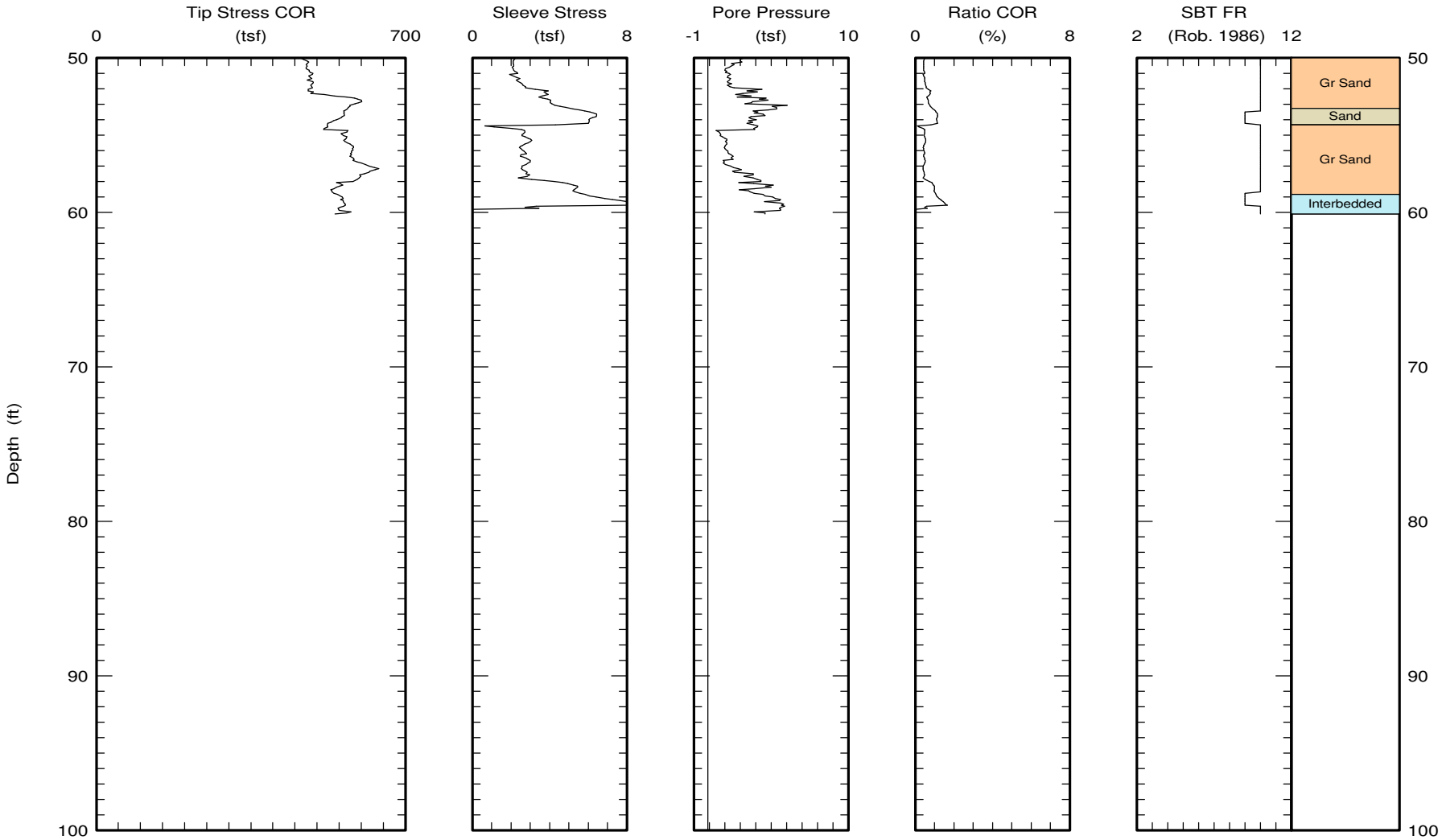
Date: 06/Mar/2009
Test ID: CPT-5
Project: LongBeach

Customer: MACTEC
Job Site: Belmont Plaza Pool



Maximum depth: 60.13 (ft)

Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 rich@kehoetesting.com skehoe@msn.com	CPT Data 30 ton rig	Date: 06/Mar/2009 Test ID: CPT-5 Project: LongBeach
	Customer: MACTEC Job Site: Belmont Plaza Pool	

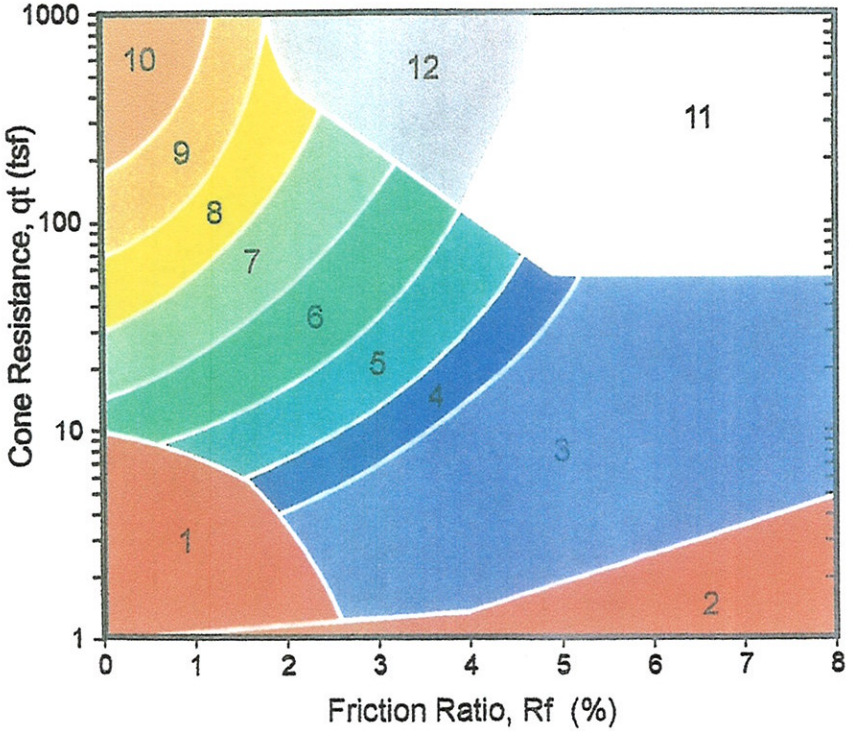


Maximum depth: 60.13 (ft)

KEHOE TESTING & ENGINEERING

CPT Classification Chart

(after Robertson and Campanella, 1988)



Zone	q_t / N	Soil Behavior Type	UCSCS
1	2	sensitive fine grained	OL-OH
2	1	organic material	Pt-OH
3	1	clay	CH
4	1.5	silty clay to clay	CL-CH
5	2	clayey silt to silty clay	ML-CL
6	2.5	sandy silt to clayey silt	MH-ML
7	3	silty sand to sandy silt	SM-ML
8	4	sand to silty sand	SP-SM
9	5	sand	SP
10	6	gravelly sand to sand	SW-SP
11	1	very stiff fine grained *	CL-MH
12	2	sand to clayey sand *	SP-SC

* overconsolidated or cemented

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" Depth (feet)	Qc(avg) (TSF)	Fs(avg) (TSF)	Rf (%)	Rf Zone (zone #)	Spt N (blow/ft)	Spt N1 (blow/ft)	Su (TSF)
0.500	41.627	0.719	1.727	7	13	20	9E9
1.500	36.510	0.632	1.731	6	14	21	9E9
2.500	170.782	0.709	0.415	9	33	50	9E9
3.500	242.150	1.565	0.646	9	46	69	9E9
4.500	229.550	2.460	1.072	9	44	66	9E9
5.500	149.770	1.548	1.034	9	29	44	9E9
6.500	98.075	1.066	1.087	8	23	35	9E9
7.500	39.311	0.854	2.174	6	15	23	9E9
8.500	186.822	0.713	0.382	9	36	54	9E9
9.500	252.190	0.733	0.291	10	40	60	9E9
10.500	212.311	1.151	0.542	9	41	62	9E9
11.500	167.422	0.929	0.555	9	32	47	9E9
12.500	80.433	1.053	1.310	8	19	27	9E9
13.500	40.863	1.224	2.995	6	16	22	9E9
14.500	56.050	2.215	3.948	5	27	35	3.680
15.500	54.833	2.714	4.943	4	35	44	3.596
16.500	41.450	2.326	5.602	3	40	49	2.699
17.500	31.262	1.682	5.373	3	30	36	2.014
18.500	46.090	2.194	4.753	4	29	34	3.000
19.500	52.117	1.990	3.814	5	25	28	3.397
20.500	68.031	2.441	3.585	5	33	36	4.454
21.500	57.171	1.957	3.420	5	27	29	3.725
22.500	107.269	2.105	1.962	7	34	35	9E9
23.500	163.879	1.731	1.056	9	31	31	9E9
24.500	201.880	2.131	1.055	9	39	39	9E9
25.500	218.233	1.907	0.874	9	42	41	9E9
26.500	315.011	2.146	0.681	10	50	47	9E9
27.500	328.078	2.634	0.803	9	63	58	9E9
28.500	310.222	2.409	0.776	9	59	53	9E9
29.500	331.589	2.349	0.708	10	53	47	9E9
30.500	335.650	2.120	0.631	10	54	47	9E9
31.500	303.450	2.393	0.788	9	58	49	9E9
32.500	340.811	2.683	0.787	10	54	45	9E9
33.500	296.540	2.804	0.945	9	57	46	9E9
34.500	226.060	3.774	1.669	8	54	43	9E9
35.500	328.278	2.769	0.843	9	63	50	9E9
36.500	329.389	1.057	0.321	10	53	41	9E9
37.500	312.980	1.055	0.337	10	50	38	9E9
38.500	265.100	0.950	0.358	10	42	31	9E9
39.500	240.182	0.764	0.318	10	38	28	9E9
40.500	153.300	1.954	1.274	8	37	27	9E9
41.499	407.318	3.386	0.831	10	65	46	9E9
42.499	427.325	3.401	0.796	10	68	47	9E9
43.499	213.017	3.194	1.498	8	51	35	9E9
44.499	173.418	2.530	1.455	8	42	28	9E9
45.499	211.354	4.438	2.095	7	68	45	9E9
46.499	335.733	2.407	0.716	10	54	35	9E9
47.499	364.630	2.540	0.696	10	58	38	9E9
48.499	422.330	3.408	0.806	10	67	43	9E9
49.499	539.492	3.132	0.580	10	86	54	9E9

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" Depth (feet)	Qc(avg) (TSF)	Fs(avg) (TSF)	Rf (%)	Rf Zone (zone #)	Spt N (blow/ft)	Spt N1 (blow/ft)	Su (TSF)
50.499	562.885	5.473	0.971	10	90	56	9E9
51.499	521.085	5.129	0.984	9	100	61	9E9
52.499	527.917	5.698	1.079	9	101	61	9E9
53.499	492.575	4.417	0.896	10	79	47	9E9
54.499	436.967	2.344	0.536	10	70	41	9E9
55.499	345.727	1.995	0.577	10	55	32	9E9
56.499	293.290	1.561	0.532	10	47	27	9E9
57.499	243.478	0.682	0.280	10	39	22	9E9
58.499	152.133	0.404	0.266	9	29	16	9E9
59.499	71.000	0.242	0.341	8	17	9	9E9
60.499	106.450	0.000	0.000	9	9E9	9E9	9E9

INPUT FILE: C:\temp\CPT-2.CSV

" Depth (feet)	Qc(avg) (TSF)	Fs(avg) (TSF)	Rf (%)	Rf Zone (zone #)	Spt N (blow/ft)	Spt N1 (blow/ft)	Su (TSF)
0.500	29.950	0.241	0.805	7	10	15	9E9
1.500	19.800	0.237	1.198	6	8	12	9E9
2.500	51.656	0.693	1.342	7	16	24	9E9
3.500	81.280	1.879	2.308	7	26	39	9E9
4.500	231.720	2.515	1.084	9	44	66	9E9
5.500	418.609	3.908	0.933	9	80	120	9E9
6.500	269.620	2.169	0.804	9	52	78	9E9
7.500	178.600	1.398	0.782	9	34	51	9E9
8.500	197.340	1.152	0.583	9	38	57	9E9
9.500	249.978	1.347	0.538	9	48	72	9E9
10.500	323.544	1.161	0.359	10	52	78	9E9
11.500	329.011	1.254	0.381	10	53	78	9E9
12.500	215.060	0.911	0.423	9	41	58	9E9
13.500	37.650	0.877	2.325	6	14	19	9E9
14.500	20.436	0.463	2.213	6	8	11	9E9
15.500	21.162	0.625	2.899	5	10	13	1.374
16.500	22.627	0.676	2.956	5	11	14	1.457
17.500	10.392	0.195	1.840	5	5	6	0.634
18.500	5.191	0.117	2.186	4	3	4	0.281
19.500	8.400	0.327	3.834	3	8	9	0.489
20.500	27.143	0.963	3.511	5	13	14	1.743
21.500	80.542	3.520	4.335	5	39	42	5.325
22.500	90.625	4.093	4.498	11	87	91	9E9
23.500	85.877	2.869	3.331	6	33	34	9E9
24.500	88.300	2.991	3.387	6	34	34	9E9
25.500	70.885	2.959	4.172	5	34	33	4.622
26.500	81.645	3.713	4.547	11	78	74	9E9
27.500	68.050	2.527	3.710	5	33	31	4.426
28.500	35.429	1.392	3.920	5	17	15	2.249
29.500	45.364	1.780	3.915	5	22	20	2.908
30.500	95.492	2.347	2.453	7	31	27	9E9
31.500	254.540	2.081	0.817	9	49	42	9E9
32.500	299.700	1.651	0.551	10	48	40	9E9
33.500	332.030	1.434	0.432	10	53	43	9E9
34.500	321.600	1.393	0.433	10	51	41	9E9
35.500	304.130	1.461	0.480	10	49	39	9E9
36.500	307.289	0.993	0.323	10	49	38	9E9
37.500	151.540	0.680	0.448	9	29	22	9E9
38.500	78.833	1.975	2.497	6	30	22	9E9
39.500	318.075	1.355	0.426	10	51	37	9E9
40.500	162.023	2.410	1.484	8	39	28	9E9
41.499	232.909	1.923	0.824	9	45	32	9E9
42.499	59.870	1.707	2.786	6	23	16	9E9
43.499	45.469	1.272	2.621	6	19	13	9E9
44.499	262.067	1.642	0.622	9	51	35	9E9
45.499	438.654	2.314	0.527	10	70	47	9E9
46.499	267.700	4.516	1.679	8	64	42	9E9
47.499	140.300	3.322	2.302	7	46	30	9E9
48.499	392.254	4.806	1.222	9	75	48	9E9

49.499	472.167	4.971	1.052	9	90	57	9E9

INPUT FILE: C:\temp\CPT-2.CSV

" Depth (feet)	Qc(avg) (TSF)	Fs(avg) (TSF)	Rf (%)	Rf Zone (zone #)	Spt N (blow/ft)	Spt N1 (blow/ft)	Su (TSF)
50.499	515.379	5.311	1.030	9	99	62	9E9
51.499	549.562	2.900	0.527	10	88	54	9E9
52.499	542.943	2.359	0.434	10	87	53	9E9
53.499	586.057	2.730	0.466	10	94	56	9E9
54.499	521.921	2.251	0.431	10	83	49	9E9
55.499	312.369	1.536	0.492	10	50	29	9E9
56.499	167.423	3.010	1.794	8	40	23	9E9
57.499	339.179	1.393	0.410	10	54	31	9E9
58.499	379.108	1.166	0.307	10	61	34	9E9
59.499	486.350	1.127	0.232	10	78	43	9E9
60.499	525.550	0.000	0.000	10	9E9	9E9	9E9

INPUT FILE: C:\temp\CPT-3.CSV

" Depth (feet)	Qc(avg) (TSF)	Fs(avg) (TSF)	Rf (%)	Rf Zone (zone #)	Spt N (blow/ft)	Spt N1 (blow/ft)	Su (TSF)
0.500	75.411	0.569	0.754	8	18	27	9E9
1.500	73.756	0.866	1.173	8	18	27	9E9
2.500	62.678	0.583	0.931	8	15	23	9E9
3.500	63.011	0.548	0.869	8	15	23	9E9
4.500	76.030	0.555	0.730	8	18	27	9E9
5.500	106.667	1.202	1.127	8	26	39	9E9
6.500	134.210	1.383	1.031	8	32	48	9E9
7.500	171.133	1.132	0.662	9	33	50	9E9
8.500	173.070	1.055	0.610	9	33	50	9E9
9.500	110.110	0.527	0.479	9	21	32	9E9
10.500	12.289	0.380	3.103	4	8	12	0.773
11.500	5.390	0.090	1.648	1	3	4	0.316
12.500	8.010	0.281	3.499	3	8	11	0.484
13.500	8.244	0.418	5.095	3	8	11	0.491
14.500	5.730	0.358	6.270	3	5	7	0.321
15.500	9.333	0.544	5.861	3	9	12	0.556
16.500	13.510	0.666	4.941	3	13	16	0.831
17.500	30.290	1.401	4.607	4	19	23	1.956
18.500	34.156	1.210	3.489	5	17	20	2.236
19.500	41.436	2.048	4.855	4	27	31	2.733
20.500	58.889	2.621	4.418	4	38	42	3.871
21.500	85.170	2.552	2.985	6	33	36	9E9
22.500	60.920	2.595	4.237	5	29	31	3.990
23.500	90.870	2.485	2.728	6	35	36	9E9
24.500	152.420	2.302	1.509	8	37	37	9E9
25.500	170.320	2.339	1.373	8	41	40	9E9
26.500	248.964	2.947	1.184	9	48	46	9E9
27.500	287.192	3.495	1.217	9	55	52	9E9
28.500	297.479	3.256	1.094	9	57	52	9E9
29.500	311.677	2.832	0.908	9	60	54	9E9
30.500	300.392	1.863	0.620	10	48	42	9E9
31.500	298.167	1.610	0.540	10	48	41	9E9
32.500	332.045	1.080	0.325	10	53	45	9E9
33.500	310.156	1.902	0.613	10	50	41	9E9
34.500	243.650	1.634	0.670	9	47	38	9E9
35.500	228.390	1.130	0.495	9	44	35	9E9
36.500	67.287	1.268	1.848	7	22	17	9E9
37.500	28.378	0.627	2.133	6	11	8	9E9
38.500	33.678	0.644	1.854	6	13	10	9E9
39.500	37.190	0.949	2.446	6	15	11	9E9
40.500	43.600	1.507	3.232	5	22	16	2.939
41.499	82.880	2.437	2.880	6	32	23	9E9
42.499	313.778	3.582	1.141	9	60	42	9E9
43.499	361.010	2.328	0.644	10	58	40	9E9
44.499	273.218	4.868	1.778	8	66	45	9E9
45.499	364.433	6.027	1.649	8	88	59	9E9
46.499	534.293	4.295	0.803	10	85	57	9E9
47.499	489.279	5.385	1.100	9	94	62	9E9
48.499	415.164	4.449	1.070	9	80	52	9E9
49.499	492.240	5.888	1.195	9	94	60	9E9

INPUT FILE: C:\temp\CPT-3.CSV

" Depth (feet)	Qc(avg) (TSF)	Fs(avg) (TSF)	Rf (%)	Rf Zone (zone #)	Spt N (blow/ft)	Spt N1 (blow/ft)	Su (TSF)
50.499	485.569	5.441	1.119	9	93	59	9E9
51.499	434.450	5.741	1.320	9	83	52	9E9
52.499	278.277	4.566	1.639	8	67	41	9E9
53.499	295.777	3.425	1.156	9	57	35	9E9
54.499	392.500	1.463	0.373	10	63	38	9E9
55.499	291.893	2.097	0.718	9	56	33	9E9
56.499	350.992	2.187	0.623	10	56	33	9E9
57.499	462.586	2.557	0.552	10	74	43	9E9
58.499	476.269	4.361	0.914	10	76	43	9E9
59.499	436.436	4.076	0.932	9	84	47	9E9
60.499	406.150	0.000	0.000	10	9E9	9E9	9E9

INPUT FILE: C:\temp\CPT-4.CSV

" Depth (feet)	Qc(avg) (TSF)	Fs(avg) (TSF)	Rf (%)	Rf Zone (zone #)	Spt N (blow/ft)	Spt N1 (blow/ft)	Su (TSF)
0.500	50.355	1.120	2.223	6	19	29	9E9
1.500	44.191	1.506	3.408	5	21	32	2.940
2.500	40.127	1.411	3.514	5	19	29	2.666
3.500	41.618	1.289	3.095	5	20	30	2.762
4.500	44.409	0.715	1.610	7	14	21	9E9
5.500	89.908	0.706	0.785	8	22	33	9E9
6.500	196.027	1.334	0.680	9	38	57	9E9
7.500	247.209	1.313	0.531	9	47	71	9E9
8.500	212.691	1.035	0.487	9	41	62	9E9
9.500	154.442	0.481	0.311	9	30	45	9E9
10.500	122.525	0.433	0.354	9	23	35	9E9
11.500	108.558	0.448	0.413	9	21	31	9E9
12.500	108.525	0.519	0.478	9	21	30	9E9
13.500	102.855	0.655	0.637	8	25	34	9E9
14.500	85.982	0.684	0.795	8	21	28	9E9
15.500	57.391	0.636	1.109	7	18	23	9E9
16.500	57.709	0.558	0.968	7	18	22	9E9
17.500	37.745	0.503	1.333	7	12	14	9E9
18.500	33.736	0.561	1.663	6	13	15	9E9
19.500	30.458	1.110	3.647	5	15	17	1.948
20.500	35.564	1.443	4.063	4	23	25	2.282
21.500	53.945	2.166	4.020	5	26	28	3.504
22.500	74.355	2.216	2.981	6	28	29	9E9
23.500	205.209	2.043	0.995	9	39	40	9E9
24.500	218.883	1.977	0.903	9	42	42	9E9
25.500	250.233	1.618	0.647	9	48	47	9E9
26.500	284.909	1.665	0.585	10	45	43	9E9
27.500	310.773	1.936	0.623	10	50	46	9E9
28.500	330.027	2.235	0.677	10	53	48	9E9
29.500	307.583	1.773	0.576	10	49	43	9E9
30.500	360.367	1.658	0.460	10	58	50	9E9
31.500	370.433	1.431	0.386	10	59	50	9E9
32.500	338.200	1.204	0.356	10	54	45	9E9
33.500	198.692	1.292	0.650	9	38	31	9E9
34.500	148.492	1.241	0.836	9	28	22	9E9
35.500	201.217	1.123	0.558	9	39	31	9E9
36.500	186.625	0.579	0.310	9	36	28	9E9
37.500	102.517	0.888	0.867	8	25	19	9E9
38.500	182.808	0.713	0.390	9	35	26	9E9
39.500	259.609	1.025	0.395	10	41	30	9E9
40.500	218.300	1.259	0.577	9	42	30	9E9
41.499	390.158	2.233	0.572	10	62	44	9E9
42.499	431.108	3.878	0.899	9	83	58	9E9
43.499	404.917	4.375	1.080	9	78	54	9E9
44.499	423.282	4.384	1.035	9	81	55	9E9
45.499	403.831	4.508	1.115	9	77	52	9E9
46.499	477.662	4.486	0.939	9	92	61	9E9
47.499	496.400	5.737	1.155	9	95	62	9E9
48.499	475.229	5.912	1.243	9	91	58	9E9
49.499	460.779	2.501	0.542	10	74	47	9E9

INPUT FILE: C:\temp\CPT-4.CSV

" Depth (feet)	Qc(avg) (TSF)	Fs(avg) (TSF)	Rf (%)	Rf Zone (zone #)	Spt N (blow/ft)	Spt N1 (blow/ft)	Su (TSF)
50.499	518.662	2.701	0.521	10	83	52	9E9
51.499	528.664	3.778	0.714	10	84	52	9E9
52.499	532.393	4.452	0.836	10	85	52	9E9
53.499	516.262	5.526	1.070	9	99	59	9E9
54.499	530.117	4.794	0.904	10	85	50	9E9
55.499	563.642	5.787	1.026	9	108	63	9E9
56.499	571.180	5.073	0.888	10	91	52	9E9
57.499	575.877	6.798	1.179	9	110	63	9E9
58.499	477.307	5.444	1.140	9	92	52	9E9
59.499	144.592	5.432	3.700	12	70	39	9E9
60.499	121.733	0.000	0.000	9	9E9	9E9	9E9

INPUT FILE: C:\temp\CPT-5.CSV

" Depth (feet)	Qc(avg) (TSF)	Fs(avg) (TSF)	Rf (%)	Rf Zone (zone #)	Spt N (blow/ft)	Spt N1 (blow/ft)	Su (TSF)
0.500	129.490	0.188	0.145	9	25	38	9E9
1.500	332.325	2.606	0.784	9	64	96	9E9
2.500	180.382	2.435	1.349	8	43	65	9E9
3.500	254.733	1.762	0.692	9	49	74	9E9
4.500	205.863	1.669	0.811	9	39	59	9E9
5.500	154.133	1.730	1.122	8	37	56	9E9
6.500	132.967	1.021	0.768	9	25	38	9E9
7.500	128.925	0.772	0.599	9	25	38	9E9
8.500	109.637	0.640	0.584	9	21	32	9E9
9.500	84.860	0.611	0.720	8	20	30	9E9
10.500	82.864	0.967	1.167	8	20	30	9E9
11.500	109.023	1.059	0.972	8	26	38	9E9
12.500	123.292	0.945	0.767	9	24	34	9E9
13.500	107.900	0.927	0.859	8	26	35	9E9
14.500	127.758	1.223	0.958	8	31	41	9E9
15.500	171.423	0.768	0.448	9	33	42	9E9
16.500	22.950	0.553	2.407	6	9	11	9E9
17.500	11.570	0.402	3.466	4	7	8	0.700
18.500	31.820	1.178	3.695	5	15	17	2.048
19.500	65.085	1.184	1.819	7	21	24	9E9
20.500	150.000	1.701	1.134	8	36	40	9E9
21.500	145.740	1.465	1.005	9	28	30	9E9
22.500	122.038	1.217	0.997	8	29	30	9E9
23.500	130.129	1.744	1.340	8	31	32	9E9
24.500	173.323	1.978	1.141	9	33	33	9E9
25.500	228.800	2.215	0.968	9	44	43	9E9
26.500	308.892	2.812	0.910	9	59	56	9E9
27.500	334.744	3.184	0.951	9	64	59	9E9
28.500	314.360	2.923	0.930	9	60	54	9E9
29.500	318.730	2.159	0.677	10	51	45	9E9
30.500	332.744	1.597	0.480	10	53	46	9E9
31.500	356.450	1.549	0.434	10	57	48	9E9
32.500	374.900	1.915	0.511	10	60	50	9E9
33.500	372.611	1.641	0.440	10	60	49	9E9
34.500	307.570	1.546	0.502	10	49	39	9E9
35.500	300.664	1.295	0.431	10	48	38	9E9
36.500	359.233	2.606	0.725	10	57	44	9E9
37.500	310.987	1.241	0.399	10	50	38	9E9
38.500	322.530	0.953	0.295	10	52	39	9E9
39.500	351.618	1.468	0.417	10	56	41	9E9
40.500	450.589	3.153	0.699	10	72	52	9E9
41.499	235.767	2.902	1.227	9	45	32	9E9
42.499	88.789	2.340	2.573	6	35	24	9E9
43.499	112.478	4.701	4.098	11	110	75	9E9
44.499	167.120	3.310	1.950	7	54	36	9E9
45.499	400.511	2.858	0.713	10	64	43	9E9
46.499	348.850	1.950	0.559	10	56	37	9E9
47.499	433.113	2.848	0.657	10	69	44	9E9
48.499	427.142	2.237	0.523	10	68	43	9E9
49.499	402.885	3.538	0.877	9	77	48	9E9

INPUT FILE: C:\temp\CPT-5.CSV

" Depth (feet)	Qc(avg) (TSF)	Fs(avg) (TSF)	Rf (%)	Rf Zone (zone #)	Spt N (blow/ft)	Spt N1 (blow/ft)	Su (TSF)
50.499	476.733	2.157	0.452	10	76	47	9E9
51.499	487.033	2.423	0.497	10	78	48	9E9
52.499	547.100	3.815	0.696	10	87	52	9E9
53.499	562.446	5.711	1.014	9	108	64	9E9
54.499	537.062	3.502	0.652	10	86	50	9E9
55.499	571.485	2.736	0.479	10	91	53	9E9
56.499	584.258	2.781	0.476	10	93	53	9E9
57.499	610.633	2.803	0.459	10	98	55	9E9
58.499	546.000	5.282	0.966	10	87	48	9E9
59.499	557.314	5.183	0.928	10	89	49	9E9
60.499	553.200	0.000	0.000	10	9E9	9E9	9E9

4000 E. Olympic Plaza
Long Beach, CA

CPT-3

CPT Shear Wave Measurements

Depth (ft)	Travel Distance (ft)	S-Wave Arrival (msec)	S-Wave Velocity from Surface (ft/sec)	Interval S-Wave Velocity (ft/sec)
5.02	7.09	9.17	772.65	
10.09	11.26	15.35	733.61	675.68
15.05	15.86	24.87	637.67	482.97
20.03	20.64	31.48	655.80	724.02
25.01	25.50	37.34	683.05	829.40
30.00	30.41	43.47	699.65	800.80
35.26	35.61	49.95	712.97	802.30
40.10	40.41	55.43	729.04	875.51
45.10	45.38	61.27	740.60	850.31
50.07	50.32	66.40	757.82	963.49
55.01	55.24	70.76	780.62	1127.92
60.00	60.21	75.57	796.72	1033.52

Shear Wave Source Offset = 5 ft

S-Wave Velocity from Surface = Travel Distance/S-Wave Arrival
Interval S-Wave Velocity = (Travel Dist2-Travel Dist1)/(Time2-Time1)

Program: CPTINT - CPT Cone Interpretation Program
Version: 5.2
Table File by: Dr. R. G. (DICK) Campanella, P.Eng.
Rev. Dated: April 3, 2002

Parameter	Methods	Refer. Number	Valid Soil Type	Valid Zone
Depth average see NOTE #1	Depth averaged over specified range (see menu)		All	All
Parameter Averaging	Averaged over range specified for depth. If no values exist, your choice is zero's or no value		All	All
Qc, Tip Stress	measured tip force/area	#6,#8	All	All
Qt corrtd for U2 see NOTE #2	Qt = Qc + (1 - a) x U2 and a = tip area ratio Defaults to U2 if given or uses U1 or U3 times Const.	#6,#8	All	All
[Note: Input value from input file is used if defined, not calculated]				
Q (Qt Normalized)	$Q = \frac{Qt - sv}{sv'}$	#9 & 13	All	All
Fs	measured sleeve force/area	#6,#8	All	All
Rf Friction Ratio (if Rf>8, Rf=8)	$Rf = \frac{Fs}{Qt} \times 100\%$	#6,#8	All	All
F (Rf Normalized)	$F = \frac{Fs}{(Qt - sv)} \times 100\%$	#9 & 13	All	All
Gamma Total Unit Weight (Soil + Water) see NOTE #3	Based on Rf or Bq Classif. Zone Zone # Gamma = kN/m^3 1 Qt<4bar 15.70 1 Qt=4bar 17.30 2 Rf<5% 13.36 2 Rf=5% 11.80 2 Bq Zone 12.58 3 Qt<10bar 18.86 3 Qt=10bar 19.65 4, 5 & 6 Qt<20bar 18.86 4, 5 & 6 Qt=20bar 19.65 7 18.86 8 & 9 19.65 10 20.44 11 & 12 21.22		All	All

Parameter	Methods	Refer. Number	Valid Soil Type	Valid Zone
U Penetration Pore Pressure see NOTE #4	U1, measured on Face of tip U2, measured Behind Tip at shoulder (std location) U3, measured Behind Friction Sleeve		All	All
Water Table	Depth below ground surface to where pore pressure = 0 Make negative if water level is above ground		All	All
Uo Hydrostatic Pore Pressure see NOTE #4	Uo = water depth, Hw x unit weight water, Gamma or Uo=Hw=depth-depth to water table if depth<water table, Uo = 0		All	All
dU Excess Pore Pressure	dU = U2 - Uo Defaults to U2 if given or uses U1 or U3 x const.		All	All
DPPR (Differential Pore Pressure Ratio)	$DPPR = \frac{dU}{Qt} = \frac{U - Uo}{Qt}$ Defaults to U2 if given or uses U1 or U3 x const.	#6, #8	All	All
Bq	$Bq = \frac{dU}{Qt - sv}$	# 4 # 8 # 13	All	All
OS (Overburden Stress)	OS = sv = S (Gamma x Depth)		All	All
EOS (Effective Overburden Stress)	EOS = sv' = OS - Uo = sv - Uo		All	All
Rf Zone Soil Behavior Type see NOTE #5	Classification chart for Qc and Rf Zone # = Soil Behavior Type 1=sensitive fine grained 2=organic material 3=clay 4=silty clay 5=clayey silt 6=sandy silt 7=silty sand 8=fine sand 9=sand 10=gravelly sand 11=very stiff fine grained ¥ 12=sand to clayey sand ¥ ¥ overconsolidated or cemented	#6 #8, Fig4.3	All	1<Qt<1000bar 0<Rf<8%

Parameter	Methods	Refer. Number	Valid Soil Type	Valid Zone
Bq Zone Soil Behavior Type	Classification chart for Qc and Bq (same zone #'s as Rf above)	#8 Fig 4.3	All	0<Qt<1000bar -0.1<Bq<1.4
Spt N(60) Standard Penetration Test (Blows/foot) at 60% Energy After R&C(1983) see NOTE #6	Qt/N ratio per zone Zone # Qt/N Zone # Qt/N 1 2 7 3 2 1 8 4 3 1 9 5 4 1.5 10 6 5 2 11 1 6 2.5 12 2	# 7 #8 Fig 4.2	All	All
Spt N1(60) Normalized for Overburden str	Spt N1(60) = Cn x Spt N(60) where Cn = (sv')^(-0.77)	# 8	All	0.5<Cn<1.5
Dr Relative Density see NOTE #7	Specific Sands: $Dr = \frac{100}{C2} * \ln \left(\frac{Qc}{C1 \cdot C0 \cdot sv'} \right)$ where: All are NC & UNAGED Sand | C0 | C1 | C2 Ticino | 17.37 | .558 | 2.58 Schmertmann | 15.32 | .520 | 2.75 ALL SANDS: NC, OC, ALL TESTS $Dr = C3 + C4 \log_{10} \left(\frac{Qc / C1}{sv' + C2 / C0} \right)$ where: C0 | C1 | C2 | C3 | C4 0.100 | 0.0981 | 0.5 | -98 | 66	# 8 # 1 # 1 # 5	Sand	7 to 10 0<Qt<500bar 0<sv'<5bar 7 to 10 (6 possible)
Phi Friction Angle	Methods: 1) Robertson & Campanella 2) Durgunoglu & Mitchell 3) Janbu beta = +15 degree 4) Janbu beta = 0 degree 5) Janbu beta = -15 degree	#6, #8 # 2 #6, #8 #6, #8 #6, #8	Sand	7 to 10 & 6 0<Qt<500bar 0<sv'<4bar 29<phi<49

Compressibility

moderate
high

all

Parameter	Methods	Refer. Number	Valid Soil Type	Valid Zone
Gmax Maximum Shear Modulus at very small strains	Clay: Gmax = alpha x Qt	# 8 Fig4.18	Clay	1 to 6
	Sand: Digitized figure of Qc vs Gmax with interpolation between sv'curves,R&C method	# 6 # 8 Fig4.13	Sand	(6 possible) 7 to 10 .25<sv'<8bar
CSR(Qc), t/s LEVEL ground Liquefaction SAND Resistance see NOTE #8	Seed's CSR vs N1(60) graph for specified equake Magnitude.Can include silty sand corr. for Zone 7. N1(60) from CPT correlations.	# 11 # 12	Sand	7 to 10 (6 possible)
CSR(Eq), t/s Cyclic Stress Ratio applied by design quake	$CSR(Eq) = 0.65 \frac{Amax}{g} \frac{sv}{svo'} rd$ Amax=max surface acceleratn including Amplification	# 12 # 3	Sand	7 to 10 (6 possible)
[Note: Input value from input file is used if defined, & not calculated]				
rd Reduction Factor to find CSR(Eq)	Digitized graph to use for depth vs rd: 1) Seed's mean 2) Fraser Delta	# 12 # 3	Sand	(6 possible) 7 to 10 0<depth<30m
FL,Safety Factor against Liquefaction	FL = CSR(Qc)/CSR(Eq)	# 3	Sand	7 to 10 (6 possible)
Qcr Critical Bearng required to resist Liquefctn	Qcr backcalculated from CSR(Eq) for a specified FL. Qcr is only for the given GWT,EOS,OS,Amax/g & Eq.Mag	# 12	Sand	7 to 10 (6 possible)
Su, Undrained Shear Strength of CLAY METHODS:	Nk: $Su = \frac{Qc - st}{Nk}$	# 8	Clay	1 to 6
	Nke: $Su = \frac{Qt - U2}{Nke}$		Clay	1 to 6
	Nkt: $Su = \frac{Qt - sv}{Nkt}$		Clay	1 to 6
	Nc: $Su = \frac{Qt}{Nc}$		Clay	1 to 6
see NOTE #9	NdU: $Su = \frac{dU2 \text{ (dU1 or dU3)}}{NdU}$		Clay	1 to 6

Parameter	Methods	Refer. Number	Valid Soil Type	Valid Zone
Su/EOS	$Su/EOS = \frac{Su}{sv'}$	# 8	Clay	1 to 6
Ko (NC) Normally Consolidated	$(Ko)NC = 1 - Sin(f)$ see NOTE #10	# 8	Sand	7 to 10 (6 possible)
Ko (OC) Over Consolidated	$(Ko)OC = (Ko)NC \times OCR^{0.42}$	# 8	Sand	7 to 10 (6 possible)
E25 Youngs Modulus	E25 = alpha x Qt where user input alpha	# 8 4.11&12	Sand	(6) 7 to 10 0<Qt<500bar
M Constrained Modulus	CLAY: M = alpha x Qt where user input alpha	# 8 Tabl4.3	Clay	1 to 6
	SAND: Methods: Qt: $M = alpha \times Qt$	# 8 Fig4.10	Sand	7 to 10 (6 possible)
	Baldi: $\frac{M}{Qt} = C0 \times pa \left[\frac{sv' + C1}{pa}\right] \times OCR^{C2} \times \exp(C3\ Dr)$		Sand	7 to 10
OCR (Clay) Over-Consolidation Ratio see NOTE #11	$OCR = \left[\frac{\frac{Su}{svo'}}{\left[\frac{Su}{svo'} + NC\right]}\right]^{+1.25}$	# 6 # 8 Fig4.19	Clay	1 to 6
Ic Material Index After J&D(1993) see NOTE #18	$Ic = \left[3 - \log_{10}(Q(1-Bq))\right]^{2} + \left[1.5 + 1.3\log_{10} F\right]^{2+0.5}$	# 13 # 17	All	All
Spt N(60) Standard Penetration Test (Blows/foot) at 60% Energy After J&D(1993) see NOTE #16	$Qc/N = 8.5(1-(Ic/4.75))$ where Qc in bars	# 13	All	All

Parameter	Methods	Refer. Number	Valid Soil Type	Valid Zone
State Parameter State, (e-units) Current Void Void Ratio minus Critical Void Ratio	$\text{State} = \ln \left[\frac{3M + 8.5M/F}{Q(1-Bq)} \right] \Big/ (11.9 - 1.33F)$ $M = \frac{6 \sin fcv}{3 - \sin fcv}$ fcv = const. vol. Phi angle	# 14	All	All
Fines Content FC(%) Percent less than #200 Sieve After Davies,99	FC(%) = 42.4179(Ic) - 54.8574 FC(%) = 0% if Ic < 1.2933 FC(%) = 100% if Ic > 3.6508	# 15	All	All
OCR (Clay) Overcons. Ratio by Pore Press. U1 & U2 or U1 & U3 see NOTE #17	OCR = 0.5 + 1.50(PPD) PPD = (U1 - U2)/Uo or PPD = (U1 - U3)/Uo and default 0.5 & 1.5 are settable	# 16	Clay	1 to 6

1. Depth averaging may be in 0.5, 1, 2.5 or 5 ft. intervals or 0.1, 0.25, 0.5 or 1.0 m intervals, or no depth averaging if zero is selected. The average is the mean value of the readings in the interval. The depth value is the mid-depth of the averaged interval. It is convenient to start at half the depth averaging interval. For example, if you want "even" depths and the depth averaging is set at 0.50 m then start at 0.25 to get values of depth of 0.5, 1.0, 1.5, etc.

2. Basic input CPTU data columns are for Depth, Qc, Fs, U1, U2, U3, INC and TEMP may be selected. In addition the following parameters may also be specified as an INPUT data column: Qt, Gamma, Uo, Spt N, Rf Zone, Bq Zone and CSR(EQ). These values will be used where required to obtain other interpreted parameters. If they are not specified the program will estimate them when they are required. For example, you can create an OUTPUT data file of any of the above parameters and then edit some or all of the values to suite your measurements or your desires to specify their values. You can do that with "Gamma" values to input your measurements of unit weight, or with "Uo" if you want to input values of pore water pressure other than hydrostatic, or with any of the other input parameters. You would use your edited file of adjusted data as your new INPUT data file. Thus, you can specify these parameters if you want to override the Program's values.

You can also use the designated value of "9E9" to denote an unknown value.

You can use the "OTHER" designation to input other data that exists on your input file and identify its units. This allows you to output it, without operating on it, if you choose.

It is best NOT to use depth averaging when using input data that is not continuous at regular depth intervals. Always use DEPTH AVERAGING with extreme caution since the program averages ALL INPUT parameters over the interval chosen irregardless of soil type. Careful use of start and end depth choises can make depth averaging very effective.

3. Since there is no data in the file within the initial depth interval, a default Gamma (unit weight) must be specified from the surface to the starting depth. This is done in the "Param" Menu in units of kN/m^3 (1kN/m^3=6.36pcf). Also, you can specify the values of Gamma to be used by the program as in NOTE #2 above.

4. If pore pressures are not measured by the cone then the program will take Qc as being equal to Qt for all interpretations requiring Qt. Also, Uo may be specified in the input file as a column of Uo vs depth values, if the water pressures are not hydrostatic. See NOTE #2 for more info on customizing input data.

5. You can choose to use either the Rf classif. Zone or the Bq classif. Zone to divide soil into Undrained Parameters (Zones 1 to 6) and Drained Parameters (Zones 7 to 10) in the "Param" Menu. (However, in order to use the Bq Zone you must have Pore Pressure, U2, data.) Also, you may choose to switch Zone 6 to a Drained Zone from its Undrained Zone status. This is done if you feel that the soil identified as Zone 6 (sandy silt) is really coaser (using other sources of information) and/or you want it analyzed as a Drained rather than Undrained soil. Finally, the soil behavior names in each zone were shortened in version 5.0 for simplicity. For example, Zone 6 was named "sandy silt to clayey silt" but was shortened to "sandy silt".

6. Spt N is the same as Spt N(60) for 60% transferred energy. This value is calculated from the Qt/N ratios given for each Soil Zone (you can specify either Rf or Bq Zone) and these values are used in the Level Ground Liquefaction analysis. Values of Spt N may be specified in the Input File, if indepedently measured values are to be used. We suggest that you not use depth averaging if you only have selected Spt N values at a few depths. You may use "9E9" for missing data.

7. If Dr values are negative then soil is very loose or likely more of an undrained soil like a silty sand rather than a drained soil for which the Dr correlations were developed. Use Dr interpretations very cautiously since they also assume the soil is free draining, uncemented, unaged and has the same compressibility of grains as the soil used for the correlations in chamber calibration tests.

8. The simplified sand liquefaction analysis for level ground according to Seed et al requires Spt N1(60) and earthquake magnitude to obtain the cyclic stress ratio to cause liquefaction, CSR(Qc). The design maximum ground acceleration, the depth-reduction factor, Rd, and overburden total and effective stresses are required to calculate the cyclic stress ratio applied by the design earthquake, CSR(EQ). The program estimates the N1(60) values from the cone stresses, the operator identifies the earthquake magnitude and Seed et al chart is used to get CSR(Qc). The program also calculates CSR(EQ) from the user specified maximum ground acceleration including any amplification factors, the calculated overburden stresses and either Seed's mean or the Fraser Delta Rd factor. The Fraser Delta is used only when amplification factors of the order of 2 or more are used. See Reference Nos. 3, 6, 11 and 12 for more information. The user can INPUT specific values for Spt N, CSR(EQ), Soil Zones, Gamma's, etc. in order to customize the analysis for the existing data base of information. It is recommended that you do not use depth averaging when using specific input data but make calculations at specific depths where external input data exists. The calculated value of Qcr is the minimum value of cone bearing stress required at a given depth such that the factor of safety against liquefaction, or the ratio FL = CSR(Qc)/CSR(EQ) have the specified value for a given earthquake magnitude, max. ground acceleration, depth reduction factor, and calculated overburden stresses. This value of Qcr is useful to identify the required minimum level of soil improvement for a given design condition.

9. The NdU method to calculate undrained shear strength has been extended to allow the user to choose either dU1, or dU2 or dU3 provided such pore pressure measurements exist.

10. The Overconsolidation Ratio, OCR, for the sand must be estimated by the user in the "Param" menu if you want to estimate Ko in the sand layers. For the typical normally consolidated sand, OCR = 1.0.

11. It is currently only possible to estimate the OCR for a clay, which makes use of the correlations obtained from extensive laboratory tests.

12. An improved calculation and print routine was added to version 5.0 which uses swap routines to reduce memory requirements, but slows down the calculations.

13. The classification charts for Rf has been extended at all boundaries such that values of Rf>8 and values of Qc<1.00 are possible. The Bq classification chart which requires dU2 and can now accept values of Bq>1.2 and Qt<1. Unfortunately, this feature does not work.

14. Version 5.1ppd added several enhancements to the program. You may input an average vertical flow gradient, which is applied over the entire profile depth to be analysed so adjust the depth of interest accordingly. Zero gives hydrostatic and no flow, a negative gradient is upward flow which increases pore pressure and reduces vertical effective stress. A positive gradient gives downward flow.

15. A State Parameter or current void ratio minus critical void ratio is calculated according to the paper by Ref. 14, Plewes, Davies and Jefferies, 1994.

16. An alternate method to estimate SPT from CPT is provided according to Ref. 13, Jefferies and Davies, 1993 in ASTM.

17. An alternate method to estimate OCR in clays is provided which uses the measured pore pressure difference, ppd, so both U1 and U2 or U1 and U3 must be measured at the same time. (see Ref. 16)

18. Version 5.2 added the value Ic (Material Index) according to Jefferies & Davies, 1993, 1991 (Ref. 13 & 17) which combines all Normalized parameters Q, F and Bq.
(Note: QtN was changed to Q and RfN to F.)

18A. In Version 5.2, if at any depth the value of Bq>1 (in very sensitive saturated soil)then Bq is made equal to 0.99. Also, if Rf>8 it is made 7.99. These changes have a negligable effect on the results.

19. FC(%) or percent of dry weight less than #200 sieve (.074mm) was also added according to Davies, 1999 Ref.#15)

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APPENDIX C

CORROSION STUDY

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March 24, 2009

via email: mtsai@mactec.com

Reviewed and acceptable for use
NAME: [illegible] DATE: 4/14/2009
MACTEC Engineering and Consulting, Inc.

MACTEC
5628 East Slanson Avenue
Los Angeles, CA 90040

Attention: Mr. Meng-Kang Tsai, P.E.

Re: Soil Corrosivity Study
Long Beach Belmont Plaza
Long Beach, CA
SA #09-0201SCS, M #4953-09-0301

INTRODUCTION

Laboratory tests have been completed on two soil samples provided for the referenced project. The purpose of these tests was to determine if the soils might have deleterious effects on underground utility piping and concrete structures. Schiff Associates assumes that the samples provided are representative of the most corrosive soils at the site.

The proposed construction is a swimming pool and building. The site is located at 4000 Olympic Plaza in Long Beach, CA and the water table is reportedly six feet deep.

The scope of this study is limited to a determination of soil corrosivity and general corrosion control recommendations for materials likely to be used for construction. Our recommendations do not constitute, and are not meant as a substitute for, design documents for the purpose of construction. If the architects and/or engineers desire more specific information, designs, specifications, or review of design, Schiff Associates will be happy to work with them as a separate phase of this project.

LABORATORY SOIL CORROSIVITY TESTS

The electrical resistivity of each sample was measured in a soil box per ASTM G187 in its as-received condition and again after saturation with distilled water. Resistivities are at about their lowest value when the soil is saturated. The pH of the saturated samples was measured per CTM 643. A 5:1 water:soil extract from each sample was chemically analyzed for the major soluble salts commonly found in soil per ASTM D4327 and D513. Test results are shown in Table 1.

431 West Baseline Road · Claremont, CA 91711
Phone: 909.626.0967 · Fax: 909.626.3316

SOIL CORROSIVITY

A major factor in determining soil corrosivity is electrical resistivity. The electrical resistivity of a soil is a measure of its resistance to the flow of electrical current. Corrosion of buried metal is an electrochemical process in which the amount of metal loss due to corrosion is directly proportional to the flow of electrical current (DC) from the metal into the soil. Corrosion currents, following Ohm's Law, are inversely proportional to soil resistivity. Lower electrical resistivities result from higher moisture and soluble salt contents and indicate corrosive soil.

A correlation between electrical resistivity and corrosivity toward ferrous metals is:[1]

Soil Resistivity in ohm-centimeters	Corrosivity Category
Greater than 10,000	Mildly Corrosive
2,000 to 10,000	Moderately Corrosive
1,000 to 2,000	Corrosive
0 to 1,000	Severely Corrosive

Other soil characteristics that may influence corrosivity towards metals are pH, soluble salt content, soil types, aeration, anaerobic conditions, and site drainage.

Electrical resistivities were in the corrosive and severely corrosive categories with as-received moisture. When saturated, the resistivities remained in the corrosive to severely corrosive categories. Both as-received resistivities were at their saturated values.

Soil pH values varied from 7.9 to 8.3. Both pH values are moderately alkaline.[2] These values do not particularly increase soil corrosivity.

The soluble salt content of the samples ranged from low to moderate.

Nitrate was detected in low concentrations.

Tests were not made for sulfide and negative oxidation-reduction (redox) potential because these samples did not exhibit characteristics typically associated with anaerobic conditions.

This soil is classified as severely corrosive to ferrous metals.

CORROSION CONTROL RECOMMENDATIONS

The life of buried materials depends on thickness, strength, loads, construction details, soil moisture, etc., in addition to soil corrosivity, and is, therefore, difficult to predict. Of more practical value are corrosion control methods that will increase the life of materials that would be subject to significant corrosion.

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[1] *Romanoff, Melvin. Underground Corrosion, NBS Circular 579. Reprinted by NACE. Houston, TX, 1989, pp. 166–167.*

[2] *Romanoff, Melvin. Underground Corrosion, NBS Circular 579. Reprinted by NACE. Houston, TX, 1989, p. 8.*

The following recommendations are based on the soil conditions discussed in the Soil Corrosivity section above. Unless otherwise indicated, these recommendations apply to the entire site or alignment.

Steel Pipe

Implement *all* the following measures:

1. Underground steel pipe with rubber gasketed, mechanical, grooved end, or other nonconductive type joints should be bonded for electrical continuity. Electrical continuity is necessary for corrosion monitoring and cathodic protection.
2. Install corrosion monitoring test stations to facilitate corrosion monitoring and the application of cathodic protection:
   a. At each end of the pipeline.
   b. At each end of all casings.
   c. Other locations as necessary so the interval between test stations does not exceed 1,200 feet.
3. To prevent dissimilar metal corrosion cells and to facilitate the application of cathodic protection, electrically isolate each buried steel pipeline per NACE Standard SP0286 from:
   a. Dissimilar metals.
   b. Dissimilarly coated piping (cement-mortar vs. dielectric).
   c. Above ground steel pipe.
   d. All existing piping.
4. Choose one of the following corrosion control options:

   OPTION 1

   a. Apply a suitable dielectric coating intended for underground use such as:
      i. Polyurethane per AWWA C222 *or*
      ii. Extruded polyethylene per AWWA C215 *or*
      iii. A tape coating system per AWWA C214 *or*
      iv. Hot applied coal tar enamel per AWWA C203 *or*
      v. Fusion bonded epoxy per AWWA C213.
   b. Apply cathodic protection to steel piping as per NACE Standard SP0169.

   OPTION 2

   a. As an alternative to dielectric coating and cathodic protection, apply a ¾-inch cement mortar coating per AWWA C205 or encase in concrete 3 inches thick, using any type of cement. Joint bonds, test stations, and insulated joints are still required for these alternatives.

NOTE: Some steel piping systems, such as for oil, gas, and high-pressure piping systems, have special corrosion and cathodic protection requirements that must be evaluated for each specific application.

Iron Pipe

Implement *all* the following measures:

1. Electrically insulate underground iron pipe from dissimilar metals and from above ground iron pipe with insulating joints per NACE Standard SP0286.
2. Bond all nonconductive type joints for electrical continuity. Electrical continuity is necessary for corrosion monitoring and cathodic protection.
3. Install corrosion monitoring test stations to facilitate corrosion monitoring and the application of cathodic protection:
   a. At each end of the pipeline.
   b. At each end of any casings.
   c. Other locations as necessary so the interval between test stations does not exceed 1,200 feet.
4. Choose one of the following corrosion control options:

OPTION 1

a. Apply a suitable coating intended for underground use such as:
   i. Polyethylene encasement per AWWA C105; *or*
   ii. Epoxy coating; *or*
   iii. Polyurethane; *or*
   iv. Wax tape.

   NOTE: The thin factory-applied asphaltic coating applied to ductile iron pipe for transportation and aesthetic purposes does not constitute a corrosion control coating.

b. Apply cathodic protection to cast and ductile iron piping as per NACE Standard SP0169.

OPTION 2

a. As an alternative to dielectric coating and cathodic protection, concrete encase all buried portions of metallic piping so that there is a minimum of 3 inches of concrete cover provided over and around surfaces of pipe, fittings, and valves using any type of cement.

Copper Tubing

Implement *all* the following measures:

1. Place cold water copper tubing in an 8-mil polyethylene sleeve or encase in double 4-mil thick polyethylene sleeves and bed and backfill with clean sand at least 2 inches thick surrounding the tubing. Clean sand should have a minimum resistivity of no less than 3000 ohm-cm, and a pH of 6.0–8.0. Copper tubing for cold water can also be treated the same as for hot water.
2. Hot water tubing may be subject to a higher corrosion rate. Protect hot copper tubing by one of the following measures:
    a. Preventing soil contact. Soil contact may be prevented by placing the tubing above ground or encasing the tubing with PVC pipe with solvent-welded joints. *or*
    b. Applying cathodic protection per NACE Standard SP0169. The amount of cathodic protection current needed can be minimized by coating the tubing.

Plastic and Vitrified Clay Pipe

1. No special precautions are required for plastic and vitrified clay piping placed underground from a corrosion viewpoint.
2. Protect all metallic fittings and valves with wax tape per AWWA C217 or epoxy.

All Pipe

1. On all pipes, appurtenances, and fittings not protected by cathodic protection, coat bare metal such as valves, bolts, flange joints, joint harnesses, and flexible couplings with wax tape per AWWA C217 after assembly.
2. Where metallic pipelines penetrate concrete structures such as building floors, vault walls, and thrust blocks use plastic sleeves, rubber seals, or other dielectric material to prevent pipe contact with the concrete and reinforcing steel.

Concrete

1. From a corrosion standpoint, any type of cement may be used for concrete structures and pipe because the sulfate concentration is negligible, 0 to 0.1 percent.[3,4,5,6]
2. Standard concrete cover over reinforcing steel may be used for concrete structures and pipe in contact with these soils due to the low chloride concentration[7] found onsite.

---

[3] *1997 Uniform Building Code (UBC) Table 19-A-4*

[4] *2006 International Building Code (IBC) which refers to American Concrete Institute (ACI-318) Table 4.3.1*

[5] *2006 International Residential Code (IRC) which refers to American Concrete Institute (ACI-318) Table 4.3.1*

[6] *2007 California Building Code (CBC) which refers to American Concrete Institute (ACI-318) Table 4.3.1*

[7] *Design Manual 303: Concrete Cylinder Pipe. Ameron. p.65*

3. Due to the high ground water table encountered at this site, cyclical or continual wetting may be an issue. Any contact between concrete structures and ground water should be prevented. Contact can be prevented with an impermeable waterproofing system.

Pre-cast Concrete Piles

1. It is assumed that prestressed concrete piles will contain about 8 sacks of type V cement per cubic yard of concrete, a water/cement ratio not exceeding 0.45, and 2 inches of concrete cover. No further corrosion control measures are required for such piles.

2. If ground water is present, solid steel lifting lugs are recommended to prevent ground water from wicking into the pile interior. If wire rope lifting lugs are used, they should be carefully drilled out 1.5 inches deep and the hole filled with epoxy.

Steel Reinforced Cast in Place Concrete Piles

1. Protect steel reinforced cast-in-place and cast-in-drilled-hole concrete piles the same way as concrete structures mentioned under the concrete structures section in this report.

CLOSURE

Our services have been performed with the usual thoroughness and competence of the engineering profession. No other warranty or representation, either expressed or implied, is included or intended.

Please call if you have any questions.

Respectfully Submitted,
SCHIFF ASSOCIATES

Ronald Z. Hodgman

Ronald Z. Hodgman

Enc: Table 1

REGISTERED PROFESSIONAL ENGINEER
RONALD Z. HODGMAN
CR 1139
EXP. 3/31/09
CORROSION
STATE OF CALIFORNIA

Leobardo Solis

SCHIFF ASSOCIATES
FIFTY YEARS OF PROFESSIONALISM
www.schiffassociates.com
Consulting Corrosion Engineers – Since 1959

Table 1 - Laboratory Tests on Soil Samples

MACTEC
Long Beach Belmont Plaza
Your #4953-09-0301, SA #09-0201SCS
13-Mar-09

Sample ID			B-1 @ 15' CL	B-2 @ 10' SP
Resistivity		**Units**		
as-received		ohm-cm	640	1,880
saturated		ohm-cm	640	1,800
pH			8.3	7.9
Electrical Conductivity		mS/cm	0.21	0.15
Chemical Analyses				
Cations				
calcium	Ca^{2+}	mg/kg	27	41
magnesium	Mg^{2+}	mg/kg	14	8.3
sodium	Na^{1+}	mg/kg	250	96
potassium	K^{1+}	mg/kg	12	31
Anions				
carbonate	CO_3^{2-}	mg/kg	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	290	113
flouride	F^{1-}	mg/kg	23	1.5
chloride	Cl^{1-}	mg/kg	47	43
sulfate	SO_4^{2-}	mg/kg	65	157
phosphate	PO_4^{3-}	mg/kg	57	2.2
Other Tests				
ammonium	NH_4^{1+}	mg/kg	ND	ND
nitrate	NO_3^{1-}	mg/kg	1.1	ND
sulfide	S^{2-}	qual	na	na
Redox		mV	na	na

Electrical conductivity in millisiemens/cm and chemical analysis were made on a 1:5 soil-to-water extract.
mg/kg = milligrams per kilogram (parts per million) of dry soil.
Redox = oxidation-reduction potential in millivolts
ND = not detected
na = not analyzed

431 West Baseline Road · Claremont, CA 91711
Phone: 909.626.0967 · Fax: 909.626.3316

APPENDIX D

PRIOR REPORT OF GEOTECHNICAL INVESTIGATION

REPORT OF FOUNDATION INVESTIGATION
PROPOSED BELMONT PLAZA BEACH CENTER
ALLIN STREET AND TERMINO AVENUE
LONG BEACH, CALIFORNIA
FOR THE
CITY OF LONG BEACH
(OUR JOB NO. A-66102)

February 13, 1967

Heusel, Homolka & Associates
730 East Third Street
Long Beach, California 90812

(Our Job No. A-66102)

Gentlemen:

Support of Swimming Pool Walls and Diving Towers
Proposed Belmont Plaza Beach Center
Allin Street and Termino Avenue
Long Beach, California
for the City of Long Beach

As requested by Mr. Wendell Wilson of Bole and Wilson, we have reviewed the Natatorium Foundation and Ground Level Framing Plan, Sheet S5, dated January 23, 1967, for the subject Beach Center. We investigated the soil conditions beneath the site of the Beach Center and submitted our report of foundation investigation on August 15, 1966. We were asked to review the Foundation Plan and confirm the feasibility of supporting the pool and diving towers on spread foundations. Sheets S6 and S17, dated January 23, were also furnished us.

A review of the Foundation Plan indicates the excavation for the pool will extend as low as Elevation -3.71 at the deepest point. The foundation for the 10- and 6.5-meter diving tower will extend to Elevation -2.69, just below the bottom of the adjacent pool floor. Two adjacent 3-meter diving towers will be supported on spread foundations at Elevation +16, approximately two feet below the pool deck. Based on information furnished by Mr. Wilson and indicated on the Foundation Plan in red pencil, the footings supporting the pool walls will impose pressures up to 1,550 pounds per square foot. Pressures imposed by the spread footing supporting the 10- and 6.5-meter tower will be 1,040 pounds per square foot. The pressure imposed by the 3-meter tower footings will be 670 pounds per square foot.

Excavation for the pool at its deepest point and excavation for the 10- and 6.5-meter tower foundation will extend through the surface sand overlying the site and into silt and clay soils. The footings supporting the walls of the pool will be supported on sand and on the underlying silt and clay where the deeper excavation is required.

The soils at the planned excavated level are moderately firm and will provide adequate support for the pool and diving towers as planned. The soils at the excavated level are capable of supporting pressures up

to 2,000 pounds per square foot. We estimate that the settlement of the pool walls and the major 10- and 6.5-meter diving tower will be on the order of one-fourth inch or less. The settlement of the 3-meter towers will be on the same order of magnitude. Significant differential settlements are not anticipated. The effect of surcharge loading from the 3-meter towers should be considered in the design of the adjacent pool walls. Also, it should be noted that proper compaction of backfill will be necessary to provide adequate support for the shallow footings of the 3-meter towers.

Yours very truly,

LeROY CRANDALL AND ASSOCIATES

by LeRoy Crandall
LeRoy Crandall

JK-SC/cr
(4 copies submitted)

cc: (2) Bole and Wilson

November 9, 1966

Heusel, Homolka & Associates
730 East Third Street
Long Beach, California 90812

(Our Job No. A-66102)

Gentlemen:

Compacted Fill Material
Proposed Belmont Plaza Beach Center
Allin Street and Termino Avenue
Long Beach, California
for the City of Long Beach

This letter confirms our discussions with Mr. Wendell Wilson of Bole and Wilson, regarding the compacted fill material required at the site of the subject Belmont Plaza Beach Center. We investigated the soil conditions beneath the site of the Center and submitted our report of foundation investigation on August 15, 1966. Based on the planned floor grades and existing topography, compacted fill ranging from four to nine feet in thickness will be required within the building areas.

We understand that the soils to be excavated from the Fidelity Federal Savings & Loan Association site at Ocean Boulevard and Atlantic Avenue may become available for use at the Belmont Center site. Based on our investigation of the Fidelity Federal site (our Job No. A-66266), the soils beneath the site consist of alternating layers of silty sand, clay, silt, and sand to the planned depth of excavation.

We believe that the soils from the Fidelity Federal site will be suitable for use beneath the Belmont Center site. The clay soils would be somewhat expansive, however, and we suggest that the clay materials be used in the lower portions of the required fills to the extent possible. Although laboratory compaction tests were not performed, we estimate that the shrinkage of the excavated soils when compacted would be less than 15%. That is, it should require no more than 1.15 yards of excavated soil to make 1 yard of compacted fill.

Yours very truly,

LeROY CRANDALL AND ASSOCIATES

by LeRoy Crandall
LeRoy Crandall

LC-JK:lb
(3 copies submitted)
cc: (1) Bole and Wilson

August 15, 1966

City of Long Beach
Long Beach, California 90802

P.O. No. 4949
(Our Job No. A-66102)

Attention: Mr. Jess D. Gilkerson
City Engineer

Gentlemen:

Our "Report of Foundation Investigation, Proposed Belmont Plaza Beach Center, Allin Street and Termino Avenue, Long Beach, California, for the City of Long Beach" is herewith submitted.

The scope of the investigation was planned in collaboration with Mr. Jess D. Gilkerson, City Engineer, and with Heusel, Homolka & Associates, Architects. We were advised of the structural features of the proposed buildings by Bole and Wilson, Structural Engineers.

The natural soils beneath the site are sufficiently firm for the support of light structures, such as the maintenance building, on conventional spread footings. However, the main buildings in the Center, which will impose heavy loads, should be supported on friction piling to minimize potential settlements. The shallow water level and caving nature of the soils would prevent the use of drilled cast-in-place piling, so that driven piling will be the most feasible foundation type. Recommendations for foundation and basement wall design and for floor slab support are presented in the report.

Respectfully submitted,

LeROY CRANDALL AND ASSOCIATES

by LeRoy Crandall
LeRoy Crandall

LC-SC/cr
(4 copies submitted)

cc: (2) Heusel, Homolka & Associates
(1) Bole and Wilson

REPORT OF FOUNDATION INVESTIGATION

PROPOSED BELMONT PLAZA BEACH CENTER

ALLIN STREET AND TERMINO AVENUE

LONG BEACH, CALIFORNIA

FOR THE

CITY OF LONG BEACH

SCOPE

This report presents the results of a foundation investigation of the site of the subject development. The locations of the proposed buildings and our exploration borings are shown on Plate 1, Plot Plan.

The investigation was authorized to determine the characteristics of the soils at the site and to provide recommendations for foundation design. The results of the field explorations and laboratory tests, which form the basis of our recommendations, are presented in the attached Appendix.

STRUCTURAL CONSIDERATIONS

The proposed buildings comprising the Center are shown in plan on Plate 1. The pool building will have 50-foot-high walls; there will be a one-story locker building to the east of the pool building, and a two-story community building with a partial basement to the west. The buildings will be of reinforced concrete construction. Column loads will be approximately 200 kips in the pool building, 100 kips in the community building, and 70 kips in the locker building. Wall loads will be on the order of 3,000 pounds per lineal foot. The maintenance building will be one story high and also of reinforced concrete construction. Foundation loads in this building will be light.

The planned floor elevations are shown on Plate 1. Based on the planned floor grades and existing topography, compacted fill some four to nine feet in thickness will be required within the building areas. Excavation approximately three feet deep will be required for the partial basement, with somewhat deeper excavation required for the proposed pool.

SITE CONDITIONS

The site is located on the beach, and is clear of existing structures. Elevations of the existing grade at selected locations are shown on Plate 1.

As disclosed by the exploration borings, the natural soils beneath the site consist of sand to a depth of approximately ten feet. Below this upper sand, the soils consist of silt, silty sand, and clay to depths of 19 to 27 feet, below which the soils consist primarily of sand. The sand deposits are generally moderately firm to firm; the cohesive soils are moderately firm.

Based on measurements made in the borings, the water level is at approximately Elevation +4. Some variations in the water level may occur with the normal fluctuation of the tide.

RECOMMENDATIONS

FOUNDATIONS

General

The upper soils are only moderately firm and are not adequate for support of the heavy buildings on spread footings. To assure adequate support of the proposed buildings, we recommend the use of driven friction piling. The installation of drilled cast-in-place piling is not feasible due to the shallow water level and caving nature of the soils. The proposed maintenance building, which will impose lighter loads, may be supported on conventional spread

footings. If the upper natural soils are reworked and the planned fill is properly compacted, such footings may be established in either the undisturbed natural soils or properly compacted fill.

Driven Piling

The downward and upward capacities of several types of driven piles are presented on Plate 2, Driven Pile Capacities. Dead plus live load capacities are shown; a one-third increase may be used when considering wind or seismic loads. If required fill is compacted as recommended, the fill may be assumed to offer support to the piles equal to that of the natural soils. Where piles are located immediately adjacent to the partial basement or swimming pool, the supporting capacity should be neglected for that portion of the piles located above a plane drawn upward through the bottom edge of the basement or pool at an angle of 45 degrees with the horizontal.

Piles in groups should be spaced at least 2½ diameters on centers, but in no event less than three feet on centers. If the piles are so spaced, there will be no reduction in the downward capacity of the piles due to group action. The maximum ultimate settlement of the proposed buildings, supported on driven piling as recommended, will be less than one-fourth inch.

Because of the dense nature of the sand, firm driving should be anticipated. Pile lengths which would be indicated by use of a dynamic formula should not be substituted for the predetermined values shown on Plate 2. Pile driving should commence near an exploration boring so that driving criteria can be established. If jetting is required, only pilot jetting should be permitted; side jetting could seriously affect the lateral capacities of the piles and should not be used. We recommend that the pile driving and any required jetting be done under the supervision of personnel of our firm, to assure that the desired capacities are developed.

The driven piles may be used to resist lateral loads. Compacted fill soils or natural soils adjacent to a concrete pile having a butt diameter of 16 inches can safely resist horizontal thrusts up to 14,000 pounds. The lateral value of a wood pile with a 12 inch butt diameter would be 7,000 pounds. The lateral resistance of other sizes of piles would be proportional to the diameter. The maximum bending moment in a pile due to a horizontal load imposed at the top of the pile may be computed by multiplying the load by an assumed moment arm of five feet. For design, it may be assumed that the maximum bending moment occurs at or near the top of the pile and that the bending moment decreases to zero at a depth of ten feet below the pile cap. The lateral capacity and reduction in the bending moment are based on the assumption that necessary backfill adjacent to pile caps and grade beams will be properly compacted.

Lateral loads may also be resisted by friction between the floor slab and the subgrade and by the passive resistance of the soils. A coefficient of friction of 0.5 may be used between the floor slabs and the supporting soils. The passive resistance of the natural soils or properly compacted backfill against pile caps and grade beams may be assumed to be equal to the pressure developed by a fluid with a density of 300 pounds per cubic foot. If the imposed lateral loads can be resisted by the piles, by friction, and/or by the passive resistance of the soils, tie-beams will not be necessary between pile foundations for seismic stability.

Spread Footings

The proposed maintenance building may be supported on conventional spread footings established in the natural soils. If the upper natural soils are reworked and required fill is compacted as recommended in a following

section on grading, footings may be established in either the compacted fill or the natural soils. If footings are to be established in the fill, the reworking of the upper natural soils and the compaction of all required fill should be controlled and certified by a competent soils engineer.

Conventional spread footings carried into undisturbed natural soils or established on properly compacted fill may be designed to impose a dead plus live load pressure of 2,000 pounds per square foot at a depth of two feet below the adjacent finished grade or floor level, whichever is lower. A one-third increase in the bearing value may be used when considering wind or seismic loads. The bearing value of the fill will depend on the materials used; however, if acceptable soils are used and are compacted as recommended, the quoted bearing value will be obtained. The maximum ultimate settlement of the proposed maintenance building, supported on spread footings in the manner recommended, will be on the order of one-fourth inch.

The soils are relatively cohesionless, and it will be necessary to form footings extending into sand. The supporting sand should be wet down prior to placing concrete. However, footing backfill and utility trench backfill should be mechanically compacted to assure adequate floor slab support.

For resisting lateral loads, a coefficient of friction of 0.5 may be used between footings or the floor slabs and the supporting soils. The passive resistance of the natural soils or properly compacted fill may be assumed to be equal to the pressure developed by a fluid with a density of 300 pounds per cubic foot.

WALLS BELOW GRADE

No difficulties are anticipated in excavating for the partial basement as planned; however, the soils will not stand vertically without shoring.

We recommend that the walls of an unshored excavation be sloped back to at least 1:1.

Building walls below grade should be designed to resist the lateral pressure developed by a fluid with a density of 30 pounds per cubic foot. Required backfill against the walls should be mechanically compacted to at least 90% of the maximum density obtainable by the ASTM Designation D1557-64T method of compaction modified to use three layers. Flooding of the backfill should not be permitted. If the backfill is compacted as recommended, hydrostatic pressures are not expected to develop against basement walls, and waterproofing will not be required. Damp-proofing of the basement walls should be satisfactory.

GRADING

To provide improved support for floor slabs and adequate support for the maintenance building footings established in fill, the upper natural soils should be reworked and all required fill must be properly compacted. If footings are to be established in the fill, the reworking of the natural soils and the compaction of all fill must be controlled and certified by a competent soils engineer. Any imported fill materials should be approved by the soils engineer.

No significant difficulties due to soil conditions are anticipated in grading the site as planned. However, the sand is relatively cohesionless, and it may be difficult to operate construction vehicles on the sand. Also, the compacted fill and the natural soils can be readily eroded, so that surface drainage should be carefully controlled.

Prior to placing any fill, the exposed natural soils should be scarified to a depth of six inches, brought to optimum moisture content, and rolled

with heavy compaction equipment. At least six passes of a heavy pneumatic-tired roller or vibratory roller are recommended to compact the upper soils. The upper six inches of natural soils should be compacted to at least 90% of the maximum density obtainable by the ASTM Designation D1557-64T method of compaction modified to use three layers. Required fill should be placed in loose lifts not more than eight inches in thickness, brought to optimum moisture content, and compacted to at least 90%.

The on-site soils, if available, may be used in required fills. Since the soils consist primarily of clean sand, it will be difficult to operate construction equipment and to maintain a uniform subgrade on the sand. Keeping the clean sands moist to wet will minimize the difficulties. All required imported material should consist of non-expansive and predominantly granular soils such as a silty sand.

FLOOR SLAB SUPPORT

If the subgrade is prepared as recommended, adequate support should be provided for the building floor slabs. If a capillary break is desired beneath the floor slabs, the slabs may be supported on a four-inch-thick layer of gravel or an impermeable membrane. A suggested gradation for this gravel layer would be as follows:

Sieve Size	Percent Passing
3/4"	90 - 100
No. 4	0 - 10
No. 100	0 - 3

If a membrane is used instead of the gravel, it should be covered with a thin layer of sand to allow curing of the concrete, or a low-slump concrete should be used to minimize possible curling of the slabs.

SWIMMING POOL

The groundwater level was measured at depths of 5 to 7 feet below the existing grade. Accordingly, depending on the pool depth, water may be encountered and dewatering may be necessary during construction of the pool. Relief valves may be required in the pool bottom to prevent uplift pressures when the pool is drained. When empty, it should be assumed that the soils above the water level will exert a pressure on the pool walls equal to that developed by a fluid with a density of 30 pounds per cubic foot; below the water level a fluid pressure of 80 pounds per cubic foot should be used.

-oOo-

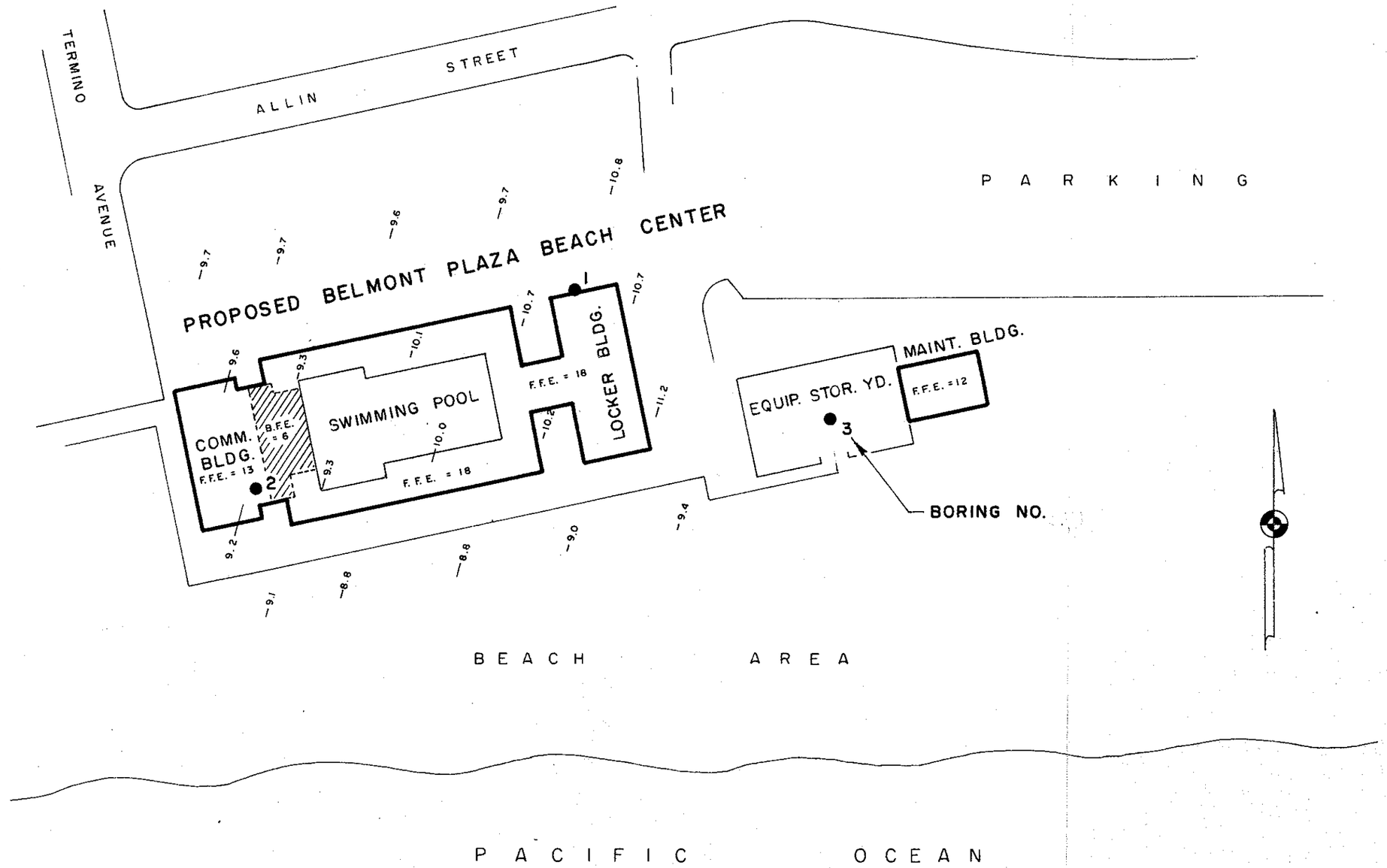
The following Plates and Appendix are attached and complete this report:

Plate 1 . . . . . Plot Plan

Plate 2 . . . . . Driven Pile Capacities

Appendix . . . . Explorations and Laboratory Tests

JOB 466102 DATE 4-27-66 DR. M.C. O.E. CHKD. RCL



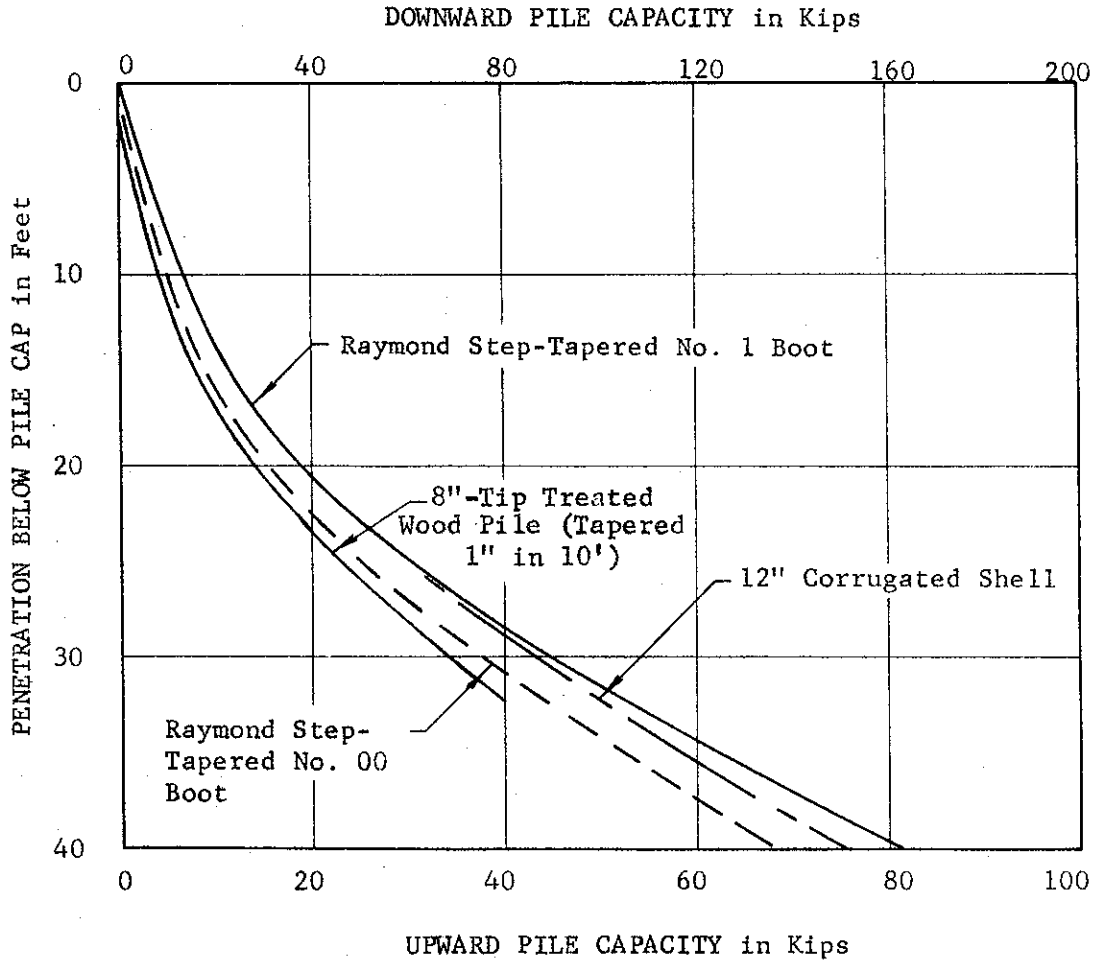
REFERENCE :
SURVEY AND PLANS PROVIDED BY
HEUSEL, HOMOLKA & ASSOCIATES, A.I.A. ARCHITECTS.

PLOT PLAN

SCALE 1" = 100'

LEROY CRANDALL & ASSOCIATES

PLATE 1



NOTES:

(1) The indicated values refer to the total of dead plus live loads; a one-third increase may be used when considering wind or seismic loads.

(2) Piles in groups should be spaced a minimum of 2½ diameters, but not less than 3 feet, on centers.

DRIVEN PILE CAPACITIES

JOB A66102 DATE 8-11-66 DR. JM O.E. SR CHKD. RL SR

APPENDIX

EXPLORATIONS

The site of the proposed development was explored by drilling three borings to depths ranging from 41 to 50 feet below the existing ground surface. The borings were drilled deeper than initially planned to provide sufficient data for design of pile foundations. The borings were drilled using 6-inch-diameter rotary wash-type drilling equipment under the supervision of our field engineer.

The soils encountered were logged by our field engineer, and undisturbed samples were obtained for laboratory inspection and testing. The boring logs are presented on Plates A-1 through A-3; the depths at which undisturbed samples were obtained are indicated to the left of the boring logs. The soils are classified in accordance with the Unified Soil Classification System shown on Plate B.

LABORATORY TESTS

The field moisture content and dry density of the soils were determined by performing tests on the undisturbed samples. The test results are shown to the left of the boring logs.

Direct shear tests were performed on selected undisturbed samples to determine the strength of the soils. The tests were performed at field and increased moisture contents and at various surcharge pressures. Selected samples were tested at two different surcharge pressures to provide more complete data. The results of the tests are presented on Plate C, Direct Shear Test Data.

Confined consolidation tests were performed on four undisturbed samples to determine the compressibility of the soils. Water was added to two of

the samples during the tests to illustrate the effect of moisture on the compressibility. The consolidation test results are presented on Plates D-1 and D-2, Consolidation Test Data.

The optimum moisture content and maximum dry density of the soils were determined by performing a compaction test on a sample from Boring 2. The test was performed in accordance with the ASTM Designation D1557-64T method of compaction modified to use three layers instead of five. The results of the test are presented on Plate E, Compaction Test Data.

-oOo-

BORING I

DATE DRILLED: April 13, 1966
EQUIPMENT USED: 6"-Diameter Rotary Wash

ELEVATION 11

ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	SAMPLE		
10		4.3	107	■	SP	SAND - fine, light brown
		14.8	102	■		Layer of SANDY SILT, greyish-brown
		8.0	99	■		Few sea shells
		17.3	114	■		▼ Water level (4/14/66)
0	10	22.3	103	■		
		20.1	111	■	CL	SILTY CLAY - brown
-10	20	24.7	103	■	SM	SILTY SAND - fine, brown
		27.6	101	■		
-20	30	17.0	109	■	SP	SAND - fine, grey
		9.4	125	■	SW	SAND - well graded, few gravel, grey
-30	40	15.8	113	■		Pockets of clay
		22.0	104	■	SP	SAND - fine, greyish-brown
						Layer of well graded SAND with gravel
-40	50	16.3	112	■		Pockets of clay and silt
	60					

NOTE: Drilling mud used in drilling process; mud removed after drilling completed. Water level measured at a depth of 7'.

*Elevations refer to datum of reference drawing; see Plate 1.

Soils classified in accordance with the Unified Soil Classification System.

LOG OF BORING

LEROY CRANDALL & ASSOCIATES

PLATE A-1

JOB A66102 DATE 4-27-66 DR. GMC O.E. CR CHKD. AC

JOB A66102 DATE 4-27-66 DR. GMC O.E. SA CHKD. PPL

BORING 2

DATE DRILLED: April 14, 1966

EQUIPMENT USED: 6"-Diameter Rotary Wash

ELEVATION 9

ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	SAMPLE		
		4.2	105	■	SP	SAND - fine, few sea shells, light brown
		14.0	103	■		
		22.4	105	■		▼ Grey Water Level (4/14/66)
		29.4	96	■		Streaks of coarse sand
0	10	28.5	98	■		
					ML	CLAYEY SILT - greyish-brown
		23.1	103	■		
					ML	SANDY SILT - some clay, bluish-grey
-10	20	23.1	101	■	SP	SAND - fine, dark grey
		17.6	107	■		
-20	30	16.4	112	■	SW	SAND - well graded, brownish-grey
		18.0	108	■		
					SP	SAND - fine, grey
-30	40	21.5	104	■		
-40	50					

NOTE: Drilling mud used in drilling process; mud removed after drilling completed. Water level measured at a depth of 5'.

LOG OF BORING

LEROY CRANDALL & ASSOCIATES

JOB A6602 DATE 4-27-66 DR. Gmc O.E. SL CHKD. RC

BORING 3

DATE DRILLED: April 14, 1966
EQUIPMENT USED: 6"-Diameter Rotary Wash

ELEVATION 10

ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	SAMPLE		
		4.6	102	■	SP	SAND - fine, few sea shells, light brown
		9.0	102	■		Slightly silty
		20.0	109	■		▼ Water Level (4/14/66)
		22.1	105	■		More sea shells, grey
0	10	23.5	101	■		
		30.0	96	■	ML	CLAYEY SILT - greyish-brown
-10	20	27.3	96	■	SP	SAND - fine, brown
		23.6	104	■		Pockets of coarse sand
-20	30	18.1	108	■		Pockets of well graded sand
		12.8	116	■	SW	SAND - well graded, greyish-brown
-30	40	25.3	100	■	CL	SILTY CLAY - some sand, light brown
-40	50					

NOTE: Drilling mud used in drilling process; mud removed after drilling completed. Water level measured at a depth of 6'.

LOG OF BORING

LEROY CRANDALL & ASSOCIATES

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOILS (More than 50% of material is LARGER than No. 200 sieve size)	GRAVELS (More than 50% of coarse fraction is LARGER than the No. 4 sieve size)	CLEAN GRAVELS (Little or no fines)	GW	Well graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly graded gravels or gravel-sand mixtures, little or no fines.
		GRAVELS WITH FINES (Appreciable amt. of fines)	GM	Silty gravels, gravel-sand-silt mixtures.
			GC	Clayey gravels, gravel-sand-clay mixtures.
	SANDS (More than 50% of coarse fraction is SMALLER than the No. 4 sieve size)	CLEAN SANDS (Little or no fines)	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		SANDS WITH FINES (Appreciable amt. of fines)	SM	Silty sands, sand-silt mixtures.
			SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS (More than 50% of material is SMALLER than No. 200 sieve size)	SILTS AND CLAYS (Liquid limit LESS than 50)		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS (Liquid limit GREATER than 50)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils.

BOUNDARY CLASSIFICATIONS: Soils possessing characteristics of two groups are designated by combinations of group symbols.

PARTICLE SIZE LIMITS

SILT OR CLAY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

NO. 200 NO. 40 NO. 10 NO. 4 3/4 in. 3 in. (12 in)

U. S. STANDARD SIEVE SIZE

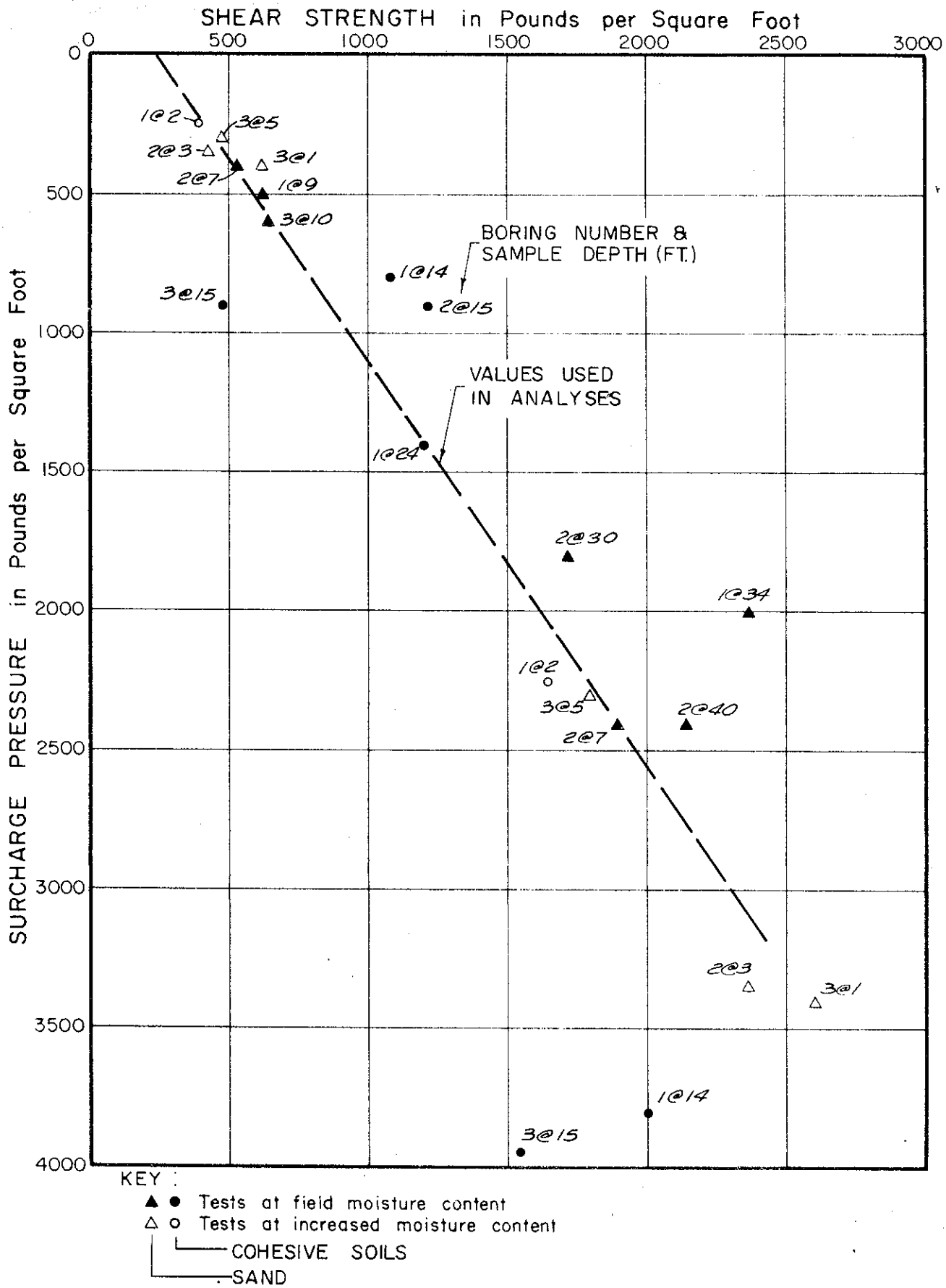
UNIFIED SOIL CLASSIFICATION SYSTEM

Reference :
The Unified Soil Classification System, Corps of Engineers, U.S. Army Technical Memorandum No. 3-357, Vol. I, March, 1953. (Revised April, 1960)

LEROY CRANDALL AND ASSOCIATES

PLATE B

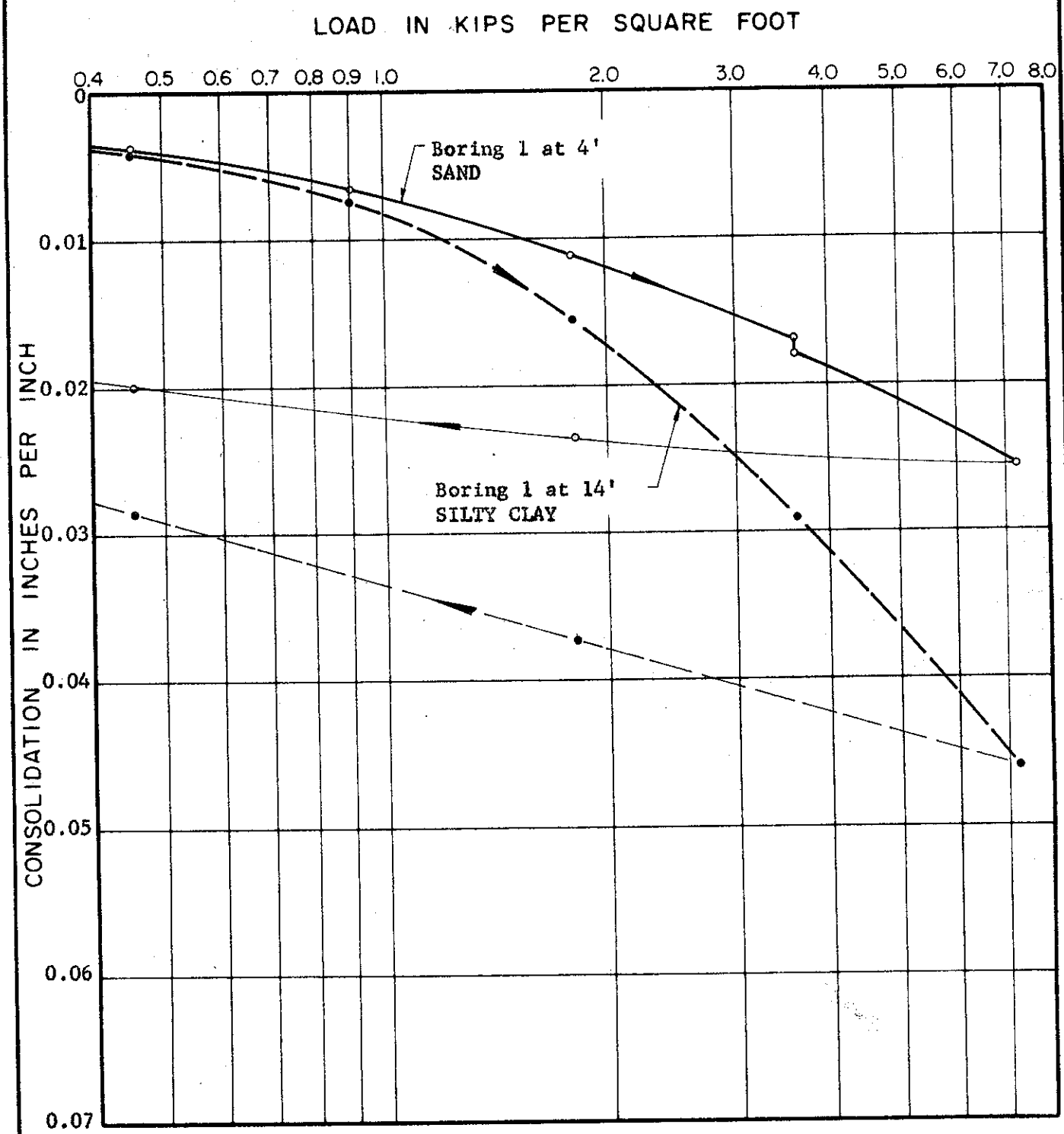
JOB A-66102 DATE 5-5-66 DR. VJM O.E. GC CHKD. TK



DIRECT SHEAR TEST DATA

LEROY CRANDALL & ASSOCIATES

PLATE C



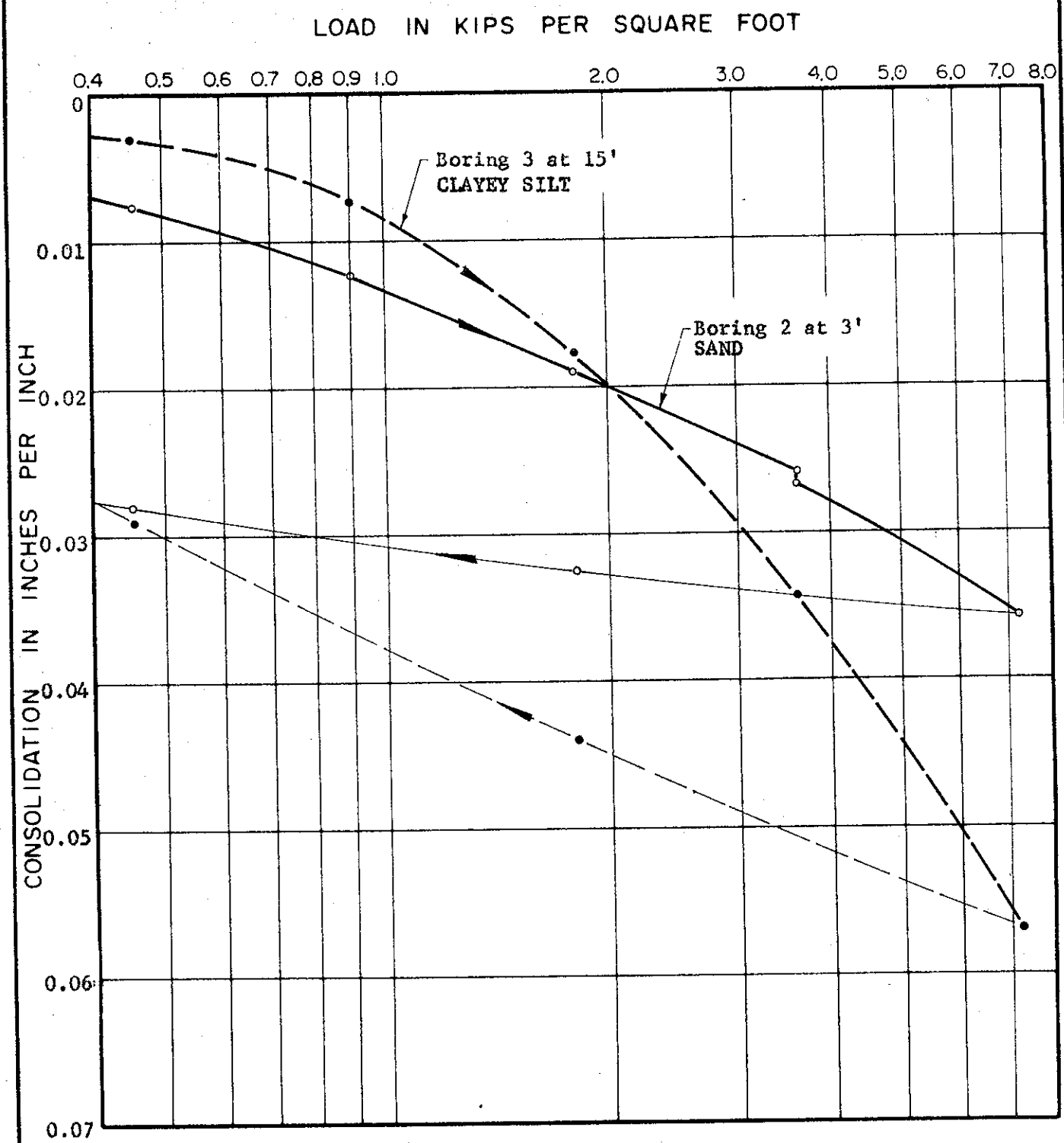
NOTE: Water added to sample from 4' after consolidation under a load of 3.6 kips per square foot. The other sample tested at field moisture content.

CONSOLIDATION TEST DATA

LEROY CRANDALL & ASSOCIATES

PLATE D-1

JOB A-66102 DATE 4-27-66 DR. U.M. O.E. Gr CHKD. RL



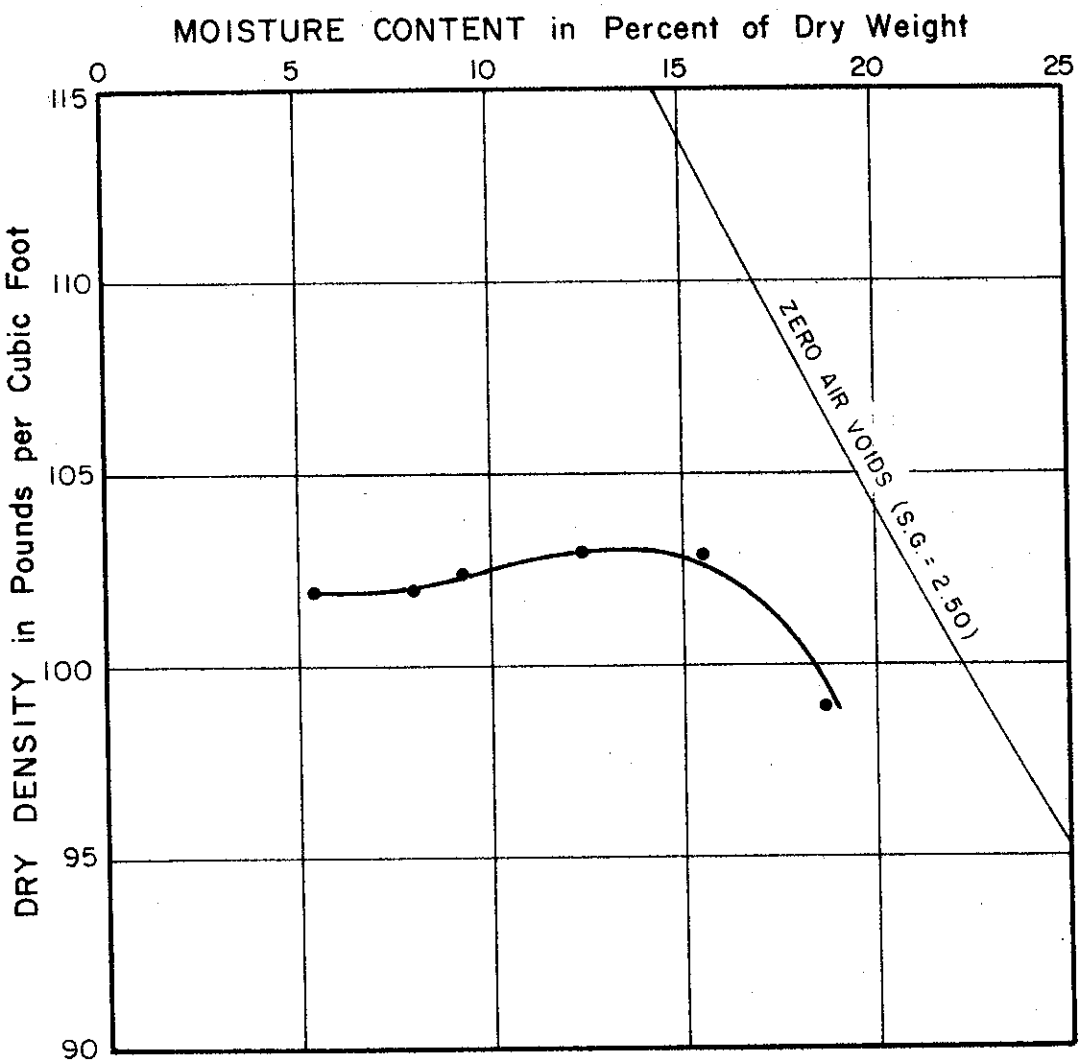
NOTE: Water added to sample from 3' after consolidation under a load of 3.6 kips per square foot. The other sample tested at field moisture content.

CONSOLIDATION TEST DATA

LEROY CRANDALL & ASSOCIATES

PLATE D-2

JOB A-66102 DATE 4-27-66 DR. VM O.E. CR CHKD. AL



SOURCE: BORING 2 AT 0'-2'
SOIL TYPE: SAND
MAXIMUM DRY DENSITY: 103 lbs. / cu. ft.
OPTIMUM MOISTURE CONTENT: 14 % of dry wt.
TEST METHOD: Modified ASTM Designation D1557-64T.
This method utilizes a 1/30-cubic-foot mold, in which each of three layers of soil is compacted by 25 blows of a 10-pound hammer falling 18 inches.

COMPACTION TEST DATA

LEROY CRANDALL & ASSOCIATES

PLATE E

JOB AG6102 DATE 4-27-66 DR. GMC O.E. CHKD.

GEOTECHNICAL INVESTIGATION FOR THE TEMPORARY MYRTHA POOL AND ASSOCIATED IMPROVEMENTS

INTRODUCTION

PURPOSE

This report presents the results of our geotechnical investigation for the temporary Myrtha Pool and associated improvements proposed in the existing public parking lot located to the east of the Belmont Plaza Olympic Pool Complex at 4000 Olympic Plaza within the City of Long Beach, California (see Plate 1 – Location Map). The purpose of this report is to provide a summary of our geotechnical investigation, data, and conclusions, and then provide geotechnical recommendations pertaining to site remedial grading and for the design and construction of the proposed temporary pool and associated site improvements.

SCOPE

1. Reviewed background information pertaining to the site, including published regional geologic maps and literature and a previous geotechnical report by Mactec for the adjacent Belmont Plaza Olympic Pool Complex.

2. Performed an initial site reconnaissance to assess current surface conditions and mark the site for Underground Service Alert.

3. Conducted a subsurface exploration program that consisted of the advancement of four CPT soundings each to a depth of 50 feet and the drilling of one hand-augered boring to a depth of 5 feet in order to physically observe the subsurface soils and to obtain samples for laboratory testing. The boring was logged by our senior engineer and samples were collected for laboratory testing.

4. Performed laboratory testing on a bulk sample that was collected during our subsurface exploration.

5. Interpreted and evaluated field conditions and laboratory data.

6. Performed geotechnical engineering analyses using the field and laboratory data in conjunction with the conceptual site plan. The analysis addressed site seismicity, anticipated settlement, groundwater, liquefaction, and concrete flatwork and pool design.

7. Prepared this report which summarizes the results of our research, subsurface exploration, laboratory and field testing, analyses, conclusions, and recommendations relative to the proposed improvements at the subject site.

SITE LOCATION AND DESCRIPTION

The temporary pool and associated improvements are proposed within a public parking lot located to the east of the Belmont Plaza Olympic Pool Complex which is located at 4000 Olympic Plaza within the City of Long Beach, California. This public parking lot is bordered on the west by Bennett Avenue, on the north by a landscaped easement and then East Ocean Boulevard, and on the south by the beach (Pacific Ocean). The parking lot is also bordered on the south by an existing pool and by a City of Long Beach maintenance building and yard that have been constructed on the beach. To the west, the parking lot continues beyond the limits of planned improvements. The general location of the site with respect to nearby roadways is shown on Plate 1.

The existing parking lot is paved with asphalt, has light bollards and parking meters between the rows of parking stalls, and is surrounded by concrete curbs and gutters. At the end of the parking stalls are planters with groundcover and palm trees.

The parking lot appears to drain by sheet flow towards the north to northwest towards the intersection of East Ocean Boulevard and Bennett Avenue. The pavement exhibits various levels of distress ranging from occasional cracks to extensive alligator cracking with local depressions. The distress is more extensive along the north side of the parking lot.

SITE RESEARCH AND PREVIOUS GEOTECHNICAL REPORTS

Reviewed materials for the site included geology maps and previously published geologic reports in order to identify site history and geologic conditions. These included:

- State of California Seismic Hazard Zones Map; Long Beach Quadrangle, base map prepared by U.S. Geologic Survey and dated 1964 (Photo revised 1981), Official Map Released March 3, 1999, Scale: 1 inch = 2000 feet

- Seismic Hazard Zone Report for the Long Beach 7.5-Minute Quadrangle, Los Angeles County, California, Seismic Hazard Zone Report 028 (California Division of Mines and Geology, 1998).

MACTEC previously performed a subsurface investigation for the adjacent Belmont Plaza Pool Complex (reference (1)). This investigation included the drilling of two exploratory borings each to a depth of 76.5 feet using a hollow-stem auger drill rig and the advancement of five Cone Penetration Test (CPT) soundings each to a depth of 50 feet. Samples of the onsite soils were obtained by MACTEC for laboratory testing. Laboratory testing associated with this

previous investigation included in-place moisture content/dry density, particle size analysis, Atterberg limits, consolidation and shear strength characteristics, and soil corrosivity.

This report recommended that the complex building either be underpinned with new pile foundations and the exterior improvements be protected from lateral spreading by ground improvement, or that the entire structure be demolished, the entire building site improved by ground improvement, and then a new building constructed.

PLANNED IMPROVEMENTS

Based on our conversations with representatives of RJM Design Group, it is our understanding that a portion of the existing parking lot to the east of the Belmont Plaza Olympic Pool Plaza will be removed and replaced with a large concrete slab that will support a temporary above-ground Myrtha pool. Two- thirds of the pool will be constructed on-grade and will be supported on a 12-inch-thick concrete slab. The remaining one-third of the pool will be constructed on 3 feet of new fill and the 12-inch-thick concrete slab. The pool walls will be constructed as braced stainless steel walls. The concrete slab will also support the braced walls while isolated concrete footings will support the raised decking and bleachers that will surround the pool.

Other improvements include temporary restroom/shower and office trailers, temporary asphalt walkways and curbs, planter areas, fencing, and 70- to 80-foot-high light poles. Some of the existing asphalt paving will also be covered with slurry and restriped.

Based on the current plans, the majority of the site will remain at existing grades; therefore, only minor cuts and fills will be required within most areas. However, the deep portion of the temporary pool will require cuts of up to 12 inches while the shallow portion of the pool will require fills of up to 3 feet to reach proposed bottom of slab grades.

SUBSURFACE EXPLORATION

Our subsurface investigation consisted of the advancement of four CPT soundings (CPT-1 through CPT-4) each to a depth of 50 feet to obtain continuous geotechnical information of the subsurface soils. In addition, a single hand-augered drill hole was advanced within a planter area to a depth of 5.5 feet to physically observe the subsurface soils and to obtain a bulk sample for geotechnical testing. The drill hole was logged by our senior engineer. The locations of the CPT soundings and drill hole are shown on Plate 2 – Geotechnical Map, and the log of the drill hole and the results of the CPT soundings are included in Appendix A.

MONITORING WELLS

In order to determine depths to groundwater, two monitoring wells were installed within the site. These monitoring wells were both 20 feet deep and were comprised of 2-inch diameter slotted pipes installed within 8-inch diameter drilled holes. The space around the slotted pipes was backfilled with clean sand to within 3 feet of existing grade and then capped with 2 feet of bentonite to seal the wells from surface water.

LABORATORY TESTING

Laboratory testing for the subject investigation was performed to determine the expansion potential and corrosion characteristics of the onsite soils. Corrosion testing included the determination of soluble sulfate and chloride concentrations, and soil pH and electrical resistivity. Laboratory procedures and test results are presented in Appendix B – Geotechnical Laboratory Procedures and Test Results. Pertinent laboratory test data is also shown on our recent drill hole log.

Laboratory test results on samples collected at the site indicate that onsite soils are:

- Non-expansive
- Moderately corrosive to concrete
- Corrosive to ferrous metals

GEOLOGIC FINDINGS

LOCAL GEOLOGY AND SUBSURFACE SOIL CONDITIONS

The site is located within an area that has been significantly altered by the construction of manmade islands and landforms and is underlain by undifferentiated older and younger artificial fill that has been placed over native young alluvium and estuarine deposits. Within our CPT soundings, it was not possible to differentiate between the fill and the native soils.

Our recent drill hole and CPT soundings indicate that the site is underlain by approximately 8 to 13 feet of poorly graded sand and silty sand, a 4- to 15-foot-thick layer of intermixed clay and silty clay, and then poorly graded sand and silty sand to the maximum depth explored (50 feet). Within the southern portion of the site, the clay layer is located approximately 13 feet below the ground surface and is 4 to 6.5 feet thick, while in the northern portion of the site, the clay layer is

located approximately 8 to 9 feet below the ground surface and is approximately 9 to 15 feet thick. The poorly graded sands and silty sands are loose to medium dense with rootlets in the upper 12 to 18 inches, becoming medium dense to dense below while the underlying clays and silty clays are firm.

At the locations of the CPT soundings, the existing asphalt was observed to range from approximately 2.5 to 3 inches thick. The underlying base is intermixed with varying amounts of sand and ranges from approximately 6 to 7 inches thick.

GROUNDWATER

Groundwater was originally measured within our drill hole and CPT soundings at the time of our subsurface exploration. Groundwater was measured three more times within our monitoring wells. The groundwater depths ranged from 6 to 6.8 feet below existing grades. These depths to groundwater are in general agreement with the depths of historically high groundwater provided in the reference (2) Seismic Hazard Zone Map for the Long Beach Quadrangle. Since the project does not include any subsurface structures, groundwater is not expected to adversely impact the proposed grading or construction.

FAULTING AND SEISMICITY

The site is not located within an Official Alquist-Priolo Earthquake Fault Zone and no known active faults are shown on current geologic maps for the site. The nearest known active fault is the Newport-Inglewood Fault, which is located approximately 2.3 kilometers from the site and is capable of generating a maximum earthquake magnitude (Mw) of 7.1. The site is also located within 10.1 kilometers of the Palos Verdes fault, which is capable of generating a maximum earthquake magnitude (Mw) of 7.3. Given the proximity of the site to these and numerous other active and potentially active faults, the site will likely be subject to earthquake ground motions in the future.

In order to evaluate the likelihood of future earthquake ground motions occurring at the site, a probabilistic seismic hazard analysis (PSHA) of horizontal ground shaking was performed using the commercial computer program EZ-FRISK ver. 7.43. The PSHA utilized seismic sources and attenuation equations consistent with the 2008 USGS National Seismic Hazard Mapping Project. Assuming a conservative risk level of 10 percent probability of exceedance in 50 years (i.e., ~475 year ARP), the PHGA is 0.38g.

It should be noted that this peak ground acceleration has a 10 percent probability of being exceeded in 50 years (which is roughly equivalent to the design life of an average long-term development).

For the subject project, the temporary Myrtha pool will only be used for approximately 2 years or less and the probability of a significant earthquake occurring during this time span is so low that the corresponding PGA would be essentially zero. Therefore, the PGA we used can be considered to be very conservative for this temporary structure.

SEISMIC HAZARD ZONES

The subject property is not located within an area mapped as having the potential for seismic-induced landsliding; however, it is located within an area mapped as having the potential for seismic-induced liquefaction as shown on the reference (2) Seismic Hazard Zone Map for the Long Beach Quadrangle.

GEOTECHNICAL ENGINEERING FINDINGS

LIQUEFACTION, SEISMIC SETTLEMENT, AND LATERAL SPREADING

Liquefaction Investigation

Since the site is located within a zone mapped as having the potential for earthquake induced liquefaction, liquefaction and related hazards were quantitatively evaluated utilizing the CPT soundings to a maximum depth of 50 feet.

Design Earthquake and Mode Magnitude

Based on our site specific PSHA with deaggregation, a very conservative PGA of 0.38g, Modal Magnitude 7.2, and modal distance of 11.2 km were calculated for this study.

Design Groundwater Level

Actual groundwater levels encountered during our recent exploration indicate a groundwater level of approximately 6 to 6.5 feet below existing site grades, which are in agreement with the depths of historically high groundwater provided in the reference (2) Seismic Hazard Zone Map for the Long Beach Quadrangle. Therefore our analysis was performed using the worst case condition (5 feet b.g.s.).

Liquefaction Analyses

GMU utilized CLiq to evaluate CPT data for liquefaction. CLiq is a commercial computer software program that applies the latest NCEER methods for liquefaction analysis including post-earthquake settlement and lateral displacement.

Liquefaction, Seismic Settlement, and Lateral Spreading Potential

Our analysis indicates that discrete zones within the underlying soils below the groundwater level may be subject to liquefaction during a design seismic event. Based on our analysis, the site has a slight to moderate potential for any adverse effects of liquefaction due to the depth and discrete nature of the liquefiable zones. Liquefaction seismic settlement calculations indicate approximately 0.3 to 1.75 inches of settlement could occur during a design earthquake. The results of our analyses are presented in Appendix C.

The site also has a moderate potential for adverse effects due to seismic-induced lateral spreading. Our calculations indicate that lateral displacements at the points of exploration could range from approximately 9 to 80 inches. The results of our analyses are presented in Appendix C.

Based on the thickness and depth of liquefiable layers shown in our liquefaction analysis (Appendix C), the guidelines provided by Southern California Earthquake Center (1999), and the design curves proposed by Ishihara (1985) which provide criteria for identifying conditions causing or not causing damage to foundations, the site is not anticipated to be subject to liquefaction-induced foundation bearing failure.

Although there is the potential for liquefaction-induced settlement and lateral spreading using the very conservative design PGA, the actual probability of a seismic event actually occurring during the short time of service for the proposed temporary pool is essentially zero. However, due to this very small potential, it is recommended that the pool be underlain by a reinforced concrete slab described in a subsequent section of this report. This slab will help mitigate the potential for liquefaction to adversely affect the proposed temporary pool.

STATIC SETTLEMENT/COMPRESSIBILITY

The on-site granular soils were found to be medium dense to dense while the fine-grained soils were found to be firm and are not susceptible to significant consolidation. Total static settlements can be expected to range from approximately ½ to ¾-inch with a differential settlement of ½-inch over a span of 40 feet.

SOIL EXPANSION

The on-site granular soils to depths of at least 8 feet are non-expansive while the underlying clay can be classified as having a moderate expansion potential based on our assessment of the soil classifications provided in the CPT logs in Appendix A and the results of expansion index testing contained in Appendix B. A non-expansive potential should therefore be assumed for planning purposes of the structures proposed on-grade.

SOIL CORROSION

To evaluate the corrosion potential of the on-site soils to both ferrous metals and concrete, representative samples were tested for pH, minimum resistivity, soluble chlorides, and soluble sulfates. The results of chemical testing contained in Appendix B indicate that the soil sample tested contains a negligible concentration of sulfates and severe concentrations of chlorides. In addition, due to the proximity of the site to the nearby ocean, there is a high potential for onsite structures to come in contact with seawater. Thus, the onsite soils should be considered moderately corrosive to concrete and severely corrosive to ferrous metals.

EXCAVATION CHARACTERISTICS

Rippability

The soil materials underlying the site can be easily excavated with conventional grading equipment such as loaders, excavators, and backhoes.

Trenching

We expect that excavation of new utility trenches can be accomplished utilizing conventional trenching machines and backhoes. Trench support requirements will be limited to those required by safety laws or other locations where trench slopes will need to be flattened or supported by shoring designed to suit the specific conditions exposed.

Volume Change

For the rough determination of earthwork quantities, we estimate that the change in volume of on-site disturbed surficial fills that are excavated and placed as new compacted fill at an average relative compaction of 92% will result in an average of about 5% loss in volume. It should be noted that the aforementioned value is approximate and is for rough planning purposes only.

CONCLUSIONS AND RECOMMENDATIONS

DEVELOPMENT FEASIBILITY

Based on the geologic and geotechnical findings, it is our opinion that proposed grading and construction is feasible and practical from a geotechnical standpoint if accomplished in accordance with the City of Long Beach grading and building requirements and the recommendations presented herein. It is also the opinion of GMU Geotechnical that proposed grading and construction will not adversely affect the geologic stability of adjoining properties provided grading and construction are performed in accordance with the recommendations provided in this report.

A summary of conclusions is as follows:

1. The site should be considered developable and not expected to adversely impact adjacent properties from a geotechnical perspective utilizing standard grading techniques.

2. Site soils are artificial fill materials overlying native alluvial and estuarine deposits. The upper 12 to 18 inches of the fill materials are medium dense and will require re-processing.

3. Groundwater was measured at 6 to 6.8 feet below existing grade, which agrees with the historic high groundwater level.

4. There are no known active faults within the subject site. The site seismicity is typical for the Long Beach area. Structure design should be in accordance with the current CBC.

5. Based on visual observations and laboratory testing of the on-site materials, corrective grading at the site will be limited to the overexcavation and recompaction of the surficial engineer fill materials that are expected to be disturbed during demolition operations within the site. Additional removals may be necessary depending on the materials encountered during grading.

6. Some of the existing asphalt pavement sections will need to be demolished. Due to the limited amount of grading and fill placement that will occur, the old asphalt and base materials generated from the removal of the existing pavement sections should be collected and hauled off-site.

7. Existing subsurface utility lines, depending on their depths and locations in relation to the proposed development, may need to be excavated and removed, or abandoned in-place.

8. The potential for liquefaction and lateral spreading is conservatively estimated to be low to

moderate. Due to the short service life of the temporary pool, the probability is essentially zero. Estimated vertical seismic settlements range from 0.3 to 1.75 inches with a differential seismic settlement of less than ½-inch over a span of 40 feet. Lateral displacements at the points of exploration could range from approximately 9 to 80 inches. Due to the very remote possibility of seismic-induced settlement and lateral spreading as a result of liquefaction, the temporary pool should be constructed on a reinforced concrete slab.

9. Site soils to a depth of at least 8 feet are non-expansive. Future site improvements will not require any special design for expansive soil conditions.

10. Corrosion testing indicates that the on-site soils are severely corrosive to ferrous metals. Consequently, any metal exposed to the soil will need protection and all reinforcing steel will need to be properly covered by concrete.

11. The on-site testing indicates negligible amounts of sulfate. However, due to the proximity of the site to the nearby ocean, there is a high potential for onsite structures to come in contact with seawater. Therefore, it is recommended that a moderate level of sulfate exposure (i.e., Type II/V cement with a water/cement ratio of 0.50) be assumed for proposed concrete slabs.

SITE PREPARATION AND GRADING

General

The subject site should be precise graded in accordance with the City of Long Beach grading code requirements (and all other applicable codes and ordinances) and the recommendations as outlined in the following sections of this report. The geotechnical aspects of future grading plans and improvement plans should be reviewed by GMU Geotechnical prior to grading and construction. Particular care should be taken to confirm that all project plans conform to the recommendations provided in this report. All planned and corrective grading should also be monitored by GMU Geotechnical to verify general compliance with the recommendations outlined in this report.

Demolition and Clearing

Prior to the start of the planned improvements, some of the existing asphalt pavement will need to be demolished. The old asphalt and base materials generated from the removal of the existing pavement sections may be collected and used as compacted fill provided that it is thoroughly crushed and broken down with no fragments greater than 3 inches in maximum diameter.

The on-site soils are suitable for use as compacted fill from a geotechnical perspective if care is taken to remove all significant organic and other decomposable debris.

Cavities and excavations created upon removal of subsurface obstructions, such as existing buried utilities, should be cleared of loose soil, shaped to provide access for backfilling and compaction equipment, and then backfilled with properly compacted fill.

The project geotechnical consultant should provide periodic observation and testing services during demolition operations to document compliance with the above recommendations. In addition, should unusual or adverse soil conditions or buried structures be encountered during grading that are not described herein, these conditions should be brought to the immediate attention of the project geotechnical consultant for corrective recommendations.

Corrective Grading – Existing Grades

Existing soils that comprise the upper 12 to 18 inches feet of the site are damp to moist and medium dense. In addition, it is expected that the surficial soils will be disturbed during the demolition of the existing asphalt pavement sections. Therefore, to provide adequate support of proposed improvements, the subgrade soils exposed after demolition should be overexcavated to a depth of at least 18 inches, moisture conditioned (as necessary) to at least 2% above the optimum moisture content, and then replaced as properly compacted fill at a minimum relative compaction of 90%.

FILL MATERIAL AND PLACEMENT

Suitability

All on-site soils are considered suitable for use as compacted fill from a geotechnical perspective if care is taken to remove all significant organic and other decomposable debris, and separate and stockpile rock materials larger than 6 inches in maximum diameter.

Compaction Standard and Methodology

All soil material used as compacted fill, or material processed in-place or used to backfill trenches, should be moistened, dried, or blended as necessary and densified to at least 90% relative compaction as determined by ASTM Test Method D 1557. It is recommended that fills be placed a minimum of 2% above optimum moisture content.

Material Blending

The existing surficial engineered fill materials are expected to be generally slightly below optimum moisture content but may have variable moisture content depending on the season in which work is performed. The majority of the materials to be handled during grading will require some blending

and addition of water to meet acceptable moisture ranges for sufficient compaction (i.e., minimum 2% above optimum moisture content).

Use of Rock or Broken Asphalt

As described previously, the existing asphalt and base materials may be used as new fill provided that it is thoroughly crushed and broken down with no fragments greater than 3 inches in maximum diameter. In addition, these materials should also be blended with the onsite soils prior to being placed as compacted fill.

TEMPORARY EXCAVATION STABILITY

Trench excavations will also be required for new utility lines, if any. The sidewalls of these temporary excavations are expected to expose non-cohesive granular silty sands and sands.

Based on the anticipated engineering characteristics of these materials, temporary excavations for any new utility trench walls to a depth of 3 feet may be made vertically without shoring subject to verification of safety by the contractor. Deeper excavations should be braced, shored, or sloped back no steeper than 1:1 (horizontal to vertical). In addition, no surcharge loads should be allowed within 5 feet of the trench walls.

We anticipate the trench walls to be temporarily stable to a height of 3 feet provided the above recommendations are followed. However, deeper excavations will encounter saturated conditions or groundwater which will adversely affect the stability of the trench bottoms and sidewalls. Modifications to our recommendations may be required based on our observations of the actual conditions exposed in the field.

Our temporary excavation recommendations are provided only as general guidelines and all work associated with temporary excavations should meet the minimal requirements as set forth by CAL-OSHA. Temporary slope and trench excavation construction, maintenance, and safety are the responsibility of the contractor.

POST-GRADING CONSIDERATIONS

Utility Trench Backfill Considerations

Backfill compaction of utility trenches should be such that no significant settlement will occur. Backfill for all of these trenches should be compacted to at least 90% relative compaction subject to sufficient observation and testing. In the event that granular material having a sand equivalent of 30

or greater is used for backfill and this material is thoroughly flooded into place, extensive testing is not required. If native material with a sand equivalent less than 30 is used for backfill, it should be placed at near-optimum moisture content and mechanically compacted.

Jetting or flooding will not densify native soil materials with a sand equivalent less than 30 due to its silty to clayey nature. Also, jetting or flooding of granular material should not be used to consolidate backfill in trenches adjacent to any foundation elements.

Where trenches closely parallel a footing (i.e., for retaining walls) and the trench bottom is located within a 1 horizontal to 1 vertical plane projected downward and outward from any structure footing, concrete slurry backfill should be utilized to backfill the portion of the trench below this plane. The use of concrete slurry is not required for backfill where a narrow trench crosses a footing at about right angles.

We suggest that these recommendations be included as a specification in all subcontracts for underground improvements. In addition, the design of all underground conduits, pipelines, or utilities should also consider the potentially corrosive nature of the on-site soils to metals, as previously described in this report.

Surface Drainage

Surface drainage should be carefully controlled to prevent runoff over graded slope surfaces and ponding of water on flat pad areas. Positive drainage away from graded slopes is essential to reduce the potential for erosion or saturation of slope surfaces. All drainage at the site should be in minimum conformance with the applicable City of Long Beach codes and standards.

PRELIMINARY FOUNDATION DESIGN RECOMMENDATIONS

Structure Seismic Design

No active or potentially active faults are known to cross the site, therefore, the potential for primary ground rupture due to faulting on-site is very low to negligible. However, the site will likely be subject to seismic shaking at some time in the future. For design of future buildings, retaining walls, or other structural improvements, site-specific seismic design parameters were determined using the USGS computer program titled "Seismic Hazard Curves and Uniform Hazard Response Spectra, Version 5.0.8." The site coordinates used in the analysis were 33.7579° North Latitude and 118.1441° West Longitude and the site class designation "D" was determined by the shear wave testing performed within the CPT-1 sounding. On-site structures should be designed in accordance with the following 2010 CBC criteria:

Parameter	Factor	Value
0.2s Period Spectral Response	S_s	1.742g
1.0s Period Spectral Response	S_1	0.665g
Soil Profile Type	Site Class	D
Site Coefficient	F_a	1.0
Site Coefficient	F_v	1.5
Adjusted Spectral Response	SM_s	1.742g
	SM_1	0.998g
Adjusted Spectral Response	SD_s	1.161g
	SD_1	0.665g

It should be recognized that much of southern California is subject to some level of damaging ground shaking as a result of movement along the major active (and potentially active) fault zones that characterize this region. Design utilizing the 2010 CBC is not meant to completely protect against damage or loss of function. Therefore, the preceding parameters should be considered as minimum design criteria.

CONCRETE SLAB DESIGN

Modulus of Subgrade Reaction

For the design of the concrete slab to support the temporary pool, a modulus of subgrade reaction of 150 pci may be used.

Thickness and Reinforcement

The temporary pool and surrounding pool decking and bleachers will be supported on a new concrete slab. Since this new slab may be exposed to future movements (including liquefaction-induced settlement and lateral spreading), it should have a minimum thickness of 12 inches and be minimally reinforced with No. 4 bars at 18 inches on center.

Final determination of slab thickness and reinforcement should be determined by the structural engineer based on actual loading conditions.

Subgrade Soil Moisture Content

The foundation subgrade should be moisture conditioned/pre-saturated as necessary to at least 2% over the optimum moisture content to a minimum depth of 18 inches. The moisture content of the subgrade soils should be verified by GMU prior to initiating foundation construction.

CONCRETE

It is anticipated that the onsite soils will have a moderate sulfate exposure per Section 1904.3 of the 2010 CBC. Therefore, Type II/V cement along with a maximum water/cement ratio of 0.50 should be used for all concrete in contact with the onsite soils. This recommendation will serve to minimize the potential of water and/or vapor transmission through the concrete and minimize the potential for physical attack to concrete from non-sulfate based salts. In addition, wet curing of the concrete as described in ACI Publication 308 should be considered.

The aforementioned recommendations in regards to concrete are made from a soils perspective only. Final concrete mix design as well as any concrete testing is outside our purview. All applicable codes, ordinances, regulations, and guidelines should be followed in regard to designing a durable concrete with respect to the potential for detrimental exposure from the on-site soils and/or changes in the environment.

CORROSION PROTECTION OF METAL STRUCTURES

The results of the laboratory chemical tests performed on soil samples collected within and adjacent to the subject area indicate that the on-site soils are severely corrosive to ferrous metals. Consequently, metal structures which will be in direct contact with the soil (i.e., underground metal conduits, pipelines, metal sign posts, metal door frames, etc.) and/or in close proximity to the soil (wrought iron fencing, etc.) may be subject to corrosion. The use of special coatings or cathodic protection around buried metal structures has been shown to be beneficial in reducing corrosion potential. The potential for corrosion of ferrous metal reinforcing elements embedded in structural concrete will be reduced by the use of the recommended maximum water/cement ratio for concrete.

The laboratory testing program performed for this project does not address the potential for corrosion to copper piping. In this regard, a corrosion engineer should be consulted to perform more detailed testing and develop appropriate mitigation measures (if necessary). Otherwise, the on-site soils should be considered corrosive to copper.

The above discussion is provided for general guidance in regards to the corrosiveness of the on-site soils to typical metal structures used for construction. Detailed corrosion testing and recommendations for protecting buried ferrous metal and/or copper elements is beyond our purview.

FOUNDATIONS FOR RAISED DECKS, BLEACHERS AND FENCING

The raised decks and bleachers and the fencing around the pool will be supported on individual footings. Recommendations for these footings are provided in the following section.

Foundation Design Parameters

• Minimum Foundation Width	18 inches
• Minimum Depth	18 inches below lowest outside adjacent grade
• Bearing Materials	Engineered fill
• Minimum Footing Reinforcement	Four #4 bars; two at top and two at bottom of footing
• Allowable Bearing Capacity	2,000 psf with minimum embedment of 18 inches (may be increased 20% for each additional foot of width or embedment to a maximum of 3,000 psf).
• Coefficient of Friction	0.35
• Unit Weight of Backfill	125 pcf
• Passive Earth Pressure	250 pcf on flat ground (disregard upper 6 inches and reduce passive by one-third when combining friction and passive pressure).
• Concrete	0.50 w/c ratio; Type II/V cement (geotechnical perspective only).

POLE FOUNDATIONS

Pole foundations will be required for new light bollards within the subject site. As a minimum, the pole foundations should be at least 18 inches in diameter and at least 3 feet deep; however, the actual dimensions should be determined by the project structural engineer based on anticipated lateral loads and on the following design parameters.

Bearing Materials. The pole foundations may bear into competent native soils approved by a representative from GMU.

Bearing Values. End-bearing capacity and skin friction may be combined to determine the allowable bearing capacities of the pole foundations. An allowable bearing pressure of 2000 pounds per square foot (psf) may be used for pole foundations at least 18 inches in diameter and embedded a minimum of 3 feet below the lowest adjacent grade. A value of

250 pounds per square foot may be used to determine the skin friction between the concrete and surrounding soil.

Lateral Load Design. Lateral loads may be resisted by friction at the base of the foundations and by passive resistance within the adjacent earth materials. A coefficient of friction of 0.35 may be used between the foundations and the recommended bearing material. For passive resistance, an allowable passive earth pressure of 200 pounds per foot of pile diameter per foot of depth into competent bearing material may be used; however, passive resistance should be disregarded within the upper 2 feet due to possible disturbance during drilling. The passive resistance may be assumed to be acting over an area equivalent to two pile diameters.

Construction Considerations. It should be noted that the site is underlain by shallow groundwater. Groundwater can be expected to be encountered at a depth of approximately 6 feet below the existing ground surface. As a result of capillary action, the subsurface soils can be expected to be saturated at a depth of 5 feet below the existing ground surface. Based on these conditions, severe caving can be expected to occur within the pole foundation excavations below a depth of 5 feet. Therefore, temporary casing will be required to advance the pole foundation excavations to their required depths.

NEW ASPHALT PAVEMENT AND SLURRY

It is proposed to construct new asphalt walkways on top of the existing asphalt pavement to support pedestrian traffic. New asphalt will also be placed on top of the existing asphalt pavement to support the new restroom/shower trailers and office trailer.

It is recommended that this new asphalt be at least 3 inches thick and that the existing asphalt be cleaned and tack coated prior to the placement of the new asphalt.

It is also proposed to slurry a portion of the existing parking lot that will serve as the main access to the existing parking lot. Since this area of proposed slurry will serve as the main access, it is expected that it will experience a significant amount of traffic. In addition, during our site reconnaissance end subsurface exploration, it was noted that the existing asphalt is in very poor condition with extensive cracking, depressions, and areas where the asphalt has completely fragmented and broken apart. Therefore, in lieu of conventional slurry, it is recommended that the existing pavement be covered with a higher strength "chip" seal or "cape" seal. Specific recommendations for these products can be provided, if requested.

FUTURE PLAN REVIEW

GMU should review future project plans to check for conformance to the recommendations provided herein, and to provide additional recommendations as needed. Specifically, GMU should review the final grading plans and landscape plans.

GEOTECHNICAL OBSERVATION AND TESTING

It is recommended that geotechnical observation and testing be performed by this firm during the following stages of grading and construction:

- During site clearing and grubbing.
- During all phases of grading including corrective grading, scarification, ground preparation, moisture conditioning, and placement and compaction of all fill materials.
- During placement and compaction of aggregate base.
- During excavation of foundations for new walls and similar structures.
- When any unusual conditions are encountered.

LIMITATIONS

All parties reviewing or utilizing this report should recognize that the findings, conclusions, and recommendations presented represent the results of our professional geological and geotechnical engineering efforts and judgments. Due to the inexact nature of the state of the art of these professions and the possible occurrence of undetected variables in subsurface conditions, we cannot guarantee that the conditions actually encountered during grading and site construction will be identical to those observed, sampled, and interpreted during our study, or that there are no unknown subsurface conditions which could have an adverse effect on the use of the property.

We have exercised a degree of care comparable to the standard of practice presently maintained by other professionals in the fields of geotechnical engineering and engineering geology, and believe that our findings present a reasonably representative description of geotechnical conditions and their probable influence on the grading and use of the property.

Because our conclusions and recommendations are based on a limited amount of current and previous geotechnical exploration and analysis, all parties should recognize the need for possible revisions to our conclusions and recommendations during grading of the project.

Additionally, our conclusions and recommendations are based on the assumption that our firm will act as the geotechnical engineer of record during construction and grading of the project to observe the actual conditions exposed, to verify our design concepts and the grading contractor's general compliance with the project geotechnical specifications, and to provide our revised conclusions and recommendations should subsurface conditions differ significantly from those used as the basis for our conclusions and recommendations presented in this report.

It should be further noted that the recommendations presented herein are intended solely to minimize the effects of post-construction soil movements. Consequently, minor cracking and/or distortion of all on-site improvements should be anticipated.

SUPPORTING DATA

The following Plates and Appendices which complete this report are listed in the Table of Contents.

Respectfully submitted,

GMU GEOTECHNICAL, INC.

David Hansen, M.Sc., RCE 56591
Senior Geotechnical Engineer

Gregory Silver, M.Sc., PE, GE 2336
Principal Geotechnical Engineer

dwh/12-137-00R (4-3-13)

REFERENCES

SITE-SPECIFIC REFERENCE

(1) "Report of Preliminary Geotechnical Investigation, Proposed Belmont Plaza Olympic Pool Revitalization Project, 4000 East Olympic Plaza, Long Beach, California," report by MACTEC, dated April 14, 2009.

TECHNICAL REFERENCES

Boore, D.M. and Atkinson, G.M., 2008, Ground-Motion Prediction Equations for the Average Horizontal Component of PGA, PGV, and 5%-Damped PSA at Spectral Periods between 0.01 s and 10 s: Earthquake Spectra, Vol. 24, No. 1, p. 99-138.

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Chiou, B.S., and Youngs, R.R., 2008, An NGA Model for the Average Horizontal Component of Peak Ground Motion and Response Spectra: Earthquake Spectra, Vol. 24, No. 1, p. 173-215.

"Seismic Hazard Zone Report for the Long Beach 7.5-Minute Quadrangle, Los Angeles County, California," prepared by the Department of Conservation, Division of Mines and Geology, Seismic Hazard Zone Report 028, dated 1998.

"State of California Seismic Hazard Zones Map; Long Beach Quadrangle," base map prepared by U.S. Geologic Survey and dated 1964 (Photo revised 1981), Official Map Released March 3, 1999, scale: 1 inch = 2000 feet.

PRELIMINARY GEOTECHNICAL REPORT

AESCO

Orange County
17782 Georgetown Lane
Huntington Beach, California 92647
Tele: (714) 375-3830
Fax: (714) 375-3831

San Bernardino County
14163 Arrow Boulevard
Fontana, California 92335
Tele: (909) 284-9200
Fax: (909) 284-9201

PRELIMINARY GEOTECHNICAL REPORT
BELMONT PLAZA POOL REBUILD-REVITALIZATION
4000 EAST OLYMPIC PLAZA
LONG BEACH,, CALIFORNIA
AESCO PROJECT NO. 20140185-C8050

Prepared for:

Harley Ellis Devereaux
601 South Figueroa Street, Suite 500
Los Angeles, CA 90017

Attention: Mr. Diego Matzkin

Prepared By:

AESCO
17782 Georgetown Lane
Huntington Beach, California 92647
Adam Chamaa, MSCE, P.E., Manager

April 24, 2014

AESCO

Orange County
17782 Georgetown Lane
Huntington Beach, California 92647
Tele: (714) 375-3830
Fax: (714) 375-3831

San Bernardino County
14163 Arrow Boulevard
Fontana, California 92335
Tele: (909) 284-9200
Fax: (909) 284-9201

April 24, 2014

Mr. Diego Matzkin
Harley Ellis Devereaux
601 South Figueroa Street, Suite 500
Los Angeles, CA 90017

Subject: **Preliminary Geotechnical Report**
Belmont Plaza pool Rebuild-Revitalization
4000 East Olympic Plaza
Long Beach, California
AESCO Project No. 20140185-C8050

Dear Mr. Matzkin:

AESCO is pleased to provide you with the preliminary geotechnical report for the above-referenced pool facility to be constructed at the subject site. This initial report is to assist in the preliminary design evaluation and cannot be used for the final design. The project generally consists of constructing an indoor pool which is approximately 85 feet by 190 feet in plan dimension housed inside an approximately 60, 000 square foot structure. The structure will include a second-story banquet facility, locker rooms, restrooms, pool storage, offices and a basement level mechanical room. An outdoor pool with a plan dimension of approximately 85 feet by 185 feet will be located just east of the structure. There will also be a restaurant near the southwest side of the structure. Two small pools will also be constructed; a teaching pool just south of the indoor pool and an outdoor recreation pool just south of the outdoor pool.

Please do not hesitate to contact us if you have any questions or if we may be of any additional assistance. We look forward to assisting you during the construction of the proposed facility.

Sincerely,

AESCO, Inc.

Debra L. Perez
Project Manager

Russell J. Scharlin, MSCE, P.E., G.E.
Senior Project Engineer

Adam Chamaa, MSCE, P.E., G.E.
Engineering Manager

REGISTERED PROFESSIONAL ENGINEER
ADAM CHAMAA
GE 2784
Exp. 12/31/2015
GEOTECHNICAL
STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER
RUSSELL J. SCHARLIN
GE 751
Exp. 12/31/2015
GEOTECHNICAL
STATE OF CALIFORNIA

Preliminary Geotechnical Report
Belmont Plaza Pool Rebuild-Revitalization
4000 East Olympic Plaza
Long Beach, California
AESCO Project No. 20140185-C8050

This report (authorized by Harley Ellis Devereaux), presents the preliminary results of a geotechnical investigation performed by AESCO for the Belmont Plaza Pool Rebuild-Revitalization at 4000 East Olympic Plaza, Long Beach, California. The layout of the proposed facility is shown on the Site Plan, Figure 1.

We understand that the project generally consists of constructing an indoor pool which is approximately 85 feet by 190 feet in plan dimension housed inside an approximately 60,000 square foot structure. The structure will include a second-story banquet facility, locker rooms, restrooms, pool storage, offices and a basement level mechanical room. An outdoor pool with a plan dimension of approximately 85 feet by 185 feet will be located just east of the structure. There will also be a restaurant near the southwest side of the structure. Two small pools will also be constructed; a teaching pool just south of the indoor pool and an outdoor recreation pool just south of the outdoor pool.

The purpose of this study was to provide preliminary geotechnical input for design of the new pool facility. The scope of our services included the following:

- Coordinating site access for the field investigation;
- Obtaining utility clearances for the field investigation;
- Performing geotechnical drilling and sampling at the site;
- Performing monitoring during drilling with a PID monitor (photoionization detector);
- Obtaining samples for environmental testing and analysis;
- Performing laboratory testing of representative samples;
- Conducting a seismic hazards screening;
- Engineering analyses; and
- Preparing this preliminary report.

This preliminary report summarizes our findings and presents geotechnical recommendations for the design of this facility. Additional borings will be drilled once the existing building is demolished and/or the building location is finalized and a final geotechnical report will be prepared.

Preliminary Field Investigation

A preliminary field investigation was conducted at the site between April 3 and April 9, 2014 to obtain information on the subsurface conditions. The borings (B-1 through B-10) were placed as close to the footprint of the new facility as possible and were drilled with a hollow stem auger and rotary drilling methods. Drilling mud was introduced into the boreholes to minimize disturbance of granular soils and to keep the borings open during drilling operations. The borings ranged in depth from 35 feet to 80 feet below the existing ground surface. The boring location is shown on the Site Plan, Figure 1. The site plan is based on a proposed site layout drawing by Hastings and Chivetta, dated April 21, 2014. AESCO's field engineer logged the borings and visually classified and collected samples of the subsurface materials encountered in the borings. The borings were backfilled with cement grout and cuttings were transported from the site. The Logs of Borings B-1 through B-10 are presented in the attached Appendix. Additional borings are planned for the final geotechnical investigation. All samples were monitored with a photoionization detector (PID) and results are recorded on the boring logs.

All equipment that came into contact with potentially impacted soil or water was decontaminated before sampling and between borings to prevent cross-contamination of samples. Disposable equipment intended for single-use was packaged for appropriate disposal. Decontamination of equipment occurred prior to, and after, each use. Decontamination was performed using a non-phosphate detergent and water wash and a deionized/distilled water rinse. All soil samples collected for laboratory analysis were placed in appropriate sample containers, properly labeled and preserved, immediately placed in an iced cooler and submitted to the Associated testing laboratories along with the chain-of-custody.

Drive samples were taken in the borings using either a Standard Penetration Test (SPT) sampler or a Modified California (MC) sampler. The sampler was driven 18 inches into the bottom of the borehole using a 140-pound hammer falling a distance of 30 inches. The MC sampler barrel was lined with stainless steel liners to collect relatively undisturbed soil samples. All of the samples were sealed and packaged to help preserve the natural moisture content and to protect them from further disturbance.

Laboratory Testing

Geotechnical Testing

All testing was performed in accordance with ASTM Standards and California Test Methods. Laboratory testing performed in our Huntington Beach, California geotechnical laboratory consisted of water content (ASTM D4959), dry density (ASTM D2937), direct shear (ASTM

D3080), Atterberg Limits (ASTM D4318), and washed sieve analysis (ASTM D1140). Results of the laboratory tests are summarized on the Boring Log and are included in the attached Appendix. Chemical analyses, including pH (ASTM D1293), soluble sulfates (CT417) and soluble chlorides (CT422), and minimum resistivity (CT 301) were also performed. The results of the chemical testing will be presented in the final report.

Environmental Testing

Selected soils samples and groundwater grab samples were preserved and sent to Associated Laboratories for hydrocarbon and heavy metals testing. The test results and the Chain-of-Custody forms are included in the Appendix.

Grab Groundwater Sampling

On April 9, 2014, AESCO collected a grab sample of the water in borings B-3, B-5, B-6, and B-9. The water samples were analyzed for:

- TPHg by EPA Method 8015m,
- TPHd by EPA Method 8015m,
- TPH motor oil by EPA Method 8015m.

Grab Soil Sampling

On April 9, 2014, grab soil samples were collected from borings B-3, B-5, B-6, and B-9. A total of 10 soil samples were collected (two from B-3, B-6, and B-9 and four from B-5) and analyzed for:

- pH by EPA Method 9045,
- CAM 17 Metals by EPA Method 6010B and 7471A,
- VOC by EPA Method 8260B,
- TPH by EPA Method 418.1,
- TPH by EPA Method 8015B, and
- BTEX by EPA Method, 8021B.

Regional Geologic Setting

The project site is located in Long Beach, California within the coastal margin of the Los Angeles basin. The site is located at the transition between the northern portion of the Peninsular Ranges Physiographic Province and the southern portion of the Transverse Ranges Physiographic Province. The project area is considered to be within the Transverse Ranges

physiographic province by Norris and Webb (1990) and within the Peninsular Ranges physiographic province by Yerkes, et al. (1965). These two physiographic provinces have contrasting tectonic characteristics that overlap within the Los Angeles basin resulting in a complex tectonic environment marked by active faulting and historic seismicity.

Local Geologic Setting

The Belmont Pool site is located adjacent to the beach and is underlain by 3 feet of fill material generally comprised of silty sand. Alluvial sediments beneath the fill are generally composed of sands, silty sands, sandy silt, and sandy clays. The AMSL is approximately 5 feet or less. The site slopes very gently in a southerly direction.

Site and Subsurface Conditions

The proposed improvements are located just south of the beach and will encompass the entire area of the existing Belmont Plaza. The site is currently covered with the existing structure on the south side and a park which is located on the north side of the site. The existing structure will be demolished prior to construction. The site is relatively flat. Existing underground utilities are present within the site boundary.

Based on our preliminary findings, the material underlying the borings (B-1 through B-10) generally consist of 3 feet of silty sand fill material which is underlain by medium dense to very dense sand, very soft to very stiff sandy silt, very soft to very stiff sandy clay and silty clay, medium dense to very dense sand/silty sand, and medium dense to dense silty sand.

Groundwater was encountered within the borings at depths ranging from 6 feet to 9 feet below the existing ground surface. Based on regional data, historical high groundwater is anticipated to occur at a depth of less than 10 feet (CGS 1998). The depth to groundwater may fluctuate and perched groundwater may occur, depending on rainfall and possible groundwater recharge or pumping activity in the site vicinity.

Seismic Design

A seismic hazards screening was performed for this site to evaluate potential seismic hazards. The seismic hazards screening consisted of reviewing available data published by the California Geological Survey (CGS), the 2013 California Building Code (CBC), and the 2012 International Building Code (IBC). The site is located in the United States Geological Survey Long Beach Quadrangle.

Site Class	D
Spectral Response 'Ss'	1.561g
Spectral Response 'SMs'	1.561g
Spectral Response 'S1'	0.582g
Spectral Response 'SM1'	0.873g

The computer program (EQFAULT, Version 3.00b) and data published by the CGS "The Revised 2002 California Probabilistic Seismic Hazard Maps," June 2003, were reviewed. Results of the fault search are presented in the Appendix. The search indicates that the Newport-Inglewood (L.A. Basin) fault is approximately 3.8 kilometers from the site. The CGS (CDMG, 2000-003) does not delineate this site as being within an Alquist-Priolo Earthquake Fault Zone. However, with the active faults in the region, the site could be subjected to future strong ground shaking that may result from earthquakes on local to distant sources.

Liquefaction Potential

Liquefaction is a mode of ground failure that results from the generation of high pore water pressures during earthquake ground shaking, causing loss of shear strength. Liquefaction is typically a hazard where loose sandy soils exist below groundwater. The CGS has designated certain areas within southern California as potential liquefaction hazard zones. These are areas considered at a risk of liquefaction-related ground failure during a seismic event, based upon mapped surficial deposits and the presence of a relatively shallow water table. The project site is located within a potential liquefaction hazard zone as designated by the CGS (1999). Materials encountered at the project site generally consist of medium dense to very dense granular material and very soft to very stiff cohesive soil. Groundwater was encountered within the borings at depths ranging between 6 feet and 9 feet below the existing ground surface. Historical high groundwater in the project vicinity is less than 10 feet below the ground surface (CGS, 1998). Based on this, we conclude that the potential for liquefaction at the site is high. Other geologic hazards related to liquefaction, such as lateral spreading, are therefore also high. A more in-depth liquefaction analysis will be provided within the final geotechnical report.

Environmental Test Results, Evaluations and Recommendations

Grab Groundwater Test Results

The test results indicated that TPHg, TPHd, and TPH motor oil was not detected in the grab groundwater samples collect from borings B-3, B-6, and B-9.

TPH was detected in B-5 at the following concentrations:

- TPHg at 0.60 mg/l,
- TPHd at 0.94 mg/l, and
- TPHmotor oil at 0.37 mg/l.

Grab Soil Sampling Test Results

Arsenic was detected above California Human Health Screening Levels (CHHSL) and Regional Screening Levels (RSL) in all samples collected. Lead was detected at or slightly above the RSL in all samples collected. TPHg was not detected in any of the samples. TPHd was detected in boring B-5 at 13 feet to 15 feet below the existing ground surface, in boring B-5 at 5 feet to 7 feet, and in boring B-6 at 5 feet to 7 feet. TPH motor oil was detected in boring B-5 at 13 feet to 15 feet and in boring B-6 at 5 feet to 7 feet below the existing ground surface.

Conclusions

Although arsenic was detected above the RSL and CHHSL it remains below the RSL determined by the Department of Toxic Substance Control (DTSC) in their document *Determination of a Southern California Regional Background Arsenic Concentration in Soil* and is not considered a constituent of concern based on the future use of the site. A review of the Draft Phase I report prepared by Ninyo and Moore, dated June 7, 2013, indicated two open-case Leaking Underground Storage Tank (LUST) sites up-gradient of the site, one closed-case site up-gradient of the site, and one closed-case site cross-gradient of the site. The two open-case LUST sites up-gradient of the site were determined to be potential environmental concerns.

Therefore, additional sampling in the vicinity of borings B-5 and B-6 is warranted. At a minimum, two borings should be placed up-gradient of borings B-5 and B-6 to further evaluate the potential environmental concern of on-site migration of TPH from potential up-gradient sources and two borings cross- and down-gradient of the borings to further define the extent of the current on-site conditions.

Structural Loads

The structural loads were not available at this time but are anticipated to be relatively high.

Foundation Recommendations

Based on the anticipated column loads for the structure, prestressed concrete piles or drilled cast-in-place concrete piers should be used to support the structure. Steel pipe piles were not considered due to the high corrosion potential of the salt water environment.

Prestressed Concrete Piles

The concrete piles derive their axial load resistance by skin friction between the piles and the soil. The design axial loads for various sizes of the piles based on a safety factor of 2.0 are presented in the Appendix. These loads are based on anticipated settlements of less than ½ inch total and ¼ inch differential. Lateral loads rely on passive resistance of the soils against the face of the pile. As a minimum, the upper three feet below the existing surface should be over-excavated and re-compacted to at least 95 percent relative compaction as determined by ASTM D1557.

For preliminary evaluation of lateral loads an equivalent fluid with a density of 150 pounds per cubic foot may be assumed for determining the lateral resistance of the soils against the projected width of the pier to a depth of 10 feet. The maximum lateral resistance should be limited to 1500 pounds per square foot at depths greater than 10 feet below the ground surface. The contribution of lateral resistance to a depth equal to four feet should be neglected. Once actual loading conditions are identified a lateral load analysis can be performed using p-y analysis. If lateral loads are high, batter piles can be considered for lateral resistance.

The hammer shall be either a single or double acting air, steam, or diesel hammer having a minimum energy rating of 15,000 foot pounds, however, prior to casting of productive piles, we recommended that test program of indicator piles be performed. The number of indicator piles should be approximately 5 percent of the total production piles. Further, we recommend that a Pile Driving Analyzer (PDA) and Case Pile Wave Analysis Program (CAPWAP) be used for each of the indicator piles to evaluate the design, to evaluate the appropriate size of hammer, and to establish refusal criteria for the project. Indicator piles should be 10 feet longer than the preliminary design and should be reinforced to allow cut-off at any length. Upon completion of the indicator pile program, AESCO can provide pile driving recommendations for the project. Swinging or fixed leads should be used to hold the piles vertical. All piles shall be driven their full length into the ground in a continuous operation. Piles shall be driven at the specified location and at a vertical tolerance of no more than 2 percent. If any piles do not meet the above requirements then the foundation shall be redesigned. The "as driven" locations shall be

surveyed by a licensed surveyor. The minimum spacing for these piles should be 2 to 2½ diameters on center; larger spacing will increase the group efficiency.

The equation for determining the group factors for axial loads is presented below.

$$\eta = 1 - \theta \frac{(n-1)m + (m-1)n}{90mn}$$

η = group efficiency factor

m = number of rows of piles

n = number of piles per row

$\theta = \tan^{-1}(B/S)$

B = diameter of single pile

S = center to center spacing of piles

$$P_{ag} = \eta N P_a$$

N = number of piles in group

P_a = allowable upward or downward capacity of single isolated pile

P_{ag}= allowable upward or downward capacity of si pile group

Group effects for lateral loads will be highly dependent on the size and spacing of the group and the lateral loads and should be evaluated once loads and layout have been established. The soil engineer and structural engineer shall design the foundation as necessary.

An accurate record of blow counts shall be maintained by a qualified pile inspector under supervision of the Soil Engineer and shall have, as a minimum, the following information: type of piles, dimension, date and time of start and completion of driving, penetration resistance of pile in blows per foot, the operation speed of the hammer, height of ram drop, and any unusual driving phenomenon. The test pile should be supervised by the Soil Engineer to establish driving criteria.

Since pile driving may cause remote vibrations to existing structures, vibration monitoring of adjacent structures is required. Pilot holes with a diameter of 90 percent of the pile dimension drilled to a minimum depth of 10 feet below the existing ground surface are necessary to minimize vibrations to structures less than 50 feet from the pile driving location. Refusal is considered when driving resistance is more than 50 blows per foot for the specified hammer. If the refusal occurs with a minimum penetration of 20 feet, then load test the pile for twice the designed load. If the pile load test is passed then the pile can be used for the support. When a failure occurs, another pile must be driven adjacent to the refusal with a pilot hole 90% of the pile smallest width to a depth of 30 feet.

Drilled Pier Foundations

As an option, the proposed structures may be supported on typical, reinforced concrete drilled piers. The support from the piers for axial loads will be derived from side friction for axial loads, and from passive soil resistance for lateral and over-turning forces.

Based on our exploratory borings, design parameters are provided in the Load Data for Drilled Piers table in the Appendix. (The table provides for shafts ranging between 18 inches to 60 inches in diameter). The allowable axial loads for the drilled piers are tabulated for various sizes of shafts to a depth of 80 feet. The allowable load is calculated for a safety factor of 2.0. It should be noted that the axial capacity is based on the static strengths of the materials, assuming no liquefaction occurs. The results of liquefaction are typically loss of shear strength. This typically equates to large total and differential settlements should the design earthquake occur. We estimate that settlement of the pier would be less than ½ inch. The minimum spacing for these piers should be 3 diameters on center.

The minimum spacing for these piers should be 3 diameters on center; larger spacing will increase the group efficiency. For clusters greater than 4, the group efficiency should be checked using the "perimeter-shear" method which is the ratio of the perimeter of the group for axial loading given for driven piles.

For preliminary evaluation of lateral loads an equivalent fluid with a density of 150 pounds per cubic foot may be assumed for determining the lateral resistance of the soils against the projected width of the pier to a depth of 10 feet. The maximum lateral resistance should be increased to 1500 pounds per square foot at depths greater than 10 feet below the ground surface. The contribution of lateral resistance to a depth equal to four feet should be neglected. Once actual loading conditions are identified a lateral load analysis can be performed using p-y analysis.

The pier foundation should be designed and constructed in accordance with applicable procedures established by the 2013 CBC and the American Concrete Institute (ACI). The specifications should be patterned after recommendations included in the "Standards and Specifications for the Drilled Shaft Industry" published by the Association for Drilled Shaft Contractors (ADSC). We recommend that potential foundation contractors be pre-qualified with a heavy emphasis on local experience as recommended by ADSC.

Special drilling equipment will likely be required for excavating the pier shaft. Drilling difficulties and caving may be expected due to the sandy material and a high groundwater (6 to 9 feet). The contractor should be prepared to control caving by using temporary casing and/or drilling mud. If temporary casing is used it should be removed as concrete is generally placed

with at least a 3-foot head of concrete maintained within the casing to ensure the minimum required shaft diameter and prevent side wall collapse. If the excavation cannot be pumped dry, concrete may be pumped below the water table using a tremie pipe. Because the foundation design counts on side friction and passive resistance for bearing capacity and lateral stability, casing must be removed. The use of temporary casing is at the discretion of the contractor. The pier shaft should not be left open for any prolonged period of time. Drilling of the piers should be continuously inspected by a representative of AESCO.

Shallow Foundations

Lightly loaded structures outside the main structure, such as possibly the restaurant, may be founded on shallow foundations. To mitigate the potential for settlement and/or liquefaction impact to these structures, the upper 5 feet below the existing surface or 4 feet below the bottom of the footing, whichever is deeper, should be over-excavated. The excavation should extend 5 feet beyond the footprint of the entire slab (where possible). The bottom 3 feet of the excavation should be backfilled with crushed aggregate material. The crushed aggregate should be compacted to a minimum of 95 percent of the maximum dry density (relative compaction) as determined by ASTM D1557. The aggregate material should be wrapped (top, bottom and sides) with geofabric, such as Mirafi 1120N. The remainder of the excavation should then be backfilled and recompacted to at least 95 percent relative compaction as determined by ASTM D1557. Select engineered fill with an Expansion Index of less than 20 may be used as recommended in the "Site Preparation and Earthwork," section below. The side slopes of shallow excavations less than 5 feet high should be cut to a gradient no steeper than 1½:1 (h:v). Steeper excavations should be supported by shoring. Excavations should not extend below an imaginary 1½:1 inclined plane projecting below the bottom edge of adjacent existing foundations and/or utilities unless properly shored or specifically analyzed further. All excavations should be observed by AESCO to confirm that all unsuitable material is removed from beneath the planned construction prior to placing fill.

AESCO recommends continuous and spread footings be a minimum of 18 inches and 24 inches wide, respectively, to mitigate the potential for shear failure. All footings shall have a minimum embedment of 24 inches below the lowest adjacent finished grade in properly placed and compacted fill. The final design of foundation reinforcement shall be performed by the structural engineer.

Assuming these recommendations are followed, an allowable bearing pressure, for dead plus live loads, of 1800 psf is recommended in the design of shallow spread footings and 1400 psf for continuous footings supported on engineered fill. These pressures can be increased by 33% for

temporary loads, such as, seismic loads and wind loads. A passive soil resistance of 150 pcf and a coefficient of friction of 0.35 may be assumed for design against lateral forces.

Any undocumented fill should be removed and replaced with compacted engineered fill. A representative of AESCO should confirm the depth of fill at the time of construction.

All foundations adjacent to any existing buildings, walkways and separately poured porches should be tied into the adjacent slabs with #5 rebar, 30 inches in length, on 18-inch centers, embedded a minimum of 8 inches into the existing, or adjacent slabs, to reduce separation and differential settlement.

Total settlement of the footings is estimated to be 1 inch or less. Differential settlement between similarly loaded footings is expected to be about one-half the total settlement. This estimate is applicable for static conditions.

Concrete Slabs on Grade

We anticipate that concrete slabs on grade for the new structures will extend 5 inches above the final adjacent grade and will be have an 18-inch thickened edge on non-bearing walls. We recommend that the slab sections be properly reinforced with a minimum of #4 bars, at 16 inches, on center, positioned mid height placed on prepared subgrade. The actual reinforcement should be designed by the structural engineer. Slabs on grade should be underlain by the prepared subgrade as recommended in the "Site Preparation and Earthwork," section below. Selective grading will be required to choose the most granular material to place beneath the slabs. A ten mil PVC or polyethylene membrane (vapor barrier) shall be placed on top of a 4-inch thick gravel layer which should be provided beneath all interior slabs to prevent moisture migration. Outside slabs (sidewalks, drives, etc.) should be constructed with expansion joints placed at maximum 12 foot spacing each way to minimize cracking due to shrinkage and expansion of the concrete.

Recommendations for Walls Below Grade

Portions of the structure will be below grade, such as, the mechanical room and the bottoms of the pools. These should be designed for buoyant conditions below an elevation of +5' (AMSL).

Lateral Earth Pressures

Walls below grade will be subjected to lateral earth pressures from the retained soils and surcharge loads. Accordingly, these structures should be designed to resist appropriate lateral earth pressures.

For design purposes, a triangular distribution of lateral earth pressures with an equivalent fluid pressure of 80 pounds per cubic foot (pcf) should be used in design of walls below grade for a restrained condition. This assumes a horizontal grade behind the wall.

Total lateral earth pressures acting on the wall during a seismic event will likely include the static force and the dynamic increment. Using the Mononobe-Okabe procedure, a dynamic lateral earth pressure increment (for a 0.43g) peak ground acceleration based on 10% probability of being exceeded in 50 years, (CGS) of 35H may be assumed for design purposes, where H (in units of feet) is the height of the soil behind the wall. This dynamic increment should be applied to the wall as a triangle pressure over the wall height starting from the bottom of the wall to the top, and are added to the static earth pressures. The lateral earth pressures recommended above are based upon the assumption that the backfill is granular, the ground surface behind the wall is level, and the wall backfill is well drained. The pressure should be increased by 35 percent for sloping backfill with a 2:1 (H:V) slope.

The design values assume free-draining backfill materials are placed behind the wall. Surcharge pressures (dead or live) should be added to the above lateral earth pressures where surcharge loads may be located adjacent to the wall. Surcharge pressures should be applied as a uniform (rectangular) pressure distribution by using a pressure equal to 0.5 times the surcharge pressures. Vertical surcharges set back behind the wall a horizontal distance greater than the wall height need not be added to the design pressure. The above coefficient assumes a uniform surcharge load.

Wall Backfill

Backfill behind walls below grade should consist of granular backfill that is placed directly above and behind the drain material. To reduce the potential for settlement of backfill, it is essential that wall backfill be properly compacted in lifts. The minimum compaction standard for wall backfill should be 90 percent relative compaction in accordance with ASTM D1557. In the event that the wall backfill will support structures or facilities, the compaction standard should be increased to 95 percent relative compaction. Heavy compaction equipment should not be used within 5 feet of the wall. Small hand-operated compaction equipment should be used adjacent to the wall so as not to overstress the wall. The lift thickness with the smaller equipment should not be more than six inches.

Excavation and Shoring

The material can be classified as soil type (C) based on CAL-OSHA classification. Temporary construction slopes should not be steeper than 1½:1 (H:V). Alternatively, shoring may be used to support the excavation. For the proposed foundation excavation, shoring may consist of soldier piles and lagging or another suitable system to retain the sides of the excavation. Shoring should be designed by a licensed engineer experienced in shoring design and submitted for our review.

For the design of cantilever shoring, a minimum equivalent fluid pressure of 35H psf per foot of depth below grade may be used, where (H) is the height in feet. For the design of braced shoring supporting a sloping grade, we recommend such shoring be designed using a rectangular-shaped distribution of lateral earth pressure for a maximum earth pressure of 25 (in psf).

For the design of soldier piles spaced at least three diameters on centers, the passive resistance of the soils adjacent to the piles may be assumed to be 150 psf per foot of embedment depth for the projected width of the pile, up to 1500 psf maximum. The effective width of soldier piles installed can be increased by an adjustment factor of 1.6. The soldier piles may be installed in drilled excavations. Soldier pile members placed in drilled holes should be properly backfilled with sand/cement slurry or lean concrete in order to develop the required passive resistance.

The design of the shored excavation should be performed by an engineer knowledgeable and experienced with the on-site soil conditions. The contractor should be aware that slope height, slope inclination or excavation depths should in no case exceed those specified in local, state or federal safety regulations, e.g. OSHA Health and Safety Standards for Excavation, 29 CFR Part 1926, or successor regulations. Such regulations are strictly enforced and, if not followed, the owner or the contractor could be liable for substantial penalties.

Site Preparation and Earthwork

All grading and site preparation should be observed by experienced personnel reporting to the project Geotechnical Engineer. Field monitoring services are an essential continuation of prior studies to confirm and correlate the findings and prior recommendations with the actual subsurface conditions exposed during construction, and to confirm that suitable fill soils are placed and properly compacted.

The site should be cleared of vegetation, debris, concrete, organic matter, abandoned utility lines, contaminated soils (if any), and unsuitable material. Any existing fill encountered during site preparation should be excavated to the depth of the fill and to a horizontal distance equal to the

depth of excavation. A California Licensed Geotechnical Engineer should confirm the depth of fill at the time of construction. As a minimum, the upper three feet below the existing or finished surface should be over-excavated and recompacted to at least 90 percent relative compaction as determined by ASTM D1557 at moisture contents 1 to 3 percent above optimum moisture. For the shallow foundation alternative, a deeper excavation will be required, as described in the Shallow Foundations section, above. The bottom of the excavation shall be inspected by the Geotechnical Engineer to confirm competent soil is reached. The side slopes of shallow excavations should be cut at a gradient no steeper than 1:1 (horizontal to vertical), while excavations greater than 5 feet high should be cut to a gradient no steeper than 1½:1. Excavations should not extend below an imaginary 1.5:1 inclined plane projecting below the bottom edge of adjacent existing foundations and/or utilities unless properly shored or specifically analyzed further. All excavations should be observed by a California Licensed Geotechnical Engineer to confirm that all unsuitable material is removed from beneath the planned construction prior to placing fill.

The bottom of all excavations should be lightly compacted and inspected by the Geotechnical Engineer prior to the placement of any fill. Excavations below the final grade level should be properly backfilled using approved fill material. The backfill and any additional fill should be placed in loose lifts less than 8 inches thick, moisture conditioned to 0 to 4 percent above optimum water content, and compactedto a minimum of 90 percent relative compaction in accordance with ADTM D1557. Engineered fill should consist of soils with a maximum particle size of 3 inches, at least 80 percent passing the ¾-inch sieve and with an expansion index not greater than 20. Fill materials should be free of construction debris, roots, organic matter, rubble, contaminated soils, and any other unsuitable or deleterious material as determined by the Geotechnical Engineer. Any imported fill material used shall be analyzed for acceptability by the Geotechncial Engineer prior to importing it to the site for use as engineered fill.

A representative of the Geotechncial Engineer should observe all footing and slab subgrade surfaces and confirm that the exposed materials are firm. If loose, spongy, soft or other unacceptable materials, including undocumented fill, are encountered in the subgrade they should be removed to firm materials as determined by the Geotechnical Engineer's representative and replaced with either concrete or compacted engineered fill.

Utility Trenches

It is anticipated that the on-site soils will provide suitable support for underground utilities and piping that may be installed. Any soft and/or unsuitable material encountered at the bottom of excavations for such facilities should be removed and be replaced with an adequate bedding material. A non-expansive granular material with a sand equivalent greater than 30 should be used for bedding and shading of utilities.

On-site material should be suitable for backfill of utility and pipe trenches from one foot above the top of the pipe to the final ground surface, provided the material is free of organic matter, deleterious substances, and contamination. Trench backfill should be mechanically placed and compacted in maximum 8-inch lifts to at least 90 percent of the maximum dry density as determined by ASTM Test Method D 1557 (i.e. 90 percent relative compaction) at 1 to 2 percent above optimum moisture content. Where trenches are placed beneath slabs or footings the backfill shall satisfy the gradation and expansion index requirements of engineered fill (see "Site Preparation and Earthwork" section, above). Trenches in the footprint of the pavement shall be backfilled and compacted to a minimum of 90 percent. Flooding or jetting for placement and compaction of backfill is not recommended.

Limitations

It must be recognized that conclusions reached in this report are based on conditions, which exist at the boring location and are assumed to exist over the entire site. In any subsoil investigation, it is necessary to assume that the subsoil conditions between boring(s) do not change significantly. The number of the borings, locations, and spacing are chosen in such a manner as to decrease the possibility of undiscovered anomalies, while considering the nature of loading, size, existing structures, and cost of the project. Note that the boring(s) were placed as close to the location of the proposed structure(s) as possible. The boring locations are approximate and surveying is beyond the scope of our work. Consequently, careful observations must be made during construction to detect significant deviations of actual conditions throughout the construction area from those inferred from the exploratory borings.

In the event that significant changes in design loads or structural characteristics are made, AESCO should be retained to review our original design recommendations and their applicability to the revised design plans. In this way, any required supplemental recommendations can be made in a timely manner.

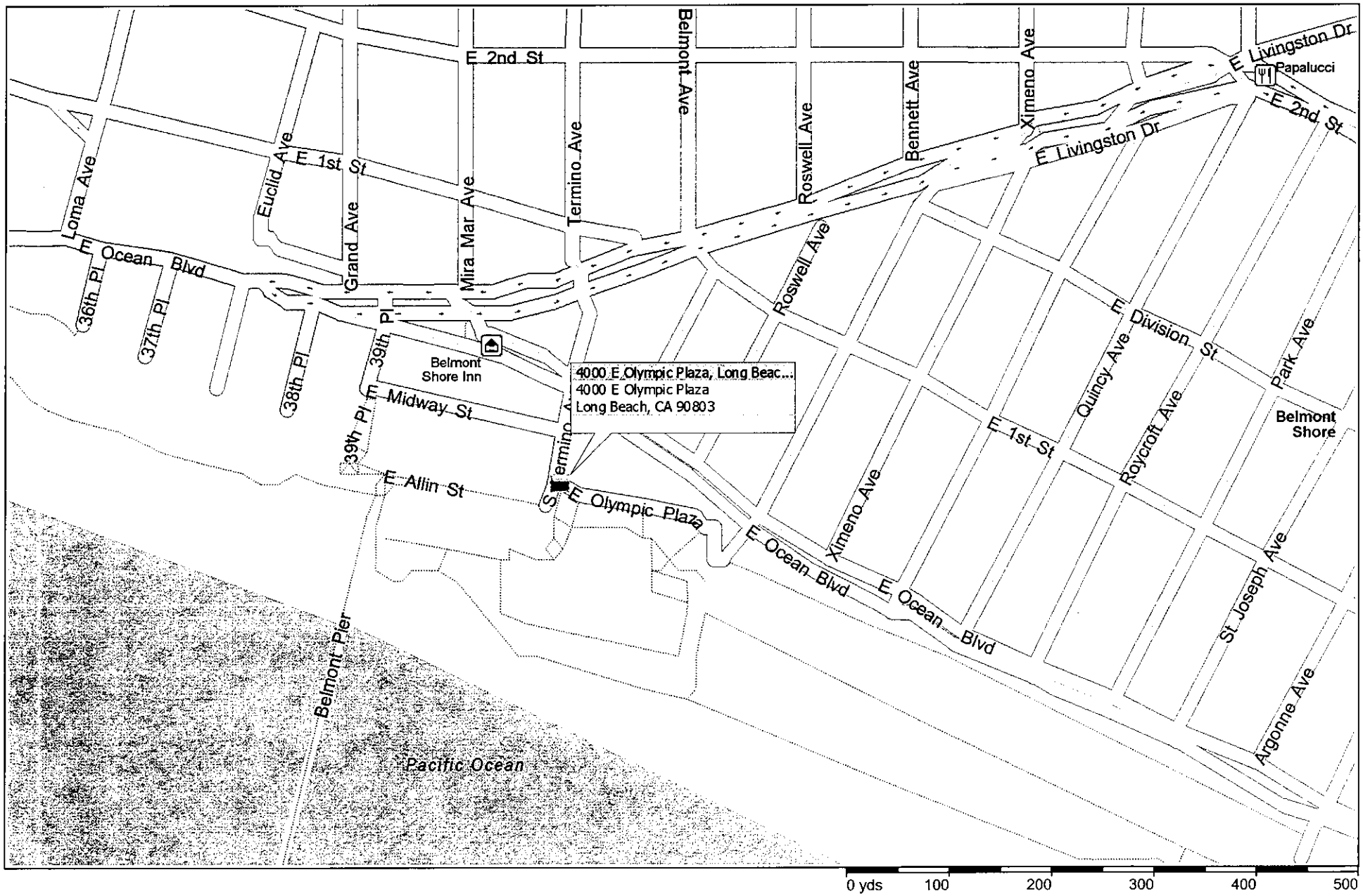
Should any unusual conditions be encountered during construction, this office should be notified immediately so that further investigations and supplemental recommendations can be made. Geotechnical observations and testing should be provided on a continuous basis during grading, excavation, and installation of the foundations. If parties other than AESCO are engaged to provide geotechnical services during construction they will be required to assume the full responsibility for the geotechnical phase of the project by adhering to the recommendations of this report.

Analysis by:

Russell J. Scharlin, P.E., GE

APPENDIX
SITE VICINITY PLAN
SITE PLAN

20140185



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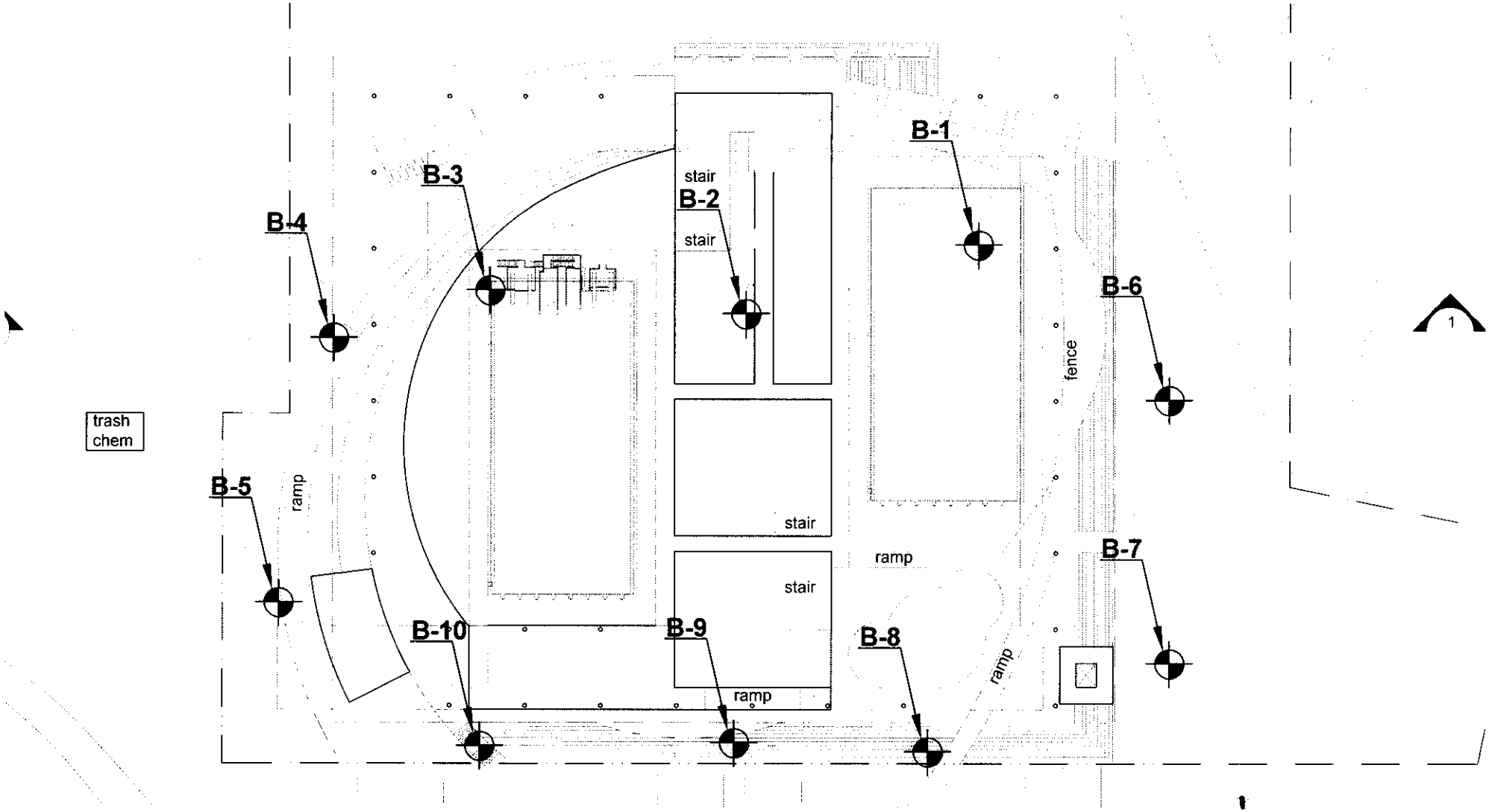


LEGEND

B-1 Approximate Location of Boring

AESCO

City of Long Beach		
Project No. : 20140185-C8050	Scale: *1 inch ≅ 75 feet*	
Site Name: Harley Ellis Devereaux Belmont Plaza Pool Rebuild-Revitalization		
Site Address: 4000 Olympic Plaza, Long Beach, CA		
SITE PLAN	Date: 4-24-2014	Figure 1

APPENDIX
LOGS OF BORINGS B-1 through B-10

MAJOR DIVISION			GRAPHIC SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVEL (LITTLE OR NO FINES)		GW	WELL GRADED GRAVELS, GRAVEL SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY GRADED GRAVELS, GRAVEL SAND MIXTURES, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVEL WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL SAND SILT MIXTURE
				GC	CLAYEY GRAVELS, GRAVEL SAND CLAY MIXTURES
MORE THAN 50% BY WEIGHT OF MATERIAL IS LARGER THAN 200 SIEVE	SAND AND SANDY SOILS	CLEAN SAND (LITTLE OR NO FINES)		SW	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	SANDS WITH FINE (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND-SILT MIXTURES
				SC	CLAYEY SANDS, SAND-CLAY MIXTURES
FINE GRAINED SOILS	SILTS AND CLAYS	LIQUID LIMIT <50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
MORE THAN 50% BY WEIGHT OF MATERIAL IS SMALLER THAN 200 SIEVE	SILTS AND CLAYS	LIQUID LIMIT >50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLATICITY, FAT CLAYS
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

AESCO Technologies, Inc.

UNIFIED SOIL CLASSIFICATION SYSTEM

KEY

- ☒ Split Spoon Sample (SPT)
- C California Modified Sample
- Hand Auger Sample
- ▼ Ground Water Level
- N SPT Blows/ft
- P Penetrometer TSF

LOG OF BORING NO. B - 1

AESCO

Project: Belmont Plaza Pool Rebuild-Revitalization
Location: 4000 East Olympic Plaza, Long Beach, CA

WATER: Encountered at 6 Feet

Client: Harley Ellis Devereaux
Date: 04/03/14
Logger:
Project No. 20140185-C8050

DRILLING: Hollow Stem Auger with Drilling Mud

FIELD DATA: SOIL SYMBOL		DEPTH (FT)	TESTS: N= / T= / P=	LABORATORY DATA: MOISTURE CONTENT %	DRY DENSITY PCF	LIQUID LIMITS %	PLASTIC LIMITS %	PLASTICITY INDEX %	Unconfined Comp. TSF	Unconfined Comp. Strain %	PASSING 200 SIEVE %	DIRECT SHEAR COHESION PSF	DIRECT SHEAR ANGLE Deg	EXPANSION INDEX	DESCRIPTION OF STRATUM
		3		6.1											FILL-Red-brown silty SAND (SM), moist, 0.1ppm
		5	N=27	3.6											Gray-brown SAND (SP), medium dense, dry, coarse grained, 0.3 ppm
	C	7	N=19 P=0.5	13.9	107.1						1.2				Saturated below 6', 0.2 ppm
		8													Gray-brown sandy SILT (ML), very soft, saturated
		10	N=PUSH/ 18"	24.3							78.5				0.2 ppm
		13													
	C	15	N=1												No sample recovery at 13'
		18													
		20	N=PUSH/ 18"	35.4											Brown silty CLAY (CL), very soft, saturated, 0.1 ppm
		23													
	C	25	N=19	27.2	103.4						81.3				Very stiff at 23', 0.2 ppm
		28													
		30	N=27	30.8											Increase in density at 28', 0.2 ppm
		33													
	C	35	N=9 P=0.5	30.3	98.6										Stiff, w/interbedded gray sand at 35', 0.2 ppm
		38													Gray SAND/silty SAND (SP/SM), dense, saturated, 0.6 ppm
		40	N=40	15.2							9.6				
		43													
	C	45	N=26	18.4											Medium dense at 43', 0.2 ppm
		48													
		50	N=72	12.3							7.6				Very dense, coarse grained at 48', 0.3 ppm

Boring Terminated at 50 Feet

TUBE SAMPLE
AUGER SAMPLE
C CALIFORNIA MODIFIED SAMPLER
SPLIT SPOON
NO RECOVERY

Ground Water Level
Hydrostatic Ground Water Level

SM SP ML CL SP/SM

N= SPT, BLOWS/FT
T= THD.BLOWS/FT
P= HAND PEN..TSF
0.1 ppm=PID READING

REMARKS:
NP: Non Plastic Materials
* Remolded Samples

LOG OF BORING NO. B - 3

AESCO

Project: Belmont Plaza Pool Rebuild-Revitalization
Location: 4000 East Olympic Plaza, Long Beach, CA

WATER: Encountered at 7 Feet

Client: Harley Ellis Devereaux
Date: 04/08/14
Logger:
Project No. 20140185-C8050

DRILLING: Hollow Stem Auger with Drilling Mud

FIELD DATA		TESTS	LABORATORY DATA											DESCRIPTION OF STRATUM
SOIL SYMBOL	DEPTH (FT)	N= T= P=	MOISTURE CONTENT %	DRY DENSITY PCF	LIQUID LIMITS %	PLASTIC LIMITS %	PLASTICITY INDEX %	Unconfined Comp. TSF	Unconfined Comp. Strain %	PASSING 200 SIEVE %	DIRECT SHEAR COHESION PSF	DIRECT SHEAR ANGLE Deg	EXPANSION INDEX	
	3		10.4											FILL-Brown silty SAND (SM), moist, 0.0 ppm
	5	N=33	1.4											Gray-brown SAND (SP), dense, dry, 0.1 ppm
C	7	N=21 P=0.5	5.3	102.4						4.9				Medium dense, moist at 5', 0.1 ppm
	8													Saturated below 7'
	10	N=19	21.1											Medium grained at 8', 0.1 ppm
														Brown sandy CLAY (CL), very stiff, saturated
	13													
C	15	N=17	15.9	120.4						87.0				0.0 ppm
	18													
	20	N=16	27.0											0.0 ppm
	23													
C	25	N=20	21.0	113.1										0.0 ppm
	28													
	30	N=27	23.9							65.9				Increase in sand and density at 28', 0.1 ppm
	33													
C	35	N=19 P=2.5	28.7	96.8										Decrease in sand and density at 33', 0.1 ppm
														Brown silty SAND (SM), medium dense, saturated, 0.1 ppm
	38													
	40	N=30	27.5							40.0				
	43													
C	45	N=28	26.5	104.6										Dark gray at 43', 0.1 ppm
	48													
	50	N=44	19.2							21.2				Dense, increase in sand at 48', 0.0 ppm

TUBE SAMPLE
AUGER SAMPLE
C CALIFORNIA MODIFIED SAMPLER
SPLIT SPOON
NO RECOVERY

Ground Water Level
Hydrostatic Ground Water Level

N= SPT, BLOWS/FT
T= THD,BLOWS/FT
P= HAND PEN ,TSF
0.1 ppm=PID READING

SM SP CL SP/SM ML

REMARKS:
NP: Non Plastic Materials
* Remolded Samples

LOG OF BORING NO. B - 3 (continued)

AESCO

Project: Belmont Plaza Pool Rebuild-Revitalization
Location: 4000 East Olympic Plaza, Long Beach, CA
WATER: Encountered at 6 Feet

Client: Harley Ellis Devereaux
Date: 04/08/14
Logger:
Project No. 20140185-C8050
DRILLING: Hollow Stem Auger with Drilling Mud

FIELD DATA			TESTS	LABORATORY DATA											DESCRIPTION OF STRATUM
SOIL SYMBOL		DEPTH (FT)	N= T= P=	MOISTURE CONTENT %	DRY DENSITY PCF	LIQUID LIMITS %	PLASTIC LIMITS %	PLASTICITY INDEX %	Unconfined Comp. TSF	Unconfined Comp. Strain %	PASSING 200 SIEVE %	DIRECT SHEAR COHESION PSF	DIRECT SHEAR ANGLE Deg	EXPANSION INDEX	
		53													Dark gray silty SAND (SM), dense, saturated
	C	55	N=30	24.3	107.3										Medium dense, coarse grained, w/interbedded sandy clay at 53', 0.0 ppm
		58													Brown-gray SAND/silty SAND (SP/SM), very dense, saturated, 0.1 ppm
		60	N=50	15.6							9.5				
		63													
	C	65	N=38	16.8	120.7						10.0				Dense at 63', 0.0 ppm
		68													
		70	N=29	26.9							50.1				Dark gray sandy SILT (ML), very stiff, saturated, w/interbedded sand, 0.0 ppm
		73													
	C	75	N=30	19.1	113.2										Increase in sand at 73', 0.1 ppm
		78													Dark gray SAND (SP), very dense, saturated
		80	N=64	16.7							11.8				

Boring Terminated at 80 Feet

TUBE SAMPLE
AUGER SAMPLE
C CALIFORNIA MODIFIED SAMPLER
SPLIT SPOON
NO RECOVERY
Ground Water Level
Hydrostatic Ground Water Level
N= SPT, BLOWS/FT
T= THD,BLOWS/FT
P= HAND PEN.,TSF
0.1 ppm=PID READING
SM SP ML CL SP/SM

REMARKS:
NP: Non Plastic Materials
* Remolded Samples

LOG OF BORING NO. B - 4

AESCO

Project: Belmont Plaza Pool Rebuild-Revitalization
Location: 4000 East Olympic Plaza, Long Beach, CA
WATER: Encountered at 9 Feet

Client: Harley Ellis Devereaux
Date: 04/03/14
Logger:
Project No. 20140185-C8050
DRILLING: Hollow Stem Auger with Drilling Mud

FIELD DATA		TESTS	LABORATORY DATA											DESCRIPTION OF STRATUM
SOIL SYMBOL	DEPTH (FT)	N= T= P=	MOISTURE CONTENT %	DRY DENSITY PCF	LIQUID LIMITS %	PLASTIC LIMITS %	PLASTICITY INDEX %	Unconfined Comp TSF	Unconfined Comp Strain %	PASSING 200 SIEVE %	DIRECT SHEAR COHESION PSF	DIRECT SHEAR ANGLE Deg	EXPANSION INDEX	
	3		12.2											FILL-Brown silty SAND (SM), moist, 0.1 ppm
	5	N=25	5.8											Light gray-brown SAND (SP), medium dense, moist, 0.2 ppm
C	7	N=15 P=0.5	6.4	108.7						5.9				Light gray-brown SAND/silty SAND (SP/SM), medium dense, moist, medium grained, 0.2 ppm
	8													
	10	N=23	15.8											Coarse grained at 8', 0.2 ppm Saturated below 9'
	13													Brown sandy CLAY (CL), stiff, satruated
C	15	N=14	18.6	117.8						82.2				0.2 ppm
	18													
	20	N=11	25.7											W/seashells at 18', 0.2 ppm
	23													
C	25	N=16	22.6	107.8										0.1 ppm
	28													
	30	N=29	19.0							75.7				Dark gray sandy SILT (ML), very stiff, saturated, 0.2 ppm
	33													
C	35	N=14 P=0.5	26.4	99.6										Stiff, w/interbedded sand at 33', 0.2 ppm

Boring Terminated at 35 Feet

TUBE SAMPLE
AUGER SAMPLE
C CALIFORNIA MODIFIED SAMPLER
SPLIT SPOON
NO RECOVERY

Ground Water Level
Hydrostatic Ground Water Level

N= SPT, BLOWS/FT
T= THD,BLOWS/FT
P= HAND PEN ,TSF
0.1 ppm=PID READING

SM SP SP/SM CL ML

REMARKS:
NP: Non Plastic Materials
* Remolded Samples

LOG OF BORING NO. B - 5

AESCO

Project: Belmont Plaza Pool Rebuild-Revitalization
Location: 4000 East Olympic Plaza, Long Beach, CA
WATER: Encountered at 7 Feet

Client: Harley Ellis Devereaux
Date: 04/08/14
Logger:
Project No. 20140185-C8050
DRILLING: Hollow Stem Auger with Drilling Mud

FIELD DATA			TESTS	LABORATORY DATA											DESCRIPTION OF STRATUM
SOIL SYMBOL		DEPTH (FT)	N= T= P=	MOISTURE CONTENT %	DRY DENSITY PCF	LIQUID LIMITS %	PLASTIC LIMITS %	PLASTICITY INDEX %	Unconfined Comp. TSF	Unconfined Comp. Strain %	PASSING 200 SIEVE %	DIRECT SHEAR COHESION PSF	DIRECT SHEAR ANGLE Deg	EXPANSION INDEX	
		3													FILL-Brown silty SAND (SM), 0.3 ppm
		5	N=33												Gray-brown SAND (SP), dense, wet, 5.7 ppm
	C	7	N=18												Medium dense, 4.3 ppm
		8													Saturated below 7'
		10	N=7												Gray, loose, coarse grained, w/seashells at 8', 41.0 ppm
		13													
	C	15	N=31												Dense, 59.0 ppm
		18													Gray silty SAND (SM), medium dense, saturated, w/interbedded silt, w/seashells, 16.5 ppm
		20	N=20												
		23													Gray SAND (SP), medium dense, saturated, 0.7 ppm
	C	25	N=11												
		28													Gray sandy SILT (ML), very stiff, saturated, 0.7 ppm
		30	N=26												
		33													
	C	35	N=31												0.2 ppm
		38													Gray SAND (SP), dense, saturated, 0.5 ppm
		40	N=34												
		43													
	C	45	N=34												0.2 ppm
		48													
		50	N=31												0.6 ppm

TUBE SAMPLE
AUGER SAMPLE
C CALIFORNIA MODIFIED SAMPLER
SPLIT SPOON
NO RECOVERY
Ground Water Level
Hydrostatic Ground Water Level
SM
SP
ML

N= SPT, BLOWS/FT
T= THD,BLOWS/FT
P= HAND PEN ,TSF
0.1 ppm=PID READING

REMARKS:
NP: Non Plastic Materials
* Remolded Samples

LOG OF BORING NO. B - 5 (continued)

AESCO

Project: Belmont Plaza Pool Rebuild-Revitalization
Location: 4000 East Olympic Plaza, Long Beach, CA

WATER: Encountered at 6 Feet

Client: Harley Ellis Devereaux
Date: 04/03/14
Logger:
Project No. 20140185-C8050

DRILLING: Hollow Stem Auger with Drilling Mud

FIELD DATA			TESTS	LABORATORY DATA											DESCRIPTION OF STRATUM
SOIL SYMBOL		DEPTH (FT)	N= T= P=	MOISTURE CONTENT %	DRY DENSITY PCF	LIQUID LIMITS %	PLASTIC LIMITS %	PLASTICITY INDEX %	Unconfined Comp. TSF	Unconfined Comp. Strain %	PASSING 200 SIEVE %	DIRECT SHEAR COHESION PSF	DIRECT SHEAR ANGLE Deg	EXPANSION INDEX	
		53													Gray SAND (SP), dense, saturated
	C	55	N=41												Coarse grained at 53', 0.6 ppm
		58													
		60	N=27												Medium dense at 58', 0.2 ppm
		63													
	C	65	N=38												Dense at 63', 0.1 ppm
		68													
		70	N=31												0.3 ppm
		73													
	C	75	N=50/3"												Very dense at 73', 0.1 ppm
		78													
		80	N=50/6"												0.1 ppm

Boring Terminated at 80 Feet

TUBE SAMPLE
AUGER SAMPLE
C CALIFORNIA MODIFIED SAMPLER
SPLIT SPOON
NO RECOVERY
SP
Ground Water Level
Hydrostatic Ground Water Level

N= SPT, BLOWS/FT
T= THD,BLOWS/FT
P= HAND PEN.,TSF
0.1 ppm=PID READING

REMARKS:
NP: Non Plastic Materials
* Remolded Samples

LOG OF BORING NO. B - 6

AESCO

Project: Belmont Plaza Pool Rebuild-Revitalization
Location: 4000 East Olympic Plaza, Long Beach, CA
Client: Harley Ellis Devereaux
Date: 04/03/14
Logger:
Project No. 20140185-C8050

WATER: Encountered at 7 Feet

DRILLING: Hollow Stem Auger with Drilling Mud

FIELD DATA		TESTS	LABORATORY DATA											DESCRIPTION OF STRATUM
SOIL SYMBOL	DEPTH (FT)	N= T= P=	MOISTURE CONTENT %	DRY DENSITY PCF	LIQUID LIMITS %	PLASTIC LIMITS %	PLASTICITY INDEX %	Unconfined Comp. TSF	Unconfined Comp. Strain %	PASSING 200 SIEVE %	DIRECT SHEAR COHESION PSF	DIRECT SHEAR ANGLE Deg	EXPANSION INDEX	
	3		8.2											FILL-Brown silty SAND (SM), moist, 0.1 ppm
	5	N=22	12.8											Gray-brown SAND/silty SAND (SP/SM), medium dense, moist, 0.1 ppm
C	7	N=18 P=0.5	12.6	111.4						5.4				0.1 ppm
	8													Saturated below 7'
	10	N=17	20.4											0.1 ppm
	13													Brown sandy CLAY (CL),soft, saturated, 0.1 ppm
C	15	N=4	21.9	111.4						73.6				
	18													
	20	N=4	21.8											0.1 ppm
	23													
C	25	N=23	21.8	107.8										Very stiff, 0.1 ppm
	28													
	30	N=19	27.5							84.9				0.1 ppm
	33													
C	35	N=17 P=1.0	29.9	99.1										0.1 ppm
	38													Gray-brown silty SAND (SM), medium dense, saturated, 0.1 ppm
	40	N=26	27.1							32.4				
	43													
C	45	N=17	28.2	99.5										0.1 ppm
	48													
	50	N=25	27.8							14.6				Increase in sand, 0.1 ppm

TUBE SAMPLE
AUGER SAMPLE
C CALIFORNIA MODIFIED SAMPLER
SPLIT SPOON
NO RECOVERY
Ground Water Level
Hydrostatic Ground Water Level
SM
SP/SM
CL

N= SPT, BLOWS/FT
T= THD,BLOWS/FT
P= HAND PEN.,TSF
0.1 ppm=PID READING

REMARKS:
NP: Non Plastic Materials
* Remolded Samples

LOG OF BORING NO. B - 6 (continued)

AESCO

Project: Belmont Plaza Pool Rebuild-Revitalization
Location: 4000 East Olympic Plaza, Long Beach, CA
WATER: Encountered at 6 Feet

Client: Harley Ellis Devereaux
Date: 04/03/14
Logger:
Project No. 20140185-C8050
DRILLING: Hollow Stem Auger with Drilling Mud

FIELD DATA		TESTS	LABORATORY DATA											DESCRIPTION OF STRATUM
SOIL SYMBOL	DEPTH (FT)	N= T= P=	MOISTURE CONTENT %	DRY DENSITY PCF	LIQUID LIMITS %	PLASTIC LIMITS %	PLASTICITY INDEX %	Unconfined Comp TSF	Unconfined Comp Strain %	PASSING 200 SIEVE %	DIRECT SHEAR COHESION PSF	DIRECT SHEAR ANGLE Deg	EXPANSION INDEX	
	53													Gray-brown silty SAND (SM), medium dense, saturated
C	55	N=30	20.4	110.4										Coarse grained, w/interbedded brown clay at 53', 0.1 ppm
	58													
	60	N=24	16.7							13.4				Gray at 58', 0.1 ppm
	63													
C	65	N=29 P=2.0	23.6	108.3										Gray-brown, w/interbedded brown clay at 63', 0.1 ppm
	68													
	70	N=42	20.1							47.7				Gray, dense, 0.1 ppm
	73													
C	75	N=50/6"	21.7	115.7										Brown, very dense, coarse grained at 73', 0.1 ppm
	78													Gray SAND/silty SAND (SP/SM), very dense, 0.1 ppm
	80	N=50/6"	14.5							11.5				

Boring Terminated at 80 Feet

TUBE SAMPLE
AUGER SAMPLE
C CALIFORNIA MODIFIED SAMPLER
SPLIT SPOON
NO RECOVERY
Ground Water Level
Hydrostatic Ground Water Level
SM
SP/SM

N= SPT, BLOWS/FT
T= THD,BLOWS/FT
P= HAND PEN.,TSF
0.1 ppm=PID READING

REMARKS:
NP: Non Plastic Materials
* Remolded Samples

LOG OF BORING NO. B - 7

AESCO

Project: Belmont Plaza Pool Rebuild-Revitalization
Location: 4000 East Olympic Plaza, Long Beach, CA
Client: Harley Ellis Devereaux
Date: 04/03/14
Logger:
Project No. 20140185-C8050

WATER: Encountered at 5 Feet

DRILLING: Hollow Stem Auger with Drilling Mud

FIELD DATA			TESTS	LABORATORY DATA											DESCRIPTION OF STRATUM
SOIL SYMBOL		DEPTH (FT)	N= T= P=	MOISTURE CONTENT %	DRY DENSITY PCF	LIQUID LIMITS %	PLASTIC LIMITS %	PLASTICITY INDEX %	Unconfined Comp TSF	Unconfined Comp Strain %	PASSING 200 SIEVE %	DIRECT SHEAR COHESION PSF	DIRECT SHEAR ANGLE Deg	EXPANSION INDEX	
		3		10.1											FILL-Brown silty SAND (SM), moist, w/gravel, 0.1 ppm
		5	N=8	14.0							40.2				Brown clayey SAND (SC), loose, moist, 0.1 ppm
	C	7 8	N=13 P=1.0	21.3	102.9						5.0				Brown silty SAND (SM), medium dense, saturated, 0.1 ppm
		10	N=15	23.9							4.8				Gray SAND (SP), medium dense, saturated, 0.1 ppm
	C	13 15	N=26	25.4	99.0										Gray SAND/silty SAND (SP/SM), medium dense, saturated, coarse grained, 0.1 ppm W/organics at 13'
		18 20	N=20	22.7											Gray SAND (SP), medium dense, saturated, 0.1 ppm
	C	23 25	N=11 P=0.5	24.9	103.6						38.5				Gray silty SAND (SM), medium dense, saturated 0.0 ppm
		28 30	N=19	23.5											W/seashells and interbedded sandy clay at 28', 0.1 ppm
	C	33 35	N=31	22.7	108.2						24.9				Dark gray, dense at 33', 0.1 ppm

Boring Terminated at 35 Feet

TUBE SAMPLE
AUGER SAMPLE
C CALIFORNIA MODIFIED SAMPLER
SPLIT SPOON
NO RECOVERY
Ground Water Level
Hydrostatic Ground Water Level
SM
SC
SP
SP/SM

N= SPT, BLOWS/FT
T= THD,BLOWS/FT
P= HAND PEN.,TSF
0.1 ppm=PID READING

REMARKS:
NP: Non Plastic Materials
* Remolded Samples

LOG OF BORING NO. B - 8

AESCO

Project: Belmont Plaza Pool Rebuild-Revitalization
Location: 4000 East Olympic Plaza, Long Beach, CA

Client: Harley Ellis Devereaux
Date: 04/04/14
Logger:
Project No. 20140185-C8056

WATER: Encountered at 5 Feet

DRILLING: Hollow Stem Auger with Drilling Mud

FIELD DATA		TESTS	LABORATORY DATA											DESCRIPTION OF STRATUM
SOIL SYMBOL	DEPTH (FT)	N= T= P=	MOISTURE CONTENT %	DRY DENSITY PCF	LIQUID LIMITS %	PLASTIC LIMITS %	PLASTICITY INDEX %	Unconfined Comp. TSF	Unconfined Comp. Strain %	PASSING 200 SIEVE %	DIRECT SHEAR COHESION PSF	DIRECT SHEAR ANGLE Deg	EXPANSION INDEX	
	3		8.2											FILL-Brown silty SAND (SM), moist, 0.1 ppm
	5	N=26	5.9											Gray-brown SAND (SP), medium dense, moist, 0.1 ppm
C	7	N=18	11.5	104.7						1.8				Gray at 5', 0. 1ppm
	8													
	10	N=20	23.3											Gray-brown at 8', 0.1 ppm
	13													Gray SAND/silty SAND (SP/SM), loose, saturated, 0.1 ppm
C	15	N=10 P=0.5	28.4	98.7						9.7				
	18													
	20	N=17	22.1											Medium dense at 18', 0.1 ppm
	23													
C	25	N=9 P=0.5	26.0	103.5										Loose, w/seashells at 23', 0.1 ppm
	28													Brown sandy SILT (ML), stiff, saturated, 0.0 ppm
	30	N=12	23.2							56.7				
	33													Gray SAND/silty SAND (SP/SM), medium dense,saturated, 0.1 ppm
C	35	N=19	24.1	105.9						8.4				

Boring Terminated at 35 Feet

TUBE SAMPLE
AUGER SAMPLE
C CALIFORNIA MODIFIED SAMPLER
SPLIT SPOON
NO RECOVERY

Ground Water Level
Hydrostatic Ground Water Level

SM SP SP/SM ML

N= SPT, BLOWS/FT
T= THD,BLOWS/FT
P= HAND PEN.,TSF
0.1 ppm=PID READING

REMARKS:
NP: Non Plastic Materials
* Remolded Samples

LOG OF BORING NO. B - 9

AESCO

Project: Belmont Plaza Pool Rebuild-Revitalization
Location: 4000 East Olympic Plaza, Long Beach, CA
WATER: Encountered at 7 Feet

Client: Harley Ellis Devereaux
Date: 04/08/14
Logger:
Project No. 20140185-C8050
DRILLING: Hollow Stem Auger with Drilling Mud

FIELD DATA		TESTS	LABORATORY DATA											DESCRIPTION OF STRATUM
SOIL SYMBOL	DEPTH (FT)	N= T= P=	MOISTURE CONTENT %	DRY DENSITY PCF	LIQUID LIMITS %	PLASTIC LIMITS %	PLASTICITY INDEX %	Unconfined Comp. TSF	Unconfined Comp. Strain %	PASSING 200 SIEVE %	DIRECT SHEAR COHESION PSF	DIRECT SHEAR ANGLE Deg	EXPANSION INDEX	
	3													FILL-Brown silty SAND (SM), 0.1 ppm
	5	N=12												NATIVE-Gray-brown silty SAND (SM), medium dense, 0.1 ppm
C	7	N=23												Gray SAND (SP), medium dense, wet, 0.1 ppm
	8													Saturated below 7'
	10	N=19												Gray-brown, 0.1 ppm
	13													
C	15	N=15												Gray, w/some interbedded gray silt, saturated, 0.0 ppm
	18													Brown silty SAND (SM), medium dense, saturated, 0.1 ppm
	20	N=14												
	23													Dark gray sandy SILT (ML), stiff, saturated, 0.1 ppm
C	25	N=10												
	28													
	30	N=15												Brown silty SAND (SM), medium dense, saturated, 0.0 ppm
	33													
C	35	N=23												Gray, 0.1 ppm
	38													Gray SAND (SP), dense,saturated, coarse grained, 0.0 ppm
	40	N=37												
	43													
C	45	N=39												Dark gray, 0.0 ppm
	48													
	50	N=40												0.0 ppm

TUBE SAMPLE
AUGER SAMPLE
C CALIFORNIA MODIFIED SAMPLER
SPLIT SPOON
NO RECOVERY
Ground Water Level
Hydrostatic Ground Water Level
SM
SP
ML

N= SPT, BLOWS/FT
T= THD BLOWS/FT
P= HAND PEN.,TSF
0.1 ppm=PID READING

REMARKS:
NP: Non Plastic Materials
* Remolded Samples

LOG OF BORING NO. B - 9 (continued)

AESCO

Project: Belmont Plaza Pool Rebuild-Revitalization
Location: 4000 East Olympic Plaza, Long Beach, CA

WATER: Encountered at 6 Feet

Client: Harley Ellis Devereaux
Date: 04/03/14
Logger:
Project No. 20140185-C8050

DRILLING: Hollow Stem Auger with Drilling Mud

FIELD DATA			TESTS	LABORATORY DATA											DESCRIPTION OF STRATUM
SOIL SYMBOL		DEPTH (FT)	N= T= P=	MOISTURE CONTENT %	DRY DENSITY PCF	LIQUID LIMITS %	PLASTIC LIMITS %	PLASTICITY INDEX %	Unconfined Comp. TSF	Unconfined Comp. Strain %	PASSING 200 SIEVE %	DIRECT SHEAR COHESION PSF	DIRECT SHEAR ANGLE Deg	EXPANSION INDEX	
		53													Gray SAND (SP), dense, saturated, coarse grained
	C	55	N=32												
		58													
		60	N=28												W/interbedded gray silt at 58', 0.1 ppm
		63													
	C	65	N=50/4"												Very dense at 63', 0.1 ppm
		68													
		70	N=56												
		73													
	C	75	N=20												Medium dense at 73'
		78													
		80	N=58												Very dense at 78'

Boring Terminated at 80 Feet

TUBE SAMPLE
AUGER SAMPLE
C CALIFORNIA MODIFIED SAMPLER
SPLIT SPOON
NO RECOVERY
SP
Ground Water Level
Hydrostatic Ground Water Level

N= SPT, BLOWS/FT
T= THD,BLOWS/FT
P= HAND PEN.,TSF
0.1 ppm=PID READING

REMARKS:
NP: Non Plastic Materials
* Remolded Samples

LOG OF BORING NO. B - 10

AESCO

Project: Belmont Plaza Pool Rebuild-Revitalization
Location: 4000 East Olympic Plaza, Long Beach, CA
Client: Harley Ellis Devereaux
Date: 04/03/14
Logger:
Project No. 20140185-C8050

WATER: Encountered at 4 Feet

DRILLING: Hollow Stem Auger with Drilling Mud

FIELD DATA			TESTS	LABORATORY DATA											DESCRIPTION OF STRATUM
SOIL SYMBOL		DEPTH (FT)	N= T= P=	MOISTURE CONTENT %	DRY DENSITY PCF	LIQUID LIMITS %	PLASTIC LIMITS %	PLASTICITY INDEX %	Unconfined Comp. TSF	Unconfined Comp. Strain %	PASSING 200 SIEVE %	DIRECT SHEAR COHESION PSF	DIRECT SHEAR ANGLE Deg	EXPANSION INDEX	
		3		10.7											FILL-Brown silty SAND (SM), moist, 0.0 ppm
		5	N=25	9.9											Gray-brown SAND/silty SAND (SP/SM), medium dense, saturated, 0.1 ppm
	C	7	N=19	8.9	97.9						2.2				Gray-brown SAND (SP), medium dense, saturated, 0.1 ppm
		8													
		10	N=23	21.2											0.1 ppm
		13													
	C	15	N=13 P=1.0	17.9	113.9						0.6				Gray, coarse grained at 13', 0.1 ppm
		18													
		20	N=18	18.9											0.1 ppm
		23													Gray-brown sandy SILT (ML), stiff, saturated, 0.1 ppm
	C	25	N=10 P=0.5	27.4							61.3				
		28													
		30	N=15	18.7							22.8				Gray silty SAND (SM), medium dense, saturated, 0.1 ppm
		33													
	C	35	N=18	26.7							15.2				0.1 ppm

Boring Terminated at 35 Feet

TUBE SAMPLE
AUGER SAMPLE
C CALIFORNIA MODIFIED SAMPLER
SPLIT SPOON
NO RECOVERY
Ground Water Level
Hydrostatic Ground Water Level
SM
SP/SM
SP
ML
N= SPT, BLOWS/FT
T= THD,BLOWS/FT
P= HAND PEN.,TSF
0.1 ppm=PID READING

REMARKS:
NP: Non Plastic Materials
* Remolded Samples

APPENDIX
SEISMIC DESIGN DATA

USGS Design Maps Summary Report

User-Specified Input

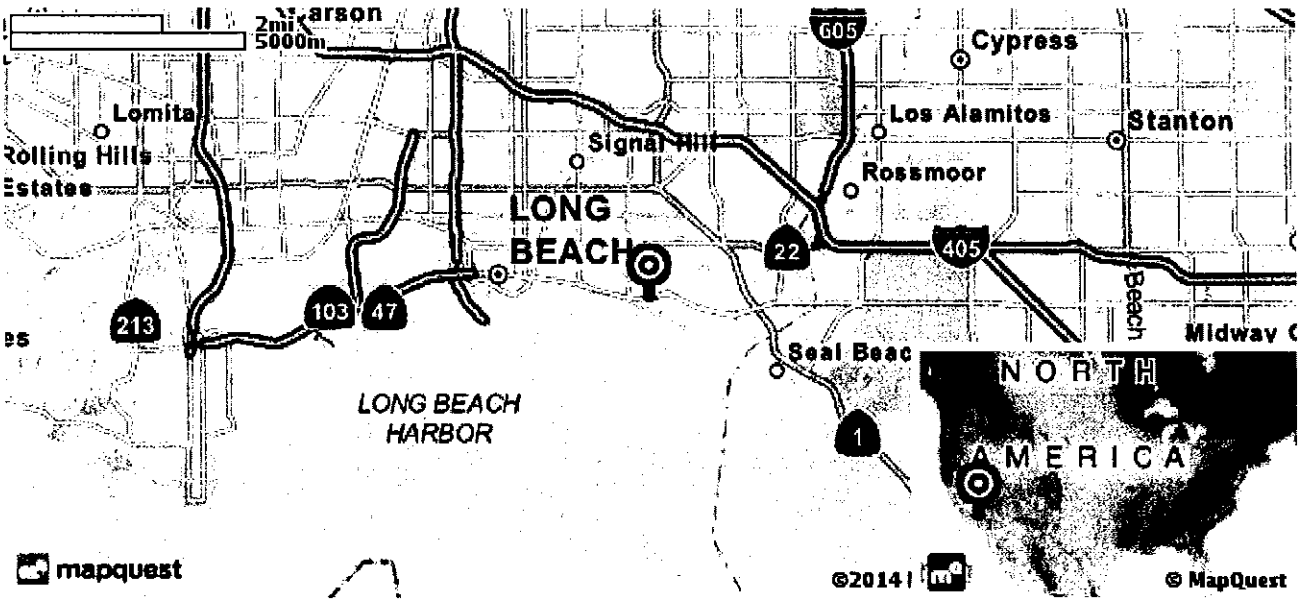
Report Title Belmont Plaza Pool Rebuild-Revitalization
Thu April 24, 2014 22:41:14 UTC

Building Code Reference Document 2012 International Building Code
(which utilizes USGS hazard data available in 2008)

Site Coordinates 33.7593°N, 118.1449°W

Site Soil Classification Site Class D – "Stiff Soil"

Risk Category I/II/III

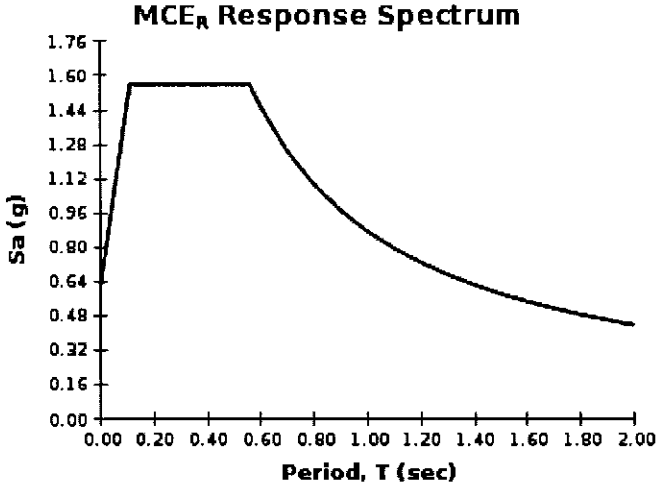


USGS-Provided Output

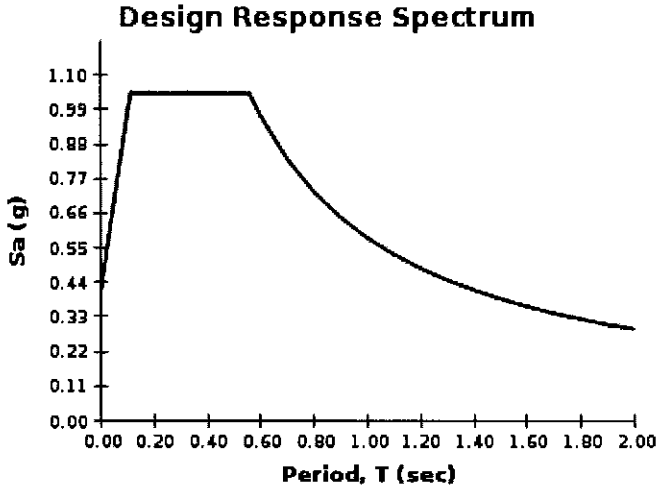
S_S =	1.561 g	S_{MS} =	1.561 g	S_{DS} =	1.041 g
S_1 =	0.582 g	S_{M1} =	0.873 g	S_{D1} =	0.582 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.

MCE_R Response Spectrum



Design Response Spectrum



Although this information is a product of the U.S. Geological Survey, we provide no warranty, expressed or implied, as to the accuracy of the data contained therein. This tool is not a substitute for technical subject-matter

knowledge.

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*    E Q F A U L T    *
*                     *
*    Version 3.00     *
*                     *
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DETERMINISTIC ESTIMATION OF
PEAK ACCELERATION FROM DIGITIZED FAULTS

JOB NUMBER: 20140185-C8050

DATE: 04-24-2014

JOB NAME: Belmont Plaza Pool Rebuild

CALCULATION NAME: Test Run Analysis

FAULT-DATA-FILE NAME: CDMGFLTE.DAT

SITE COORDINATES:
SITE LATITUDE: 33.7593
SITE LONGITUDE: 118.1449

SEARCH RADIUS: 62.1 mi

ATTENUATION RELATION: 14) Campbell & Bozorgnia (1997 Rev.) - Alluvium
UNCERTAINTY (M=Median, S=Sigma): M Number of Sigmas: 0.0
DISTANCE MEASURE: cdist
SCOND: 0
Basement Depth: 5.00 km Campbell SSR: 0 Campbell SHR: 0
COMPUTE PEAK HORIZONTAL ACCELERATION

FAULT-DATA FILE USED: CDMGFLTE.DAT

MINIMUM DEPTH VALUE (km): 3.0

EQFAULT SUMMARY

DETERMINISTIC SITE PARAMETERS

Page 1

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT MAXIMUM EARTHQUAKE MAG.(Mw)	PEAK SITE ACCEL. g	EST. SITE INTENSITY MOD.MERC.
NEWPORT-INGLEWOOD (L.A.Basin)	2.4(3.8)	6.9	0.482	X
COMPTON THRUST	3.4(5.4)	6.8	0.598	X
PALOS VERDES	6.6(10.7)	7.1	0.388	X
ELYSIAN PARK THRUST	13.7(22.1)	6.7	0.205	VIII
WHITTIER	17.4(28.0)	6.8	0.149	VIII
NEWPORT-INGLEWOOD (Offshore)	17.7(28.5)	6.9	0.158	VIII
SAN JOSE	24.9(40.1)	6.5	0.083	VII
RAYMOND	25.9(41.7)	6.5	0.079	VII
HOLLYWOOD	26.1(42.0)	6.4	0.072	VII
CHINO-CENTRAL AVE. (Elsinore)	27.0(43.4)	6.7	0.087	VII
VERDUGO	27.3(44.0)	6.7	0.085	VII
SANTA MONICA	27.5(44.2)	6.6	0.078	VII
MALIBU COAST	29.5(47.4)	6.7	0.077	VII
ELSINORE-GLEN IVY	30.0(48.2)	6.8	0.078	VII
SIERRA MADRE	31.1(50.0)	7.0	0.090	VII
CLAMSHELL-SAWPIT	31.4(50.5)	6.5	0.060	VI
NORTHRIDGE (E. Oak Ridge)	35.5(57.2)	6.9	0.069	VI
CUCAMONGA	35.6(57.3)	7.0	0.075	VII
CORONADO BANK	36.2(58.2)	7.4	0.102	VII
ANACAPA-DUME	36.7(59.0)	7.3	0.090	VII
SIERRA MADRE (San Fernando)	38.7(62.3)	6.7	0.053	VI
SAN GABRIEL	39.4(63.4)	7.0	0.066	VI
SANTA SUSANA	45.1(72.6)	6.6	0.039	V
ELSINORE-TEMECULA	46.5(74.9)	6.8	0.044	VI
HOLSER	49.3(79.4)	6.5	0.032	V
SAN JACINTO-SAN BERNARDINO	49.7(80.0)	6.7	0.037	V
SAN ANDREAS - Mojave	50.0(80.4)	7.1	0.053	VI
SAN ANDREAS - 1857 Rupture	50.0(80.4)	7.8	0.096	VII
SAN ANDREAS - San Bernardino	51.9(83.5)	7.3	0.060	VI
SAN ANDREAS - Southern	51.9(83.5)	7.4	0.065	VI
SIMI-SANTA ROSA	53.1(85.5)	6.7	0.034	V
OAK RIDGE (Onshore)	53.2(85.6)	6.9	0.039	V
CLEGHORN	54.6(87.9)	6.5	0.028	V
SAN JACINTO-SAN JACINTO VALLEY	55.1(88.7)	6.9	0.039	V
SAN CAYETANO	59.8(96.2)	6.8	0.031	V
ROSE CANYON	60.0(96.6)	6.9	0.035	V

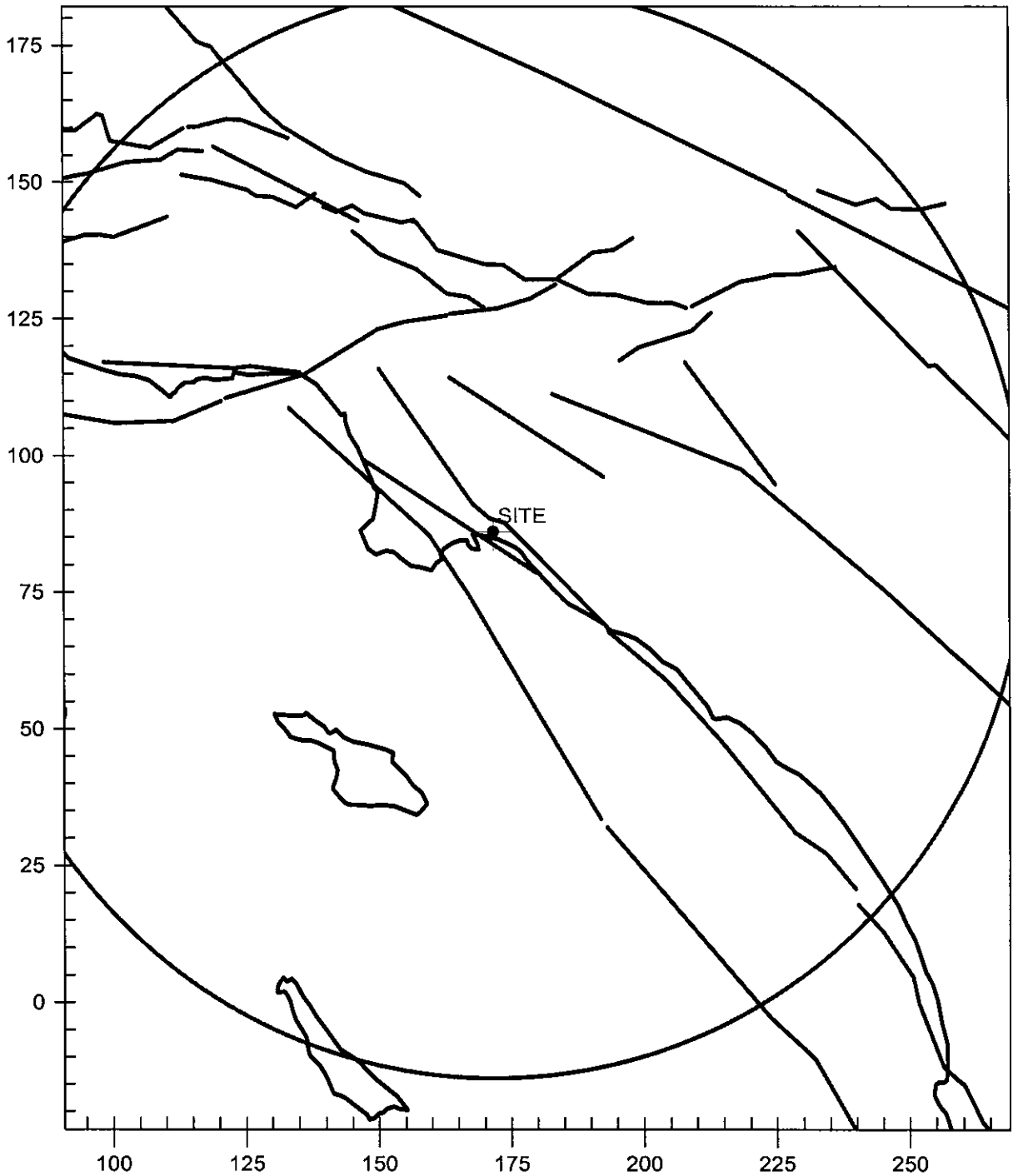
-END OF SEARCH- 36 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE NEWPORT-INGLEWOOD (L.A.Basin) FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 2.4 MILES (3.8 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.5982 g

CALIFORNIA FAULT MAP

Belmont Plaza Pool Rebuild



APPENDIX
LABORATORY TEST RESULTS

Associated Laboratories
806 N. Batavia - Orange, CA 92868
Tel (714)771-6900 Fax (714)538-1209
www.associatedlabs.com
info@associatedlabs.com

ACCREDITED IN ACCORDANCE WITH nelac
04232CA

Client: AESCO Technologies
Address: 17782 Georgetown Lane
Huntington Beach, CA 92647

Attn: Debra Perez

Comments: Harley Ellis
#20140185-C8480
4000 Olympic Plaza, Long Beach, CA

Lab Request: 339200
Report Date: 04/23/2014
Date Received: 04/09/2014
Client ID: 9650

This laboratory request covers the following listed samples which were analyzed for the parameters indicated on the attached Analytical Result Report. All analyses were conducted using the appropriate methods. Methods accredited by NELAC are indicated on the report. This cover letter is an integral part of the final report.

Sample #	Client Sample ID
339200-001	B-6 Water
339200-002	B-3 Water
339200-003	B-5 Water
339200-004	B-9 Water
339200-005	B-6 5-7' Sample A,B
339200-006	B-5 13-15' Sample A,B
339200-007	B-9 5-7' Sample A,B
339200-008	B-3 5-7' Sample A,B
339200-009	B-5 5-7' Sample A,B

Thank you for the opportunity to be of service to your company. Please feel free to call if there are any questions regarding this report or if we can be of further service.

ASSOCIATED LABORATORIES by,

Nina Prasad
President

NOTE: Unless notified in writing , all samples will be discarded by appropriate disposal protocol 45 days from date reported.

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TESTING & CONSULTING
Chemical
Microbiological
Environmental

Matrix: Water	Client: AESCO Technologies	Collector: Client
Sampled: 04/09/2014 15:15	Site:	
Sample #: 339200-001	Client Sample #: B-6 Water	Sample Type:

Analyte	Result	DF	RDL	Units	Analyzed	By	Notes
Method: EPA 8015 *NELAC*	Prep Method: EPA 3535A					QCBatchID:	QC1145584
TPH Gasoline	ND	1	0.2	mg/L	04/11/14	lyt	
TPH Diesel	ND	1	0.1	mg/L	04/11/14	lyt	
TPH Motor Oil	ND	1	0.3	mg/L	04/11/14	lyt	

Analyte	% Recovery	Limits	Notes
Triacontane (SUR)	*80*	*60-140*	

Matrix: Water	**Client:** AESCO Technologies	**Collector:** Client
Sampled: 04/09/2014 15:15	**Site:**	
Sample #: 339200-002	**Client Sample #:** B-3 Water	**Sample Type:**

Analyte	Result	DF	RDL	Units	Analyzed	By	Notes
Method: EPA 8015 *NELAC*	**Prep Method:** EPA 3535A					QCBatchID:	QC1145584
TPH Gasoline	ND	1	0.2	mg/L	04/14/14	lyt	
TPH Diesel	ND	1	0.1	mg/L	04/14/14	lyt	
TPH Motor Oil	ND	1	0.3	mg/L	04/14/14	lyt	

Analyte	% Recovery	Limits	Notes
Triacontane (SUR)	*135*	*60-140*	

Matrix: Water	**Client:** AESCO Technologies	**Collector:** Client
Sampled: 04/09/2014 15:15	**Site:**	
Sample #: 339200-003	**Client Sample #:** B-5 Water	**Sample Type:**

Analyte	Result	DF	RDL	Units	Analyzed	By	Notes
Method: EPA 8015 *NELAC*	Prep Method: EPA 3535A					QCBatchID:	QC1145584
TPH Gasoline	0.60	1	0.2	mg/L	04/11/14	lyt	
TPH Diesel	0.94	1	0.1	mg/L	04/11/14	lyt	
TPH Motor Oil	0.37	1	0.3	mg/L	04/11/14	lyt	

Analyte	% Recovery	Limits	Notes
Triacontane (SUR)	*140*	*60-140*	

Matrix: Water	**Client:** AESCO Technologies	**Collector:** Client
Sampled: 04/09/2014 15:15	**Site:**	
Sample #: 339200-004	**Client Sample #:** B-9 Water	**Sample Type:**

Analyte	Result	DF	RDL	Units	Analyzed	By	Notes
Method: EPA 8015 *NELAC*	**Prep Method:** EPA 3535A					QCBatchID:	QC1145584
TPH Gasoline	ND	1	0.2	mg/L	04/11/14	lyt	
TPH Diesel	ND	1	0.1	mg/L	04/11/14	lyt	
TPH Motor Oil	ND	1	0.3	mg/L	04/11/14	lyt	

Analyte	% Recovery	Limits	Notes
Triacontane (SUR)	*135*	*60-140*	

Matrix: Solid	Client: AESCO Technologies	Collector: Client
Sampled: 04/09/2014 15:15	Site:	
Sample #: 339200-005	Client Sample #: B-6 5-7' Sample A,B	Sample Type:

Analyte	Result	DF	RDL	Units	Analyzed	By	Notes
Method: EPA 418.1 *NELAC*	**Prep Method: Method**					**QCBatchID: QC1145660**	
Total Recoverable Petroleum Hydrocarbons	20	1	10	mg/Kg	04/15/14	thun	
Method: EPA 6010 *NELAC*	**Prep Method: EPA 3050B**					**QCBatchID: QC1145553**	
Antimony	ND	1	3	mg/Kg	04/11/14	wyu	
Arsenic	2.75	1	1	mg/Kg	04/11/14	wyu	
Barium	13.0	1	1	mg/Kg	04/11/14	wyu	
Beryllium	ND	1	0.5	mg/Kg	04/11/14	wyu	
Cadmium	ND	1	0.5	mg/Kg	04/11/14	wyu	
Chromium	5.20	1	1	mg/Kg	04/11/14	wyu	
Cobalt	2.24	1	0.5	mg/Kg	04/11/14	wyu	
Copper	5.68	1	1	mg/Kg	04/11/14	wyu	
Lead	1.49	1	0.5	mg/Kg	04/11/14	wyu	
Molybdenum	ND	1	1	mg/Kg	04/11/14	wyu	
Nickel	2.59	1	1.5	mg/Kg	04/11/14	wyu	
Selenium	ND	1	1	mg/Kg	04/11/14	wyu	
Silver	ND	1	0.5	mg/Kg	04/11/14	wyu	L
Thallium	ND	1	1	mg/Kg	04/11/14	wyu	
Vanadium	10.3	1	0.5	mg/Kg	04/11/14	wyu	
Zinc	10.2	1	5	mg/Kg	04/11/14	wyu	
Method: EPA 7471 *NELAC*	**Prep Method: EPA 7471A**					**QCBatchID: QC1145564**	
Mercury	ND	1	0.14	mg/Kg	04/11/14	wei	
Method: EPA 8015 *NELAC*	**Prep Method: EPA 3545**					**QCBatchID: QC1145550**	
TPH Gasoline	ND	1	3	mg/Kg	04/11/14	lyt	
TPH Diesel	2.05	1	1	mg/Kg	04/11/14	lyt	
TPH Motor Oil	6.15	1	5	mg/Kg	04/11/14	lyt	

Analyte	% Recovery	Limits	Notes
Triacontane (SUR)	*138*	*60-140*	

Method: EPA 8260 *NELAC*	**Prep Method: EPA 5035**					**QCBatchID: QC1145577**	
1,1,1,2-Tetrachloroethane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,1,1-Trichloroethane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,1,2,2-Tetrachloroethane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,1,2-Trichloroethane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,1,2-Trichlorotrifluoroethane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,1-Dichloroethane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,1-Dichloroethene	ND	1	5	ug/Kg	04/10/14	nicollez	
1,1-Dichloropropene	ND	1	5	ug/Kg	04/10/14	nicollez	
1,2,3-Trichlorobenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
1,2,3-Trichloropropane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,2,4-Trichlorobenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
1,2,4-Trimethylbenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
1,2-Dibromo-3-chloropropane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,2-Dibromoethane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,2-Dichlorobenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
1,2-Dichloroethane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,2-Dichloropropane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,3,5-Trimethylbenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
1,3-Dichlorobenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
1,3-Dichloropropane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,4-Dichlorobenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
2,2-Dichloropropane	ND	1	5	ug/Kg	04/10/14	nicollez	
2-Butanone (MEK)	ND	1	100	ug/Kg	04/10/14	nicollez	
2-Chloroethyl Vinyl Ether	ND	1	5	ug/Kg	04/10/14	nicollez	

Matrix: Solid | Client: AESCO Technologies | Collector: Client
Sampled: 04/09/2014 15:15 | Site:
Sample #: 339200-005 | Client Sample #: B-6 5-7' Sample A,B | Sample Type:

Analyte	Result	DF	RDL	Units	Analyzed	By	Notes
2-Chlorotoluene	ND	1	5	ug/Kg	04/10/14	nicollez	
4-Chlorotoluene	ND	1	5	ug/Kg	04/10/14	nicollez	
4-Isopropyltoluene	ND	1	5	ug/Kg	04/10/14	nicollez	
4-Methyl-2-pentanone (MIBK)	ND	1	5	ug/Kg	04/10/14	nicollez	
Acetone	ND	1	100	ug/Kg	04/10/14	nicollez	
Allyl Chloride	ND	1	5	ug/Kg	04/10/14	nicollez	
Benzene	ND	1	5	ug/Kg	04/10/14	nicollez	
Bromobenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
Bromochloromethane	ND	1	5	ug/Kg	04/10/14	nicollez	
Bromodichloromethane	ND	1	5	ug/Kg	04/10/14	nicollez	
Bromoform	ND	1	5	ug/Kg	04/10/14	nicollez	
Bromomethane	ND	1	5	ug/Kg	04/10/14	nicollez	
Carbon Tetrachloride	ND	1	5	ug/Kg	04/10/14	nicollez	
Chlorobenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
Chlorodibromomethane	ND	1	5	ug/Kg	04/10/14	nicollez	
Chloroethane	ND	1	5	ug/Kg	04/10/14	nicollez	
Chloroform	ND	1	5	ug/Kg	04/10/14	nicollez	
Chloromethane	ND	1	5	ug/Kg	04/10/14	nicollez	
cis-1,2-Dichloroethene	ND	1	5	ug/Kg	04/10/14	nicollez	
cis-1,3-dichloropropene	ND	1	5	ug/Kg	04/10/14	nicollez	
cis-1,4-dichloro-2-butene	ND	1	5	ug/Kg	04/10/14	nicollez	
Dibromomethane	ND	1	5	ug/Kg	04/10/14	nicollez	
Dichlorodifluoromethane	ND	1	5	ug/Kg	04/10/14	nicollez	
Di-isopropyl ether (DIPE)	ND	1	5	ug/Kg	04/10/14	nicollez	
Ethylbenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
Ethyl-tertbutylether (ETBE)	ND	1	5	ug/Kg	04/10/14	nicollez	
Hexachlorobutadiene	ND	1	5	ug/Kg	04/10/14	nicollez	
Isopropylbenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
m and p-Xylene	ND	1	5	ug/Kg	04/10/14	nicollez	
Methylene chloride	ND	1	5	ug/Kg	04/10/14	nicollez	
Methyl-t-butyl Ether (MTBE)	ND	1	5	ug/Kg	04/10/14	nicollez	
Naphthalene	ND	1	5	ug/Kg	04/10/14	nicollez	
N-butylbenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
N-propylbenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
o-Xylene	ND	1	5	ug/Kg	04/10/14	nicollez	
Sec-butylbenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
Styrene	ND	1	5	ug/Kg	04/10/14	nicollez	
t-Butyl alcohol (TBA)	ND	1	10	ug/Kg	04/10/14	nicollez	
Tert-amylmethylether (TAME)	ND	1	5	ug/Kg	04/10/14	nicollez	
Tert-butylbenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
Tetrachloroethene	ND	1	5	ug/Kg	04/10/14	nicollez	
Toluene	ND	1	5	ug/Kg	04/10/14	nicollez	
trans-1,2-dichloroethene	ND	1	5	ug/Kg	04/10/14	nicollez	
trans-1,3-dichloropropene	ND	1	5	ug/Kg	04/10/14	nicollez	
trans-1,4-dichloro-2-butene	ND	1	5	ug/Kg	04/10/14	nicollez	
Trichloroethene	ND	1	5	ug/Kg	04/10/14	nicollez	
Trichlorofluoromethane	ND	1	5	ug/Kg	04/10/14	nicollez	
Vinyl Chloride	ND	1	5	ug/Kg	04/10/14	nicollez	
Xylenes (Total)	ND	1	5	ug/Kg	04/10/14	nicollez	

Matrix: Solid	**Client:** AESCO Technologies	**Collector:** Client
Sampled: 04/09/2014 15:15	**Site:**	
Sample #: 339200-005	**Client Sample #:** B-6 5-7' Sample A,B	**Sample Type:**

Analyte	Result	DF	RDL	Units	Analyzed	By	Notes

Analyte	% Recovery	Limits	Notes
1,2-Dichloroethane-d4 (SUR)	*103*	*70-145*	
4-Bromofluorobenzene (SUR)	*104*	*70-145*	
Dibromodifluoromethane (SUR)	*103*	*70-145*	
Toluene-d8 (SUR)	*104*	*70-145*	

Method: EPA 9045 *NELAC* **Prep Method:** Method **QCBatchID:**

pH	**9.02**	1		04/16/14	mmegaly
Temperature (°C)	**24.8**	1	°C	04/16/14	mmegaly

Matrix: Solid	**Client:** AESCO Technologies	**Collector:** Client
Sampled: 04/09/2014 15:15	**Site:**	
Sample #: 339200-006	**Client Sample #:** B-5 13-15' Sample A,B	**Sample Type:**

Analyte	Result	DF	RDL	Units	Analyzed	By	Notes
Method: EPA 418.1 *NELAC*	**Prep Method:** Method					QCBatchID:	QC1145660
Total Recoverable Petroleum Hydrocarbons	28	1	10	mg/Kg	04/15/14	thun	
Method: EPA 6010 *NELAC*	**Prep Method:** EPA 3050B					QCBatchID:	QC1145553
Antimony	ND	1	3	mg/Kg	04/11/14	wyu	
Arsenic	2.43	1	1	mg/Kg	04/11/14	wyu	
Barium	47.1	1	1	mg/Kg	04/11/14	wyu	
Beryllium	ND	1	0.5	mg/Kg	04/11/14	wyu	
Cadmium	ND	1	0.5	mg/Kg	04/11/14	wyu	
Chromium	9.60	1	1	mg/Kg	04/11/14	wyu	
Cobalt	5.31	1	0.5	mg/Kg	04/11/14	wyu	
Copper	11.3	1	1	mg/Kg	04/11/14	wyu	
Lead	2.48	1	0.5	mg/Kg	04/11/14	wyu	
Molybdenum	ND	1	1	mg/Kg	04/11/14	wyu	
Nickel	6.23	1	1.5	mg/Kg	04/11/14	wyu	
Selenium	4.41	1	1	mg/Kg	04/11/14	wyu	
Silver	ND	1	0.5	mg/Kg	04/11/14	wyu	L
Thallium	ND	1	1	mg/Kg	04/11/14	wyu	
Vanadium	21.2	1	0.5	mg/Kg	04/11/14	wyu	
Zinc	28.9	1	5	mg/Kg	04/11/14	wyu	
Method: EPA 7471 *NELAC*	**Prep Method:** EPA 7471A					QCBatchID:	QC1145564
Mercury	ND	1	0.14	mg/Kg	04/11/14	wei	
Method: EPA 8015 *NELAC*	**Prep Method:** EPA 3545					QCBatchID:	QC1145550
TPH Gasoline	ND	1	3	mg/Kg	04/11/14	lyt	
TPH Diesel	4.48	1	1	mg/Kg	04/11/14	lyt	
TPH Motor Oil	6.29	1	5	mg/Kg	04/11/14	lyt	

Analyte	% Recovery	Limits	Notes
Triacontane (SUR)	*135*	*60-140*	

Analyte	Result	DF	RDL	Units	Analyzed	By	Notes
Method: EPA 8260 *NELAC*	**Prep Method:** EPA 5035					QCBatchID:	QC1145577
1,1,1,2-Tetrachloroethane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,1,1-Trichloroethane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,1,2,2-Tetrachloroethane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,1,2-Trichloroethane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,1,2-Trichlorotrifluoroethane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,1-Dichloroethane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,1-Dichloroethene	ND	1	5	ug/Kg	04/10/14	nicollez	
1,1-Dichloropropene	ND	1	5	ug/Kg	04/10/14	nicollez	
1,2,3-Trichlorobenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
1,2,3-Trichloropropane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,2,4-Trichlorobenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
1,2,4-Trimethylbenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
1,2-Dibromo-3-chloropropane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,2-Dibromoethane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,2-Dichlorobenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
1,2-Dichloroethane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,2-Dichloropropane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,3,5-Trimethylbenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
1,3-Dichlorobenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
1,3-Dichloropropane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,4-Dichlorobenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
2,2-Dichloropropane	ND	1	5	ug/Kg	04/10/14	nicollez	
2-Butanone (MEK)	ND	1	100	ug/Kg	04/10/14	nicollez	
2-Chloroethyl Vinyl Ether	ND	1	5	ug/Kg	04/10/14	nicollez	

Matrix: Solid	**Client:** AESCO Technologies	**Collector:** Client
Sampled: 04/09/2014 15:15	**Site:**	
Sample #: 339200-006	**Client Sample #:** B-5 13-15' Sample A;B	**Sample Type:**

Analyte	Result	DF	RDL	Units	Analyzed	By	Notes
2-Chlorotoluene	ND	1	5	ug/Kg	04/10/14	nicollez	
4-Chlorotoluene	ND	1	5	ug/Kg	04/10/14	nicollez	
4-Isopropyltoluene	ND	1	5	ug/Kg	04/10/14	nicollez	
4-Methyl-2-pentanone (MIBK)	ND	1	5	ug/Kg	04/10/14	nicollez	
Acetone	ND	1	100	ug/Kg	04/10/14	nicollez	
Allyl Chloride	ND	1	5	ug/Kg	04/10/14	nicollez	
Benzene	ND	1	5	ug/Kg	04/10/14	nicollez	
Bromobenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
Bromochloromethane	ND	1	5	ug/Kg	04/10/14	nicollez	
Bromodichloromethane	ND	1	5	ug/Kg	04/10/14	nicollez	
Bromoform	ND	1	5	ug/Kg	04/10/14	nicollez	
Bromomethane	ND	1	5	ug/Kg	04/10/14	nicollez	
Carbon Tetrachloride	ND	1	5	ug/Kg	04/10/14	nicollez	
Chlorobenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
Chlorodibromomethane	ND	1	5	ug/Kg	04/10/14	nicollez	
Chloroethane	ND	1	5	ug/Kg	04/10/14	nicollez	
Chloroform	ND	1	5	ug/Kg	04/10/14	nicollez	
Chloromethane	ND	1	5	ug/Kg	04/10/14	nicollez	
cis-1,2-Dichloroethene	ND	1	5	ug/Kg	04/10/14	nicollez	
cis-1,3-dichloropropene	ND	1	5	ug/Kg	04/10/14	nicollez	
cis-1,4-dichloro-2-butene	ND	1	5	ug/Kg	04/10/14	nicollez	
Dibromomethane	ND	1	5	ug/Kg	04/10/14	nicollez	
Dichlorodifluoromethane	ND	1	5	ug/Kg	04/10/14	nicollez	
Di-isopropyl ether (DIPE)	ND	1	5	ug/Kg	04/10/14	nicollez	
Ethylbenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
Ethyl-tertbutylether (ETBE)	ND	1	5	ug/Kg	04/10/14	nicollez	
Hexachlorobutadiene	ND	1	5	ug/Kg	04/10/14	nicollez	
Isopropylbenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
m and p-Xylene	ND	1	5	ug/Kg	04/10/14	nicollez	
Methylene chloride	ND	1	5	ug/Kg	04/10/14	nicollez	
Methyl-t-butyl Ether (MTBE)	ND	1	5	ug/Kg	04/10/14	nicollez	
Naphthalene	ND	1	5	ug/Kg	04/10/14	nicollez	
N-butylbenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
N-propylbenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
o-Xylene	ND	1	5	ug/Kg	04/10/14	nicollez	
Sec-butylbenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
Styrene	ND	1	5	ug/Kg	04/10/14	nicollez	
t-Butyl alcohol (TBA)	ND	1	10	ug/Kg	04/10/14	nicollez	
Tert-amylmethylether (TAME)	ND	1	5	ug/Kg	04/10/14	nicollez	
Tert-butylbenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
Tetrachloroethene	ND	1	5	ug/Kg	04/10/14	nicollez	
Toluene	ND	1	5	ug/Kg	04/10/14	nicollez	
trans-1,2-dichloroethene	ND	1	5	ug/Kg	04/10/14	nicollez	
trans-1,3-dichloropropene	ND	1	5	ug/Kg	04/10/14	nicollez	
trans-1,4-dichloro-2-butene	ND	1	5	ug/Kg	04/10/14	nicollez	
Trichloroethene	ND	1	5	ug/Kg	04/10/14	nicollez	
Trichlorofluoromethane	ND	1	5	ug/Kg	04/10/14	nicollez	
Vinyl Chloride	ND	1	5	ug/Kg	04/10/14	nicollez	
Xylenes (Total)	ND	1	5	ug/Kg	04/10/14	nicollez	

Matrix: Solid		**Client:** AESCO Technologies		**Collector:** Client	
Sampled: 04/09/2014 15:15		**Site:**			
Sample #: 339200-006		**Client Sample #:** B-5 13-15' Sample A,B		**Sample Type:**	

Analyte	Result	DF	RDL	Units	Analyzed	By	Notes

Analyte	% Recovery	Limits	Notes
1,2-Dichloroethane-d4 (SUR)	*108*	*70-145*	
4-Bromofluorobenzene (SUR)	*128*	*70-145*	
Dibromodifluoromethane (SUR)	*104*	*70-145*	
Toluene-d8 (SUR)	*104*	*70-145*	

Method: EPA 9045 *NELAC* **Prep Method:** Method **QCBatchID:**

pH	**9.11**	1		04/16/14	mmegaly	
Temperature (°C)	**25.0**	1	°C	04/16/14	mmegaly	

Matrix: Solid	Client: AESCO Technologies	Collector: Client
Sampled: 04/09/2014 15:15	Site:	
Sample #: 339200-007	Client Sample #: B-9 5-7' Sample A,B	Sample Type:

Analyte	Result	DF	RDL	Units	Analyzed	By	Notes
Method: EPA 418.1 NELAC	**Prep Method: Method**					**QCBatchID: QC1145660**	
Total Recoverable Petroleum Hydrocarbons	**16**	1	10	mg/Kg	04/15/14	thun	
Method: EPA 6010 NELAC	**Prep Method: EPA 3050B**					**QCBatchID: QC1145553**	
Antimony	ND	1	3	mg/Kg	04/11/14	wyu	
Arsenic	**3.27**	1	1	mg/Kg	04/11/14	wyu	
Barium	**21.0**	1	1	mg/Kg	04/11/14	wyu	
Beryllium	ND	1	0.5	mg/Kg	04/11/14	wyu	
Cadmium	ND	1	0.5	mg/Kg	04/11/14	wyu	
Chromium	**8.17**	1	1	mg/Kg	04/11/14	wyu	
Cobalt	**3.50**	1	0.5	mg/Kg	04/11/14	wyu	
Copper	**25.8**	1	1	mg/Kg	04/11/14	wyu	
Lead	**1.72**	1	0.5	mg/Kg	04/11/14	wyu	
Molybdenum	ND	1	1	mg/Kg	04/11/14	wyu	
Nickel	**4.25**	1	1.5	mg/Kg	04/11/14	wyu	
Selenium	**1.46**	1	1	mg/Kg	04/11/14	wyu	
Silver	ND	1	0.5	mg/Kg	04/11/14	wyu	L
Thallium	ND	1	1	mg/Kg	04/11/14	wyu	
Vanadium	**18.8**	1	0.5	mg/Kg	04/11/14	wyu	
Zinc	**19.8**	1	5	mg/Kg	04/11/14	wyu	
Method: EPA 7471 NELAC	**Prep Method: EPA 7471A**					**QCBatchID: QC1145564**	
Mercury	ND	1	0.14	mg/Kg	04/11/14	wei	
Method: EPA 8015 NELAC	**Prep Method: EPA 3545**					**QCBatchID: QC1145550**	
TPH Gasoline	ND	1	3	mg/Kg	04/11/14	lyt	
TPH Diesel	ND	1	1	mg/Kg	04/11/14	lyt	
TPH Motor Oil	ND	1	5	mg/Kg	04/11/14	lyt	

Analyte	% Recovery	Limits	Notes
Triacontane (SUR)	*90*	*60-140*	

Method: EPA 8260 NELAC	**Prep Method: EPA 5035**					**QCBatchID: QC1145577**	
1,1,1,2-Tetrachloroethane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,1,1-Trichloroethane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,1,2,2-Tetrachloroethane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,1,2-Trichloroethane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,1,2-Trichlorotrifluoroethane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,1-Dichloroethane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,1-Dichloroethene	ND	1	5	ug/Kg	04/10/14	nicollez	
1,1-Dichloropropene	ND	1	5	ug/Kg	04/10/14	nicollez	
1,2,3-Trichlorobenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
1,2,3-Trichloropropane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,2,4-Trichlorobenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
1,2,4-Trimethylbenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
1,2-Dibromo-3-chloropropane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,2-Dibromoethane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,2-Dichlorobenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
1,2-Dichloroethane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,2-Dichloropropane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,3,5-Trimethylbenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
1,3-Dichlorobenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
1,3-Dichloropropane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,4-Dichlorobenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
2,2-Dichloropropane	ND	1	5	ug/Kg	04/10/14	nicollez	
2-Butanone (MEK)	ND	1	100	ug/Kg	04/10/14	nicollez	
2-Chloroethyl Vinyl Ether	ND	1	5	ug/Kg	04/10/14	nicollez	

Matrix: Solid	Client: AESCO Technologies	Collector: Client
Sampled: 04/09/2014 15:15	Site:	
Sample #: 339200-007	Client Sample #: B-9 5-7' Sample A,B	Sample Type:

Analyte	Result	DF	RDL	Units	Analyzed	By	Notes
2-Chlorotoluene	ND	1	5	ug/Kg	04/10/14	nicollez	
4-Chlorotoluene	ND	1	5	ug/Kg	04/10/14	nicollez	
4-Isopropyltoluene	ND	1	5	ug/Kg	04/10/14	nicollez	
4-Methyl-2-pentanone (MIBK)	ND	1	5	ug/Kg	04/10/14	nicollez	
Acetone	ND	1	100	ug/Kg	04/10/14	nicollez	
Allyl Chloride	ND	1	5	ug/Kg	04/10/14	nicollez	
Benzene	ND	1	5	ug/Kg	04/10/14	nicollez	
Bromobenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
Bromochloromethane	ND	1	5	ug/Kg	04/10/14	nicollez	
Bromodichloromethane	ND	1	5	ug/Kg	04/10/14	nicollez	
Bromoform	ND	1	5	ug/Kg	04/10/14	nicollez	
Bromomethane	ND	1	5	ug/Kg	04/10/14	nicollez	
Carbon Tetrachloride	ND	1	5	ug/Kg	04/10/14	nicollez	
Chlorobenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
Chlorodibromomethane	ND	1	5	ug/Kg	04/10/14	nicollez	
Chloroethane	ND	1	5	ug/Kg	04/10/14	nicollez	
Chloroform	ND	1	5	ug/Kg	04/10/14	nicollez	
Chloromethane	ND	1	5	ug/Kg	04/10/14	nicollez	
cis-1,2-Dichloroethene	ND	1	5	ug/Kg	04/10/14	nicollez	
cis-1,3-dichloropropene	ND	1	5	ug/Kg	04/10/14	nicollez	
cis-1,4-dichloro-2-butene	ND	1	5	ug/Kg	04/10/14	nicollez	
Dibromomethane	ND	1	5	ug/Kg	04/10/14	nicollez	
Dichlorodifluoromethane	ND	1	5	ug/Kg	04/10/14	nicollez	
Di-isopropyl ether (DIPE)	ND	1	5	ug/Kg	04/10/14	nicollez	
Ethylbenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
Ethyl-tertbutylether (ETBE)	ND	1	5	ug/Kg	04/10/14	nicollez	
Hexachlorobutadiene	ND	1	5	ug/Kg	04/10/14	nicollez	
Isopropylbenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
m and p-Xylene	ND	1	5	ug/Kg	04/10/14	nicollez	
Methylene chloride	ND	1	5	ug/Kg	04/10/14	nicollez	
Methyl-t-butyl Ether (MTBE)	ND	1	5	ug/Kg	04/10/14	nicollez	
Naphthalene	ND	1	5	ug/Kg	04/10/14	nicollez	
N-butylbenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
N-propylbenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
o-Xylene	ND	1	5	ug/Kg	04/10/14	nicollez	
Sec-butylbenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
Styrene	ND	1	5	ug/Kg	04/10/14	nicollez	
t-Butyl alcohol (TBA)	ND	1	10	ug/Kg	04/10/14	nicollez	
Tert-amylmethylether (TAME)	ND	1	5	ug/Kg	04/10/14	nicollez	
Tert-butylbenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
Tetrachloroethene	ND	1	5	ug/Kg	04/10/14	nicollez	
Toluene	ND	1	5	ug/Kg	04/10/14	nicollez	
trans-1,2-dichloroethene	ND	1	5	ug/Kg	04/10/14	nicollez	
trans-1,3-dichloropropene	ND	1	5	ug/Kg	04/10/14	nicollez	
trans-1,4-dichloro-2-butene	ND	1	5	ug/Kg	04/10/14	nicollez	
Trichloroethene	ND	1	5	ug/Kg	04/10/14	nicollez	
Trichlorofluoromethane	ND	1	5	ug/Kg	04/10/14	nicollez	
Vinyl Chloride	ND	1	5	ug/Kg	04/10/14	nicollez	
Xylenes (Total)	ND	1	5	ug/Kg	04/10/14	nicollez	

Matrix: Solid	**Client:** AESCO Technologies		**Collector:** Client		
Sampled: 04/09/2014 15:15	**Site:**				
Sample #: 339200-007	**Client Sample #:** B-9 5-7' Sample A,B		**Sample Type:**		

Analyte	Result	DF	RDL	Units	Analyzed	By	Notes

Analyte	% Recovery	Limits	Notes
1,2-Dichloroethane-d4 (SUR)	*104*	*70-145*	
4-Bromofluorobenzene (SUR)	*100*	*70-145*	
Dibromodifluoromethane (SUR)	*101*	*70-145*	
Toluene-d8 (SUR)	*102*	*70-145*	

Method: EPA 9045 *NELAC* **Prep Method:** Method **QCBatchID:**

pH	**9.59**	1		04/16/14	mmegaly
Temperature (°C)	**24.7**	1	°C	04/16/14	mmegaly

Matrix:	Solid	**Client:**	AESCO Technologies	**Collector:**	Client
Sampled:	04/09/2014 15:15	**Site:**			
Sample #:	339200-008	**Client Sample #:**	B-3 5-7' Sample A,B	**Sample Type:**	

Analyte	Result	DF	RDL	Units	Analyzed	By	Notes
Method: EPA 418.1 *NELAC* **Prep Method: Method**						**QCBatchID: QC1145660**	
Total Recoverable Petroleum Hydrocarbons	16	1	10	mg/Kg	04/15/14	thun	
Method: EPA 6010 *NELAC* **Prep Method: EPA 3050B**						**QCBatchID: QC1145553**	
Antimony	ND	1	3	mg/Kg	04/11/14	wyu	
Arsenic	3.17	1	1	mg/Kg	04/11/14	wyu	
Barium	15.9	1	1	mg/Kg	04/11/14	wyu	
Beryllium	ND	1	0.5	mg/Kg	04/11/14	wyu	
Cadmium	ND	1	0.5	mg/Kg	04/11/14	wyu	
Chromium	5.95	1	1	mg/Kg	04/11/14	wyu	
Cobalt	2.64	1	0.5	mg/Kg	04/11/14	wyu	
Copper	14.3	1	1	mg/Kg	04/11/14	wyu	
Lead	1.44	1	0.5	mg/Kg	04/11/14	wyu	
Molybdenum	ND	1	1	mg/Kg	04/11/14	wyu	
Nickel	2.75	1	1.5	mg/Kg	04/11/14	wyu	
Selenium	ND	1	1	mg/Kg	04/11/14	wyu	
Silver	ND	1	0.5	mg/Kg	04/11/14	wyu	L
Thallium	ND	1	1	mg/Kg	04/11/14	wyu	
Vanadium	14.7	1	0.5	mg/Kg	04/11/14	wyu	
Zinc	22.8	1	5	mg/Kg	04/11/14	wyu	
Method: EPA 7471 *NELAC* **Prep Method: EPA 7471A**						**QCBatchID: QC1145564**	
Mercury	ND	1	0.14	mg/Kg	04/11/14	wei	
Method: EPA 8015 *NELAC* **Prep Method: EPA 3545**						**QCBatchID: QC1145550**	
TPH Gasoline	ND	1	3	mg/Kg	04/11/14	lyt	
TPH Diesel	ND	1	1	mg/Kg	04/11/14	lyt	
TPH Motor Oil	ND	1	5	mg/Kg	04/11/14	lyt	

Analyte	% Recovery	Limits	Notes
Triacontane (SUR)	*103*	*60-140*	

Method: EPA 8260 *NELAC* Prep Method: EPA 5035						QCBatchID: QC1145577	
1,1,1,2-Tetrachloroethane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,1,1-Trichloroethane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,1,2,2-Tetrachloroethane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,1,2-Trichloroethane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,1,2-Trichlorotrifluoroethane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,1-Dichloroethane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,1-Dichloroethene	ND	1	5	ug/Kg	04/10/14	nicollez	
1,1-Dichloropropene	ND	1	5	ug/Kg	04/10/14	nicollez	
1,2,3-Trichlorobenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
1,2,3-Trichloropropane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,2,4-Trichlorobenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
1,2,4-Trimethylbenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
1,2-Dibromo-3-chloropropane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,2-Dibromoethane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,2-Dichlorobenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
1,2-Dichloroethane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,2-Dichloropropane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,3,5-Trimethylbenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
1,3-Dichlorobenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
1,3-Dichloropropane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,4-Dichlorobenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
2,2-Dichloropropane	ND	1	5	ug/Kg	04/10/14	nicollez	
2-Butanone (MEK)	ND	1	100	ug/Kg	04/10/14	nicollez	
2-Chloroethyl Vinyl Ether	ND	1	5	ug/Kg	04/10/14	nicollez	

Matrix: Solid	**Client:** AESCO Technologies	**Collector:** Client
Sampled: 04/09/2014 15:15	**Site:**	
Sample #: 339200-008	**Client Sample #:** B-3 5-7' Sample A,B	**Sample Type:**

Analyte	Result	DF	RDL	Units	Analyzed	By	Notes
2-Chlorotoluene	ND	1	5	ug/Kg	04/10/14	nicollez	
4-Chlorotoluene	ND	1	5	ug/Kg	04/10/14	nicollez	
4-Isopropyltoluene	ND	1	5	ug/Kg	04/10/14	nicollez	
4-Methyl-2-pentanone (MIBK)	ND	1	5	ug/Kg	04/10/14	nicollez	
Acetone	ND	1	100	ug/Kg	04/10/14	nicollez	
Allyl Chloride	ND	1	5	ug/Kg	04/10/14	nicollez	
Benzene	ND	1	5	ug/Kg	04/10/14	nicollez	
Bromobenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
Bromochloromethane	ND	1	5	ug/Kg	04/10/14	nicollez	
Bromodichloromethane	ND	1	5	ug/Kg	04/10/14	nicollez	
Bromoform	ND	1	5	ug/Kg	04/10/14	nicollez	
Bromomethane	ND	1	5	ug/Kg	04/10/14	nicollez	
Carbon Tetrachloride	ND	1	5	ug/Kg	04/10/14	nicollez	
Chlorobenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
Chlorodibromomethane	ND	1	5	ug/Kg	04/10/14	nicollez	
Chloroethane	ND	1	5	ug/Kg	04/10/14	nicollez	
Chloroform	ND	1	5	ug/Kg	04/10/14	nicollez	
Chloromethane	ND	1	5	ug/Kg	04/10/14	nicollez	
cis-1,2-Dichloroethene	ND	1	5	ug/Kg	04/10/14	nicollez	
cis-1,3-dichloropropene	ND	1	5	ug/Kg	04/10/14	nicollez	
cis-1,4-dichloro-2-butene	ND	1	5	ug/Kg	04/10/14	nicollez	
Dibromomethane	ND	1	5	ug/Kg	04/10/14	nicollez	
Dichlorodifluoromethane	ND	1	5	ug/Kg	04/10/14	nicollez	
Di-isopropyl ether (DIPE)	ND	1	5	ug/Kg	04/10/14	nicollez	
Ethylbenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
Ethyl-tertbutylether (ETBE)	ND	1	5	ug/Kg	04/10/14	nicollez	
Hexachlorobutadiene	ND	1	5	ug/Kg	04/10/14	nicollez	
Isopropylbenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
m and p-Xylene	ND	1	5	ug/Kg	04/10/14	nicollez	
Methylene chloride	ND	1	5	ug/Kg	04/10/14	nicollez	
Methyl-t-butyl Ether (MTBE)	ND	1	5	ug/Kg	04/10/14	nicollez	
Naphthalene	ND	1	5	ug/Kg	04/10/14	nicollez	
N-butylbenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
N-propylbenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
o-Xylene	ND	1	5	ug/Kg	04/10/14	nicollez	
Sec-butylbenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
Styrene	ND	1	5	ug/Kg	04/10/14	nicollez	
t-Butyl alcohol (TBA)	ND	1	10	ug/Kg	04/10/14	nicollez	
Tert-amylmethylether (TAME)	ND	1	5	ug/Kg	04/10/14	nicollez	
Tert-butylbenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
Tetrachloroethene	ND	1	5	ug/Kg	04/10/14	nicollez	
Toluene	ND	1	5	ug/Kg	04/10/14	nicollez	
trans-1,2-dichloroethene	ND	1	5	ug/Kg	04/10/14	nicollez	
trans-1,3-dichloropropene	ND	1	5	ug/Kg	04/10/14	nicollez	
trans-1,4-dichloro-2-butene	ND	1	5	ug/Kg	04/10/14	nicollez	
Trichloroethene	ND	1	5	ug/Kg	04/10/14	nicollez	
Trichlorofluoromethane	ND	1	5	ug/Kg	04/10/14	nicollez	
Vinyl Chloride	ND	1	5	ug/Kg	04/10/14	nicollez	
Xylenes (Total)	ND	1	5	ug/Kg	04/10/14	nicollez	

Matrix: Solid		**Client:** AESCO Technologies		**Collector:** Client	
Sampled: 04/09/2014 15:15		**Site:**			
Sample #: 339200-008		**Client Sample #:** B-3 5-7' Sample A,B		**Sample Type:**	

Analyte	Result	DF	RDL	Units	Analyzed	By	Notes

Analyte	% Recovery	Limits	Notes
1,2-Dichloroethane-d4 (SUR)	*104*	*70-145*	
4-Bromofluorobenzene (SUR)	*97*	*70-145*	
Dibromodifluoromethane (SUR)	*103*	*70-145*	
Toluene-d8 (SUR)	*106*	*70-145*	

Method: EPA 9045 *NELAC* **Prep Method:** Method **QCBatchID:**

pH	**9.61**	1		04/16/14	mmegaly
Temperature (°C)	**24.8**	1	°C	04/16/14	mmegaly

Matrix: Solid	**Client:** AESCO Technologies	**Collector:** Client
Sampled: 04/09/2014 15:15	**Site:**	
Sample #: 339200-009	**Client Sample #:** B-5 5-7' Sample A,B	**Sample Type:**

Analyte	Result	DF	RDL	Units	Analyzed	By	Notes
Method: EPA 418.1 *NELAC* **Prep Method: Method**						**QCBatchID: QC1145660**	
Total Recoverable Petroleum Hydrocarbons	20	1	10	mg/Kg	04/15/14	thun	
Method: EPA 6010 *NELAC* **Prep Method: EPA 3050B**						**QCBatchID: QC1145553**	
Antimony	ND	1	3	mg/Kg	04/11/14	wyu	
Arsenic	3.07	1	1	mg/Kg	04/11/14	wyu	
Barium	17.6	1	1	mg/Kg	04/11/14	wyu	
Beryllium	ND	1	0.5	mg/Kg	04/11/14	wyu	
Cadmium	ND	1	0.5	mg/Kg	04/11/14	wyu	
Chromium	6.79	1	1	mg/Kg	04/11/14	wyu	
Cobalt	3.24	1	0.5	mg/Kg	04/11/14	wyu	
Copper	13.8	1	1	mg/Kg	04/11/14	wyu	
Lead	1.86	1	0.5	mg/Kg	04/11/14	wyu	
Molybdenum	ND	1	1	mg/Kg	04/11/14	wyu	
Nickel	4.30	1	1.5	mg/Kg	04/11/14	wyu	
Selenium	ND	1	1	mg/Kg	04/11/14	wyu	
Silver	ND	1	0.5	mg/Kg	04/11/14	wyu	L
Thallium	ND	1	1	mg/Kg	04/11/14	wyu	
Vanadium	13.1	1	0.5	mg/Kg	04/11/14	wyu	
Zinc	44.1	1	5	mg/Kg	04/11/14	wyu	
Method: EPA 7471 *NELAC* **Prep Method: EPA 7471A**						**QCBatchID: QC1145564**	
Mercury	ND	1	0.14	mg/Kg	04/11/14	wei	
Method: EPA 8015 *NELAC* **Prep Method: EPA 3545**						**QCBatchID: QC1145550**	
TPH Gasoline	ND	1	3	mg/Kg	04/11/14	lyt	
TPH Diesel	1.20	1	1	mg/Kg	04/11/14	lyt	
TPH Motor Oil	ND	1	5	mg/Kg	04/11/14	lyt	

Analyte	% Recovery	Limits	Notes
Triacontane (SUR)	*126*	*60-140*	

Method: EPA 8260 *NELAC* **Prep Method: EPA 5035**						**QCBatchID: QC1145577**	
1,1,1,2-Tetrachloroethane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,1,1-Trichloroethane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,1,2,2-Tetrachloroethane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,1,2-Trichloroethane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,1,2-Trichlorotrifluoroethane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,1-Dichloroethane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,1-Dichloroethene	ND	1	5	ug/Kg	04/10/14	nicollez	
1,1-Dichloropropene	ND	1	5	ug/Kg	04/10/14	nicollez	
1,2,3-Trichlorobenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
1,2,3-Trichloropropane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,2,4-Trichlorobenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
1,2,4-Trimethylbenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
1,2-Dibromo-3-chloropropane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,2-Dibromoethane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,2-Dichlorobenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
1,2-Dichloroethane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,2-Dichloropropane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,3,5-Trimethylbenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
1,3-Dichlorobenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
1,3-Dichloropropane	ND	1	5	ug/Kg	04/10/14	nicollez	
1,4-Dichlorobenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
2,2-Dichloropropane	ND	1	5	ug/Kg	04/10/14	nicollez	
2-Butanone (MEK)	ND	1	100	ug/Kg	04/10/14	nicollez	
2-Chloroethyl Vinyl Ether	ND	1	5	ug/Kg	04/10/14	nicollez	

Matrix:	Solid	Client:	AESCO Technologies	Collector:	Client
Sampled:	04/09/2014 15:15	Site:			
Sample #:	339200-009	Client Sample #:	B-5 5-7' Sample A,B	Sample Type:	

Analyte	Result	DF	RDL	Units	Analyzed	By	Notes
2-Chlorotoluene	ND	1	5	ug/Kg	04/10/14	nicollez	
4-Chlorotoluene	ND	1	5	ug/Kg	04/10/14	nicollez	
4-Isopropyltoluene	ND	1	5	ug/Kg	04/10/14	nicollez	
4-Methyl-2-pentanone (MIBK)	ND	1	5	ug/Kg	04/10/14	nicollez	
Acetone	ND	1	100	ug/Kg	04/10/14	nicollez	
Allyl Chloride	ND	1	5	ug/Kg	04/10/14	nicollez	
Benzene	ND	1	5	ug/Kg	04/10/14	nicollez	
Bromobenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
Bromochloromethane	ND	1	5	ug/Kg	04/10/14	nicollez	
Bromodichloromethane	ND	1	5	ug/Kg	04/10/14	nicollez	
Bromoform	ND	1	5	ug/Kg	04/10/14	nicollez	
Bromomethane	ND	1	5	ug/Kg	04/10/14	nicollez	
Carbon Tetrachloride	ND	1	5	ug/Kg	04/10/14	nicollez	
Chlorobenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
Chlorodibromomethane	ND	1	5	ug/Kg	04/10/14	nicollez	
Chloroethane	ND	1	5	ug/Kg	04/10/14	nicollez	
Chloroform	ND	1	5	ug/Kg	04/10/14	nicollez	
Chloromethane	ND	1	5	ug/Kg	04/10/14	nicollez	
cis-1,2-Dichloroethene	ND	1	5	ug/Kg	04/10/14	nicollez	
cis-1,3-dichloropropene	ND	1	5	ug/Kg	04/10/14	nicollez	
cis-1,4-dichloro-2-butene	ND	1	5	ug/Kg	04/10/14	nicollez	
Dibromomethane	ND	1	5	ug/Kg	04/10/14	nicollez	
Dichlorodifluoromethane	ND	1	5	ug/Kg	04/10/14	nicollez	
Di-isopropyl ether (DIPE)	ND	1	5	ug/Kg	04/10/14	nicollez	
Ethylbenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
Ethyl-tertbutylether (ETBE)	ND	1	5	ug/Kg	04/10/14	nicollez	
Hexachlorobutadiene	ND	1	5	ug/Kg	04/10/14	nicollez	
Isopropylbenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
m and p-Xylene	ND	1	5	ug/Kg	04/10/14	nicollez	
Methylene chloride	ND	1	5	ug/Kg	04/10/14	nicollez	
Methyl-t-butyl Ether (MTBE)	ND	1	5	ug/Kg	04/10/14	nicollez	
Naphthalene	ND	1	5	ug/Kg	04/10/14	nicollez	
N-butylbenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
N-propylbenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
o-Xylene	ND	1	5	ug/Kg	04/10/14	nicollez	
Sec-butylbenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
Styrene	ND	1	5	ug/Kg	04/10/14	nicollez	
t-Butyl alcohol (TBA)	ND	1	10	ug/Kg	04/10/14	nicollez	
Tert-amylmethylether (TAME)	ND	1	5	ug/Kg	04/10/14	nicollez	
Tert-butylbenzene	ND	1	5	ug/Kg	04/10/14	nicollez	
Tetrachloroethene	ND	1	5	ug/Kg	04/10/14	nicollez	
Toluene	ND	1	5	ug/Kg	04/10/14	nicollez	
trans-1,2-dichloroethene	ND	1	5	ug/Kg	04/10/14	nicollez	
trans-1,3-dichloropropene	ND	1	5	ug/Kg	04/10/14	nicollez	
trans-1,4-dichloro-2-butene	ND	1	5	ug/Kg	04/10/14	nicollez	
Trichloroethene	ND	1	5	ug/Kg	04/10/14	nicollez	
Trichlorofluoromethane	ND	1	5	ug/Kg	04/10/14	nicollez	
Vinyl Chloride	ND	1	5	ug/Kg	04/10/14	nicollez	
Xylenes (Total)	ND	1	5	ug/Kg	04/10/14	nicollez	

Matrix: Solid	**Client:** AESCO Technologies	**Collector:** Client
Sampled: 04/09/2014 15:15	**Site:**	
Sample #: 339200-009	**Client Sample #:** B-5 5-7' Sample A,B	**Sample Type:**

Analyte	Result	DF	RDL	Units	Analyzed	By	Notes

Analyte	% Recovery	Limits	Notes
1,2-Dichloroethane-d4 (SUR)	*102*	*70-145*	
4-Bromofluorobenzene (SUR)	*99*	*70-145*	
Dibromodifluoromethane (SUR)	*104*	*70-145*	
Toluene-d8 (SUR)	*108*	*70-145*	

Method: EPA 9045 *NELAC* **Prep Method:** Method **QCBatchID:**

pH	9.67	1		04/16/14	mmegaly
Temperature (°C)	24.6	1	°C	04/16/14	mmegaly

QCBatchID: QC1145550	Analyst: lytagas	Method: EPA 8015B
Matrix: Solid	Analyzed: 04/10/2014	Instrument: SVOA-GC (group)

Blank Summary

Analyte	Blank Result	Units		RDL	Notes
QC1145550MB1					
TPH (C10 to C22)	ND	mg/Kg		3	
TPH (C22 to C36)	ND	mg/Kg		5	
TPH (C6 to C10)	ND	mg/Kg		3	
TPH Diesel	ND	mg/Kg		1	
TPH Gasoline	ND	mg/Kg		3	
TPH Motor Oil	ND	mg/Kg		5	

Lab Control Spike/ Lab Control Spike Duplicate Summary

Analyte	Spike Amount		Spike Result		Units	Recoveries		RPD	Limits		Notes
	LCS	LCSD	LCS	LCSD		LCS	LCSD		%Rec	RPD	
QC1145550LCS1											
TPH Diesel	25		17.8		mg/Kg	71			70-130		

Matrix Spike/Matrix Spike Duplicate Summary

Analyte	Sample Amount	Spike Amount		Spike Result		Units	Recoveries		RPD	Limits		Notes
		MS	MSD	MS	MSD		MS	MSD		%Rec	RPD	
QC1145550MS1, QC1145550MSD1												**Source: 339082-004**
TPH Diesel		25	25	25.6	18.4	mg/Kg	102	74	32.7	70-130	30	M

QCBatchID: QC1145553	Analyst: wyu	Method: EPA 6010B
Matrix: Solid	Analyzed: 04/10/2014	Instrument: AAICP (group)

Blank Summary

Analyte	Blank Result	Units		RDL	Notes
QC1145553MB1					
Antimony	ND	mg/Kg		3	
Arsenic	ND	mg/Kg		1	
Barium	ND	mg/Kg		1	
Beryllium	ND	mg/Kg		0.5	
Cadmium	ND	mg/Kg		0.5	
Chromium	ND	mg/Kg		1	
Cobalt	ND	mg/Kg		0.5	
Copper	ND	mg/Kg		1	
Lead	ND	mg/Kg		0.5	
Molybdenum	ND	mg/Kg		1	
Nickel	ND	mg/Kg		1.5	
Selenium	ND	mg/Kg		1	
Silver	ND	mg/Kg		0.5	
Thallium	ND	mg/Kg		1	
Vanadium	ND	mg/Kg		0.5	
Zinc	ND	mg/Kg		5	

Lab Control Spike/ Lab Control Spike Duplicate Summary

Analyte	Spike Amount LCS	Spike Amount LCSD	Spike Result LCS	Spike Result LCSD	Units	Recoveries LCS	Recoveries LCSD	RPD	Limits %Rec	Limits RPD	Notes
QC1145553LCS1											
Antimony	200		196		mg/Kg	98			80-120		
Arsenic	200		201		mg/Kg	101			80-120		
Barium	200		200		mg/Kg	100			80-120		
Beryllium	200		209		mg/Kg	105			80-120		
Cadmium	200		198		mg/Kg	99			80-120		
Chromium	200		201		mg/Kg	101			80-120		
Cobalt	200		204		mg/Kg	102			80-120		
Copper	200		194		mg/Kg	97			80-120		
Lead	200		205		mg/Kg	103			80-120		
Molybdenum	200		189		mg/Kg	95			80-120		
Nickel	200		204		mg/Kg	102			80-120		
Selenium	200		195		mg/Kg	98			80-120		
Silver	100		77.6		mg/Kg	78			80-120		L
Thallium	200		205		mg/Kg	103			80-120		
Vanadium	200		196		mg/Kg	98			80-120		
Zinc	200		203		mg/Kg	102			80-120		

Matrix Spike/Matrix Spike Duplicate Summary

Analyte	Sample Amount	Spike Amount MS	Spike Amount MSD	Spike Result MS	Spike Result MSD	Units	Recoveries MS	Recoveries MSD	RPD	Limits %Rec	Limits RPD	Notes
QC1145553MS1, QC1145553MSD1												Source: 339200-005
Antimony	ND	100	100	78.4	77.5	mg/Kg	78	78	1.2	75-125	20	
Arsenic	2.75	100	100	108	105	mg/Kg	105	102	2.8	75-125	20	
Barium	13.0	100	100	118	128	mg/Kg	105	115	8.1	75-125	20	
Beryllium	0.2	100	100	107	115	mg/Kg	107	115	7.2	75-125	20	
Cadmium	ND	100	100	98.3	107	mg/Kg	98	107	8.5	75-125	20	
Chromium	5.20	100	100	102	111	mg/Kg	97	106	8.5	75-125	20	
Cobalt	2.24	100	100	101	108	mg/Kg	99	106	6.7	75-125	20	
Copper	5.68	100	100	101	109	mg/Kg	95	103	7.6	75-125	20	

QCBatchID: QC1145553	Analyst: wyu	Method: EPA 6010B
Matrix: Solid	Analyzed: 04/10/2014	Instrument: AAICP (group)

Analyte	Sample Amount	Spike Amount MS	Spike Amount MSD	Spike Result MS	Spike Result MSD	Units	Recoveries MS	Recoveries MSD	RPD	Limits %Rec	Limits RPD	Notes
QC1145553MS1, QC1145553MSD1												**Source: 339200-005**
Lead	1.49	100	100	102	99.9	mg/Kg	101	98	2.1	75-125	20	
Molybdenum	0.2	100	100	91.0	90.1	mg/Kg	91	90	1.0	75-125	20	
Nickel	2.59	100	100	102	110	mg/Kg	99	107	7.5	75-125	20	
Selenium	0.9	100	100	104	102	mg/Kg	103	101	1.9	75-125	20	
Silver	ND	50	50	21.5	21.4	mg/Kg	43	43	0.5	75-125	20	M
Thallium	ND	100	100	105	104	mg/Kg	105	104	1.0	75-125	20	
Vanadium	10.3	100	100	109	119	mg/Kg	99	109	8.8	75-125	20	
Zinc	10.2	100	100	109	115	mg/Kg	99	105	5.4	75-125	20	

QCBatchID: QC1145564	Analyst: wei	Method: EPA 7471A
Matrix: Solid	Analyzed: 04/11/2014	Instrument: AAICP-HG1

Blank Summary

Analyte	Blank Result	Units		RDL	Notes
QC1145564MB1					
Mercury	ND	mg/Kg		0.14	

Lab Control Spike/ Lab Control Spike Duplicate Summary

Analyte	Spike Amount LCS	Spike Amount LCSD	Spike Result LCS	Spike Result LCSD	Units	Recoveries LCS	Recoveries LCSD	RPD	Limits %Rec	Limits RPD	Notes
QC1145564LCS1											
Mercury	0.83		0.88		mg/Kg	106			80-120		

Matrix Spike/Matrix Spike Duplicate Summary

Analyte	Sample Amount	Spike Amount MS	Spike Amount MSD	Spike Result MS	Spike Result MSD	Units	Recoveries MS	Recoveries MSD	RPD	Limits %Rec	Limits RPD	Notes
QC1145564MS1, QC1145564MSD1												Source: 339110-001
Mercury	ND	0.83	0.83	0.85	0.75	mg/Kg	102	90	12.5	75-125	20	

QCBatchID: QC1145577	Analyst: nicollez	Method: EPA 8260B
Matrix: Solid	Analyzed: 04/10/2014	Instrument: VOA-MS (group)

Blank Summary

Analyte	Blank Result	Units		RDL	Notes
QC1145577MB1					
1,1,1,2-Tetrachloroethane	ND	ug/Kg		5	
1,1,1-Trichloroethane	ND	ug/Kg		5	
1,1,2,2-Tetrachloroethane	ND	ug/Kg		5	
1,1,2-Trichloroethane	ND	ug/Kg		5	
1,1,2-Trichlorotrifluoroethane	ND	ug/Kg		5	
1,1-Dichloroethane	ND	ug/Kg		5	
1,1-Dichloroethene	ND	ug/Kg		5	
1,1-Dichloropropene	ND	ug/Kg		5	
1,2,3-Trichlorobenzene	ND	ug/Kg		5	
1,2,3-Trichloropropane	ND	ug/Kg		5	
1,2,4-Trichlorobenzene	ND	ug/Kg		5	
1,2,4-Trimethylbenzene	ND	ug/Kg		5	
1,2-Dibromo-3-chloropropane	ND	ug/Kg		5	
1,2-Dibromoethane	ND	ug/Kg		5	
1,2-Dichlorobenzene	ND	ug/Kg		5	
1,2-Dichloroethane	ND	ug/Kg		5	
1,2-Dichloropropane	ND	ug/Kg		5	
1,3,5-Trimethylbenzene	ND	ug/Kg		5	
1,3-Dichlorobenzene	ND	ug/Kg		5	
1,3-Dichloropropane	ND	ug/Kg		5	
1,4-Dichlorobenzene	ND	ug/Kg		5	
2,2-Dichloropropane	ND	ug/Kg		5	
2-Butanone (MEK)	ND	ug/Kg		100	
2-Chloroethyl Vinyl Ether	ND	ug/Kg		5	
2-Chlorotoluene	ND	ug/Kg		5	
4-Chlorotoluene	ND	ug/Kg		5	
4-Isopropyltoluene	ND	ug/Kg		5	
4-Methyl-2-pentanone (MIBK)	ND	ug/Kg		5	
Acetone	ND	ug/Kg		100	
Allyl Chloride	ND	ug/Kg		5	
Benzene	ND	ug/Kg		5	
Bromobenzene	ND	ug/Kg		5	
Bromochloromethane	ND	ug/Kg		5	
Bromodichloromethane	ND	ug/Kg		5	
Bromoform	ND	ug/Kg		5	
Bromomethane	ND	ug/Kg		5	
Carbon Tetrachloride	ND	ug/Kg		5	
Chlorobenzene	ND	ug/Kg		5	
Chlorodibromomethane	ND	ug/Kg		5	
Chloroethane	ND	ug/Kg		5	
Chloroform	ND	ug/Kg		5	
Chloromethane	ND	ug/Kg		5	
cis-1,2-Dichloroethene	ND	ug/Kg		5	
cis-1,3-dichloropropene	ND	ug/Kg		5	
cis-1,4-dichloro-2-butene	ND	ug/Kg		5	
Dibromomethane	ND	ug/Kg		5	
Dichlorodifluoromethane	ND	ug/Kg		5	
Di-isopropyl ether (DIPE)	ND	ug/Kg		5	
Ethylbenzene	ND	ug/Kg		5	
Ethyl-tertbutylether (ETBE)	ND	ug/Kg		5	

QCBatchID: QC1145577	Analyst: nicollez	Method: EPA 8260B
Matrix: Solid	Analyzed: 04/10/2014	Instrument: VOA-MS (group)

Analyte	Blank Result	Units		RDL	Notes
QC1145577MB1					
Hexachlorobutadiene	ND	ug/Kg		5	
Isopropylbenzene	ND	ug/Kg		5	
m and p-Xylene	ND	ug/Kg		5	
Methylene chloride	ND	ug/Kg		5	
Methyl-t-butyl Ether (MTBE)	ND	ug/Kg		5	
Naphthalene	ND	ug/Kg		5	
N-butylbenzene	ND	ug/Kg		5	
N-propylbenzene	ND	ug/Kg		5	
o-Xylene	ND	ug/Kg		5	
Sec-butylbenzene	ND	ug/Kg		5	
Styrene	ND	ug/Kg		5	
t-Butyl alcohol (TBA)	ND	ug/Kg		10	
Tert-amylmethylether (TAME)	ND	ug/Kg		5	
Tert-butylbenzene	ND	ug/Kg		5	
Tetrachloroethene	ND	ug/Kg		5	
Toluene	ND	ug/Kg		5	
TPH Gasoline	ND	ug/Kg		100	
trans-1,2-dichloroethene	ND	ug/Kg		5	
trans-1,3-dichloropropene	ND	ug/Kg		5	
trans-1,4-dichloro-2-butene	ND	ug/Kg		5	
Trichloroethene	ND	ug/Kg		5	
Trichlorofluoromethane	ND	ug/Kg		5	
Vinyl Chloride	ND	ug/Kg		5	
Xylenes (Total)	ND	ug/Kg		5	

Lab Control Spike/ Lab Control Spike Duplicate Summary

Analyte	Spike Amount LCS	Spike Amount LCSD	Spike Result LCS	Spike Result LCSD	Units	Recoveries LCS	Recoveries LCSD	RPD	Limits %Rec	Limits RPD	Notes
QC1145577LCS1											
1,1-Dichloroethene	50		47		ug/Kg	94			59-172		
Benzene	50		49		ug/Kg	98			62-137		
Chlorobenzene	50		50		ug/Kg	100			60-133		
Methyl-t-butyl Ether (MTBE)	50		47		ug/Kg	94			62-137		
Toluene	50		52		ug/Kg	104			59-139		
Trichloroethene	50		49		ug/Kg	98			66-142		

Matrix Spike/Matrix Spike Duplicate Summary

Analyte	Sample Amount	Spike Amount MS	Spike Amount MSD	Spike Result MS	Spike Result MSD	Units	Recoveries MS	Recoveries MSD	RPD	Limits %Rec	Limits RPD	Notes
QC1145577MS1, QC1145577MSD1												Source: 339200-009
1,1-Dichloroethene	ND	50	50	43	43	ug/Kg	86	86	0.0	59-172	22	
Benzene	ND	50	50	46	45	ug/Kg	92	90	2.2	62-137	24	
Chlorobenzene	ND	50	50	48	46	ug/Kg	96	92	4.3	60-133	24	
Methyl-t-butyl Ether (MTBE)	ND	50	50	44	43	ug/Kg	88	86	2.3	62-137	21	
Toluene	ND	50	50	48	48	ug/Kg	96	96	0.0	59-139	21	
Trichloroethene	ND	50	50	46	44	ug/Kg	92	88	4.4	66-142	21	

QCBatchID: QC1145584	Analyst: lytagas	Method: EPA 8015B
Matrix: Water	Analyzed: 04/11/2014	Instrument: SVOA-GC (group)

Blank Summary

Analyte	Blank Result	Units		RDL	Notes
QC1145584MB1					
TPH (C10 to C22)	ND	mg/L		0.2	
TPH (C22 to C36)	ND	mg/L		0.3	
TPH (C6 to C10)	ND	mg/L		0.2	
TPH Diesel	ND	mg/L		0.1	
TPH Gasoline	ND	mg/L		0.2	
TPH Motor Oil	ND	mg/L		0.3	

Lab Control Spike/ Lab Control Spike Duplicate Summary

Analyte	Spike Amount		Spike Result		Units	Recoveries		RPD	Limits		Notes
	LCS	LCSD	LCS	LCSD		LCS	LCSD		%Rec	RPD	
QC1145584LCS1, QC1145584LCSD1											
TPH Diesel	1	1	0.85	0.90	mg/L	85	90	6	57-122	30	

QCBatchID: QC1145660	Analyst: Iytagas	Method: EPA 418.1
Matrix: Solid	Analyzed: 04/15/2014	Instrument: SVOA-GC (group)

Blank Summary

Analyte	Blank Result	Units		RDL	Notes
QC1145660MB1					
Total Recoverable Petroleum Hydrocarbons	ND	mg/Kg		10	

Lab Control Spike/ Lab Control Spike Duplicate Summary

Analyte	Spike Amount		Spike Result		Units	Recoveries		RPD	Limits		Notes
	LCS	LCSD	LCS	LCSD		LCS	LCSD		%Rec	RPD	
QC1145660LCS1, QC1145660LCSD1											
Total Recoverable Petroleum Hydrocarbons	82	82	80	72	mg/Kg	98	88	11	80-120		

Notes and Definitions

B	Analyte was present in an associated method blank. Associated sample data was reported with qualifier.
BQ1	No valid test replicates. Result may be greater. Best result was reported with qualifier. Sample toxicity possible.
BQ2	No valid test replicates.
BQ3	Minimum DO is less than 1.0 mg/L. Result may be greater and reported with qualifier.
C	Laboratory Contamination.
D	The sample duplicate RPD was not within control limits, the sample data was reported without further clarification.
DF	Dilution Factor
DW	Sample result is calculated on a dry weigh basis
J	Reported value is estimated
L	The laboratory control sample (LCS) or laboratory control sample duplicate (LCSD) was out of control limits. Associated sample data was reported with qualifier.
M	The matrix spike (MS) or matrix spike duplicate (MSD) was not within control limits due to matrix interference. The associated LCS and/or LCSD was within control limits and the sample data was reported without further clarification.
MDL	Method Detection Limit
NC	The analyte concentration in the sample exceeded the spike level by a factor of four or greater, spike recovery and limits do not apply.
ND	Analyte was not detected or was less than the detection limit.
P	Sample was received without proper preservation according to EPA guidelines.
Q1	Analyte Calibration Verification exceeds criteria and the result was reported with qualifier.
Q2	Analyte calibration was not verified and the result was estimated and reported with qualifier.
Q3	Analyte initial calibration was not available or exceeds criteria. The result was estimated and reported with qualifier.
Q4	Analyte result out of calibration range and was reported with qualifier
RDL	Reporting Detection Limit
S	The surrogate recovery was out of control limits due to matrix interference. The associated method blank surrogate recovery was within control limits and the sample data was reported without further clarification.
T	Sample was extracted/analyzed past the holding time.
T2	Sample was analyzed ASAP but received and analyzed past the 15 minute holding time.
TIC	Tentatively Identified Compounds

ASSOCIATED LABORATORIES
806 North Batavia – Orange, California 92868 – 714-771-6900 FAX 714-538-1209

SAMPLE ACCEPTANCE CHECKLIST

Section 1
Client: AESCO. Project: HARLEY ELLIS
Date Received: 4/09/14 Sampler's Name: (Yes) / No
Sample temperature: _____
Sample(s) received in cooler: (Yes) No (Skip Section 2)
Shipping Information: _____

Section 2
Was the cooler packed with: ✓ Ice ___ Ice Packs ___ Bubble Wrap ___ Styrofoam ___ Paper ___ None ___ Other _____
Cooler Temperature: 4°C

(Acceptance range is 0 to 6 Deg. C. or arrival on ice; For Microbiology sample ≤10 Deg. C or arrival on ice)

Section 3	YES	NO	N/A
Was a COC received?	✓		
Is it properly completed? (IDs, sampling date and time, signature, test)	✓		
Were custody seals present?		✓	
If Yes – were they intact?			✓
Were all samples sealed in plastic bags?		✓	
Did all samples arrive intact? If no, indicate below.	✓		
Did all bottle labels agree with COC? (ID, dates and times)	✓		
Were correct containers used for the tests required?	✓		
Was a sufficient amount of sample sent for tests indicated?	✓		
Was there headspace in VOA vials?			✓
Were the containers labeled with correct preservatives?			✓
Was total residual chlorine measured (Fish Bioassay samples only)? *			✓

*: If the answer is no, please inform Fish Bioassay Dept. immediately.

Section 4
Explanations/Comments

Section 5
Was Project Manager notified of discrepancies: Y / N N/A
Project Manager's response: _____

Completed By: [signature] Date: 4/09/14

ASSOCIATED LABORATORIES
806 North Batavia • Orange, CA 92868
Phone: (714) 771-6900 • Fax: (714) 538-1209

Chain of Custody Record

Lab Job No. 339200
Page ____ of ____

REQUIRED TURN AROUND TIME: Standard: X
72 Hours: ____ 48 Hours: ____ 24 Hours: ____

CUSTOMER INFORMATION	PROJECT INFORMATION
COMPANY: AESCO	PROJECT NAME: Harley Ellis
SEND REPORT TO: Debra Perez	NUMBER: 20140185-C8480
EMAIL: Debra.Perez@Aescotech.com	ADDRESS: 4000 Olympic Plaza Long Beach, CA
ADDRESS: 17782 Georgetown Ln Huntington Beach, CA	P.O. #:
PHONE: (714) 375-3830	SAMPLED BY: JJ

	Sample ID	Date	Time	Matrix	Container Number/Size	Pres.	PH EPA Meth 9045	CAM 17 EPA 6010B 7471A	VOC EPA 8260B	TPH EPA 418.1	BTEX EPA 8015B	TPH EPA 8021B (water)				Test Instructions & Comments
1	B-6 water	4/9/14	3:15	W	1 lit.							X				
2	B-3 water	4/9/14		W								X				
3	B-5 water			W								X				
4	B-9 water			W								X				
5	B-6 5-7' Sample A			S	8oz jar		X	X	X	X	X					
6	B-6 5-7' Sample B			S			X	X	X	X	X					
7	B-5 13-15' Sample A			S			X	X	X	X	X					
8	B-5 13-15' Sample B			S			X	X	X	X	X					
9	B-9 5-7' Sample A			S			X	X	X	X	X					
10	B-9 5-7' Sample B			S			X	X	X	X	X					
11	B-3 5-7' Sample A			S			X	X	X	X	X					
12	B-3 5-7' Sample B			S			X	X	X	X	X					
13	B-5 5-7' Sample A			S			X	X	X	X	X					
14	B-5 5-7' Sample B			S			X	X	X	X	X					
15																

8015 D:O GAS. on soil and water

Total No. of Samples: Method of Shipment: Preservative: 1= Ice 2 =HCl 3 =HNO_3 4 =H_2SO_4 5 =NaOH 6 =Other

Relinquished by 1.	Received By: 1.	Relinquished by 2.	Received By: 2.	Relinquished by 3.	Received By: 3.
JJ Harrington					
Signature:	Signature:	Signature:	Signature:	Signature:	Signature:
Printed Name:	Printed Name: ZAIN	Printed Name:	Printed Name:	Printed Name:	Printed Name:
Date: 4/09/14 Time: 1530	Date: 4/09/14 Time: 1530	Date: Time:	Date: Time:	Date: Time:	Date: Time:

Distribution: White - Laboratory Canary - Laboratory Pink - Project/Account Manager Goldenrod - Sampler/Originator

APPENDIX

LOAD DATA FOR DRILLED PIERS and DRIVEN PILES

AESCO TECHNOLOGIES
18548 Beach Blvd.
Huntington Beach, California 92648

(714) 593-2567
Fax (714) 593-0022

25 Apr 14

2014085-C8050

Belmont Plaza Pool Rebuild
4000 East Olympic Plaza, Long Beach

ALLOWABLE UPLIFT LOADS (TONS)
FOR SQUARE/RECTANGULAR PILES
FACTOR OF SAFETY = 2.0

TIP DEPTH BELOW SURFACE (FT.)	PILE DIMENSIONS (IN.) (TOP NO. = WIDTH - BOTTOM NO. = LENGTH)							
	12	14	16	18	0	0	0	0
	12	14	16	18	0	0	0	0
0	0	0	0	0	0	0	0	0
4	0	0	0	0	0	0	0	0
5	0	0	0	0	0	0	0	0
10	1	1	1	1	0	0	0	0
20	3	3	4	4	0	0	0	0
30	7	8	9	10	0	0	0	0
40	13	15	18	20	0	0	0	0
50	24	28	32	36	0	0	0	0
60	37	43	49	55	0	0	0	0
70	52	61	69	78	0	0	0	0
80	69	81	93	104	0	0	0	0

AESCO TECHNOLOGIES
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Huntington Beach, California 92648

(714) 593-2567
Fax (714) 593-0022

25 Apr 14

2014085-C8050

Belmont Plaza Pool Rebuild
4000 East Olympic Plaza, Long Beach

ALLOWABLE DOWNWARD LOADS (TONS)
FOR SQUARE/RECTANGULAR PILES
FACTOR OF SAFETY = 2.0

TIP DEPTH BELOW SURFACE (FT.)	PILE DIMENSIONS (IN.) (TOP NO. = WIDTH - BOTTOM NO. = LENGTH)							
	12	14	16	18	0	0	0	0
	12	14	16	18	0	0	0	0
0								
4	0	0	0	0	0	0	0	0
5	0	1	1	1	0	0	0	0
10	2	2	3	3	0	0	0	0
20	5	6	8	9	0	0	0	0
30	12	15	18	22	0	0	0	0
40	25	32	40	48	0	0	0	0
50	39	49	59	71	0	0	0	0
60	56	68	82	98	0	0	0	0
70	74	91	109	128	0	0	0	0
80	95	116	138	161	0	0	0	0

AESCO

17782 Georgetown Lane
Huntington Beach, CA 92647

(714) 375-3830
Fax (714) 375-3831

25 Apr 14
2014085-C8050

Belmont Plaza Pool Rebuild
4000 East Olympic Plaza, Long Beach

ALLOWABLE UPLIFT LOADS (KIPS)
FOR DRILLED SHAFTS
FACTOR OF SAFETY = 2.0

TIP DEPTH BELOW SURFACE (FT.)	SHAFT DIAMETER (IN.) 18	24	30	36	42	48	54	60
0	0	0	0	0	0	0	0	0
4	0	0	0	0	0	0	0	0
5	0	0	0	1	1	1	1	1
10	2	2	3	3	4	4	5	5
20	7	9	11	14	16	18	20	23
30	16	21	26	31	36	41	47	52
40	31	41	52	62	73	83	93	104
50	56	75	93	112	131	149	168	186
60	86	115	144	173	201	230	259	288
70	122	163	204	244	285	326	366	407
80	164	218	273	327	382	436	491	545

25 Apr 14
2014085-C8050

Belmont Plaza Pool Rebuild
4000 East Olympic Plaza, Long Beach

ALLOWABLE DOWNWARD LOADS (KIPS)
FOR DRILLED SHAFTS
FACTOR OF SAFETY = 2.0

TIP DEPTH BELOW SURFACE (FT.)	SHAFT DIAMETER (IN.) 18	24	30	36	42	48	54	60
0								
4	0	0	0	0	0	0	0	0
5	1	3	4	6	9	13	17	22
10	5	8	12	17	22	29	36	45
20	14	23	33	45	58	74	92	111
30	34	54	79	107	140	178	220	267
40	75	119	174	238	313	398	494	600
50	111	173	247	335	435	547	673	812
60	153	234	331	442	569	711	869	1043
70	201	303	423	560	716	890	1082	1292
80	254	380	527	695	884	1095	1327	1582