DRAFT
GEOTECHNICAL FEASIBILITY STUDY
PROPOSED ARTIC PHASE 1 PROJECT
ANAHEIM, CALIFORNIA

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October 23, 2009

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Attention: Ms. Donna McCormick
Principal

Subject: DRAFT
Geotechnical Feasibility Study
Proposed ARTIC Phase 1 Project
Anaheim, California

Dear Ms. McCormick:

Kleinfelder West, Inc. (Kleinfelder) is pleased to present this report summarizing the geotechnical feasibility study for the proposed Anaheim Regional Transportation Intermodal Center (ARTIC) Phase 1 project located on the east side of Douglass Road between Katella Avenue and the railroad in Anaheim, California. The purpose of this feasibility study was to evaluate the subsurface soil conditions at the site in order to provide preliminary geotechnical conclusions for project feasibility to support the project's Environmental Documents. This feasibility study is not intended to be a design-level geotechnical study, and additional field and laboratory testing will be required in order to finalize the geotechnical recommendations for the design and construction. The conclusions and recommendations presented in this report are subject to the limitations presented in Section 6.

We appreciate the opportunity to provide geotechnical engineering services to you on this project. If you have any questions regarding this report or if we can be of further service, please do not hesitate to contact the undersigned at (949) 727-4466.

Respectfully submitted,

KLEINFELDER WEST, INC.

Brian E. Crystal, P.E., G.E.
Geotechnical Group Manager

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Principal Geotechnical Engineer

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Important Information About Your

Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes

The following information is provided to help you manage your risks.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one - not even you* - should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from alight industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes - even minor ones - and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ-sometimes significantly from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final*, because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the express purpose of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in-this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely on Your ASFE-Member Geotechnical Engineer For Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your ASFE-member geotechnical engineer for more information.

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EXECUTIVE SUMMARY

This report presents the results of our geotechnical feasibility study the proposed Anaheim Regional Transportation Intermodal Center (ARTIC) Phase 1 project. Kleinfelder understands that the Orange County Transportation Authority (OCTA) and the City of Anaheim plan to develop a major transit facility, known as ARTIC. The proposed facility will serve Metrolink, Amtrak, fixed-route buses, and will be a regional terminal for the future California High Speed Train. This study was concentrated on the east side of Douglass Road and north of the railroad, where the main ARTIC building will be situated.

Preliminary recommendations for improvements within the Caltrans right-of-way were presented in a Preliminary Foundation Report, dated July 8, 2009 (Kleinfelder, 2009b). Preliminary recommendations for the remaining improvements, such as the lowering and widening of Douglass Road, pedestrian railroad crossings, and retaining structures, were presented in a Preliminary Foundation Report, dated July 17, 2009 (Kleinfelder, 2009c).

The purpose of this feasibility study is to evaluate the subsurface soil conditions at the site in order to provide preliminary geotechnical conclusions for project feasibility to support the project's Environmental Documents. This feasibility study is not intended to be a design-level geotechnical study, and additional field and laboratory testing will be required in order to finalize the geotechnical recommendations for the design and construction.

The subsurface conditions at the site were recently explored by Kleinfelder by drilling 5 borings, installing 2 groundwater monitoring wells, and advancing 7 Cone Penetration Tests (CPTs). Soil materials encountered during the subsurface explorations consisted of artificial fill underlain by young alluvium. Locally derived sand material appears to have been used as fill and compaction appears to be highly variable. This fill is considered undocumented and not suitable for structural support. The fill depth varies throughout the site and is difficult to determine due to the nature of the material. Based on our interpretation of the materials encountered, the fill depths range between about 7 and 21 feet in the vicinity of our borings. It should be noted that deeper fill may be present at other locations not explored. Alluvial deposits were observed to underlie the fill in the borings. The alluvium consists predominantly of interbedded layers and lenses of poorly graded sand, silty sand, lean clay and sandy silt.

The groundwater encountered during Kleinfelder's field exploration appears to be perched. Groundwater was measured at a depth of 23 feet (Elevation 134 feet) in one of our monitoring wells (Well W-1). It should be noted that Kleinfelder's groundwater measurements were taken during a relatively long dry period and mostly likely are not representative of the groundwater conditions during the rainy season. In 1994, wet soil samples (indication of groundwater) were logged adjacent to the site and the LOSSAN railroad corridor at a depth of approximately 50 feet (SCRRA, 1994), and in 1999 groundwater was measured at a depth of about 34 feet near the intersection of Katella Avenue and South Douglass Road (Coleman Geotechnical, 1999). In June 2006, OCWD mapped groundwater levels near the site at a depth of approximately 60 feet. In 2001, an evaluation of the historically shallowest groundwater levels was conducted by the CGS (Greenwood and Pridmore, 2001) for the area, which included the site. They determined the highest historical groundwater to be approximately 20 feet deep for the project site.

Based on the results of our field explorations performed to date, laboratory testing and geotechnical analyses conducted during this study, it is our professional opinion that the proposed project is geotechnically feasible, provided the recommendations presented in this feasibility study report and future design reports are incorporated into the project design and construction. The primary geotechnical constraints that will have a significant impact to the cost of developing the site include: 1) the compressibility of the upper alluvial soils (static settlement); 2) the potential for seismically-induced settlement and slope instability/lateral spreading due to liquefaction; 3) the presence of deep undocumented fill; and 4) the potential for shallow groundwater adversely affecting the design and construction of subterranean parking levels. The following key items are conclusions developed from our feasibility study.

- The site is within a State of California Hazard Zone for Liquefaction (CDMG, 1998). Because of the depth to groundwater and the soil types encountered during our investigation, the potential for liquefaction at the site is high. Seismically-induced settlement of saturated sandy soils due to strong ground shaking during a design-level seismic event could be on the order of 3 to 6 inches with differential settlements on the order of 2 to 4 inches.

- The site is bounded by the Santa Ana River on the east, which has been channelized. The top of the embankment to the channel bottom is approximately 15 to 20 feet high with an inclination of approximately 2:1 (horizontal to vertical). Preliminary analyses indicate that, due to liquefaction, the channel slope will not be stable during the design earthquake and may affect the site improvements. A detailed evaluation of the stability of the Santa Ana channel slope should be performed during the design-level geotechnical study in order to design mitigative measures to protect the site improvements.

- According to the 2007 CBC, sites subject to liquefaction should be classified as Site Class F, which requires a site response analysis. However, ACSE7-05, which is the basis for the 2007 CBC, suggests that for a short period (less than ½ second) structure on liquefiable soils, Site Class D or E may be used instead of Site Class F to estimate design seismic loading on the structure. The project structural engineer should determine if a site-specific response analysis is required during the design phase for the structural design.

- The long-term performance of the subterranean parking slab and subterranean walls will be affected by the water level if not considered in the design. Due to the potential for an increased groundwater elevation from rainfall, over-irrigation, and the proximity to the Santa Ana River, we recommend that a preliminary design groundwater elevation of 145 feet, which roughly corresponds to the adjacent river bottom, be used for preliminary design. We recommend that all subterranean walls and floor slabs that extend to and below Elevation 145 feet be waterproofed and designed for hydrostatic pressures.

- Based on the subsurface explorations, undocumented fill up to 21 feet was observed at the site and appears to extend near the groundwater. This fill is not considered suitable for structural support.

- Due to the compressibility of the upper alluvial soils (static settlement) and the potential for seismically-induced settlement and lateral spreading due to liquefaction, conventional shallow foundations supported on the alluvial soils or engineered fill are not recommended. Several options are available for foundation support. The decision as to which option(s) to select will likely be dictated at least partially by economics, and should be made by the owner in consultation with the design team once the design-level geotechnical study is complete. Options include ground improvement, such as Stone Columns or Deep Soil Mixing, or a deep foundation system, such as driven piles, with a structurally supported slab.

- Depending on the location and depth of the earthwork at the site, wet soils should be anticipated and significant processing of these materials will likely be required (moisture reduction) prior to placement as engineered fill. Also, additional overexcavation and recompaction or replacement and/or cement treatment may be necessary to stabilize the bottom of deep excavations where wet soils are encountered.

1.0 INTRODUCTION

This report presents the results of our geotechnical feasibility study the proposed Anaheim Regional Transportation Intermodal Center (ARTIC) Phase 1 project. Kleinfelder understands that the Orange County Transportation Authority (OCTA) and the City of Anaheim plan to develop a major transit facility, known as ARTIC. The proposed facility will serve Metrolink, Amtrak, fixed-route buses, and will be a regional terminal for the future California High Speed Train. The ARTIC Phase I project is approximately bounded by Katella Avenue to the north, the Santa Ana River to the east and by the Anaheim Stadium to the south. This study was concentrated on the east side of Douglass Road and north of the railroad, where the main ARTIC building will be situated. The project boundaries are shown on Plate 1, Site Vicinity map.

Preliminary recommendations for improvements within the Caltrans right-of-way were presented in a Preliminary Foundation Report, dated July 8, 2009 (Kleinfelder, 2009b). Preliminary recommendations for the remaining improvements, such as the lowering and widening of Douglass Road, pedestrian railroad crossings, and retaining structures, were presented in a Preliminary Foundation Report, dated July 17, 2009 (Kleinfelder, 2009c).

The purpose of this feasibility study is to evaluate the subsurface soil conditions at the site in order to provide preliminary geotechnical conclusions for project feasibility to support the project's Environmental Documents. This feasibility study is not intended to be a design-level geotechnical study, and additional field and laboratory testing will be required in order to finalize the geotechnical recommendations for the design and construction.

The scope of our services was presented in our document titled, "Revised Contract Amendment Request, Additional Geotechnical and Environmental Services, Proposed ARTIC – Phase 1, Anaheim, California", dated September 3, 2009 (Document 103567/IRV9P123). This report summarizes the data collected and presents our preliminary findings, conclusions, and recommendations for design and construction for project feasibility.

1.1 PROJECT DESCRIPTION

Kleinfelder understands that the main ARTIC facility will consist of a transit center building (approximately 220 by 300 feet in plan) located at the south end of the site near the tracks. The transit center building will be underlain by a one- or two- level subterranean parking structure, which will extend north beyond the building limits. The remaining improvements will consist mainly of surface parking and driveways with some landscape areas.

1.2 SCOPE OF SERVICES

The scope of our geotechnical feasibility study consisted of a literature review, subsurface explorations, geotechnical laboratory testing, engineering evaluation and analysis, and preparation of this report. A description of our scope of services performed for the geotechnical portion of the project follows.

Our report includes a description of the work performed, a discussion of the geotechnical conditions observed at the site, and preliminary recommendations developed from our engineering analysis of field and laboratory data. The recommendations contained within this report are subject to the limitations presented in Section 6. An information sheet prepared by ASFE (the Association of Engineering Firms Practicing in the Geosciences) is also included. We recommend that all individuals using this report read the limitations (Section 6.0) along with the attached ASFE document.

Task 1 – Background Data Review. We reviewed readily-available published and unpublished geologic literature in our files and the files of public agencies, including selected publications prepared by the California Geological Survey (formerly known as the California Division of Mines and Geology) and the U.S. Geological Survey. We also reviewed readily available seismic and faulting information, including data for designated earthquake fault zones as well as our in-house database of faulting in the general site vicinity. References used are listed in Section 7.0 (References) of this report.

Task 2 – Field Exploration. The subsurface conditions at the site were recently explored by Kleinfelder by drilling 5 borings, installing 2 groundwater monitoring wells, and advancing 7 Cone Penetration Tests (CPTs). The borings/wells were drilled to depths between approximately 51½ and 101½ feet below the existing ground surface (bgs) using truck-mounted, hollow-stem drilling equipment. The CPTs were advanced to depths between approximately 38 and 94 feet bgs. The approximate locations of the borings and CPTs are presented on Plate 2, Field Exploration Map.

Prior to commencement of the fieldwork, various geophysical techniques were used at each boring and CPT location in order to identify potential conflicts with subsurface structures. Each of our proposed field exploration locations were also cleared for buried utilities through Underground Service Alert (USA). A Kleinfelder engineer supervised the field operations and logged the borings. Selected bulk and drive samples were retrieved, sealed and transported to our laboratory for further evaluation. The number of blows necessary to drive both Standard Penetration Test (SPT) and modified California-type samplers were recorded. A description of the field exploration and the logs of the borings, including a Legend to the Logs of Borings, are presented in Appendix A. Logs of the CPTs are also presented in Appendix A.

Task 3 – Laboratory Testing. Laboratory testing was performed on representative bulk and relatively undisturbed samples to substantiate field classifications and to provide engineering parameters for geotechnical design. Laboratory testing consisted of in-situ moisture content and dry unit weight, wash sieve (% passing #200 sieve), Atterberg limits, consolidation, gradation, direct shear and preliminary corrosion potential analyses. A summary of the testing performed and the results are presented in Appendix B.

Task 4 – Geotechnical Analyses. The available field and laboratory data were analyzed in conjunction with assumed finished grades and structural loads to provide preliminary geotechnical conclusions for project feasibility and cost estimating purposes. Geotechnical considerations included an evaluation of feasible foundation systems including constructability and compatibility constraints and earthwork. Potential geologic hazards, such as ground shaking, liquefaction potential, slope stability, flood hazard, fault rupture hazard and seismically-induced settlement, were also evaluated.

Task 5 – Report Preparation. This report summarizes the work performed, data acquired, and our preliminary geotechnical findings and conclusions for project feasibility to support the project's Environmental Documents. Our report includes the following items:

- Site Vicinity Map, and Field Exploration Map showing the approximate field exploration locations;
- Logs of borings and CPTs, including approximate elevations;
- Results of laboratory tests;
- Discussion of general site conditions;
- Discussion of general subsurface conditions as encountered in our field exploration, including the depth to groundwater;
- Discussion of regional and local geology and site seismicity;
- Discussion of geologic and seismic hazards;
- Preliminary evaluation of the liquefaction potential, dynamic settlement, and lateral spreading;
- Preliminary recommendations for grading, temporary construction shoring, and earthwork, which could significantly impact cost;
- Discussion of feasible foundation systems, including preliminary design recommendations and ground improvement alternatives;
- Preliminary recommendations for support of floor slabs and slab-on-grade;
- Preliminary recommendations for seismic design parameters in accordance with the 2007 California Building Code; and
- Preliminary evaluation of the corrosion potential of the on-site soils.

2.0 SITE CONDITIONS

2.1 SITE DESCRIPTION

The ARTIC project site includes approximately 13.5 acres of land located north of the existing LOSSAN corridor, and extending westward from the Santa Ana River to, and including, Douglas Road. This area is about 1400 feet long and 300 to 550 feet wide. Douglass Road is currently a 4-lane road, which crosses under the LOSSAN railroad corridor and SR-57 bridges. The remainder of the project site contains several single-story office and maintenance buildings and work shelters, including an area to wash vehicles. Except for a narrow landscape area along Douglass Road and some trees across from the main office building, most of the site outside the buildings is asphalt paved with localized concrete flatwork. The site ranges in elevation between approximately 165 feet in the northeast corner near Katella Avenue to 156 feet (NAVD 88) at the southern end near the LOSSAN corridor. Current surface elevations of Douglass Road are approximately 165 feet near Katella Avenue dropping to about 146 feet beneath the LOSSAN corridor bridge. The LOSSAN railroad corridor rises about 10 feet above the site at an elevation of approximately 166 feet. The Santa Ana River bounds the site to the east and is separated from the site by an improved embankment (levee or berm), which rises to an elevation of about 165 to 168 feet. The levee crest is paved and currently used as an Orange County bike path and maintenance access to the river. The river bottom elevation is estimated to be approximately 140 to 145 feet.

The site includes underground utilities such as sewer, water, storm drain, electric and communication lines. The main power lines are overhead. The site is fenced and the southern two-thirds is currently used as contractor maintenance and storage yards and includes an office trailer. The northern portion of the site is presently used as parking.

2.2 SITE HISTORY

Historical aerial photography (see Section 7.0 for a complete list) and vintage topographic maps (Plate II of Mendenhall, 1905) show that the project site and general vicinity was largely undeveloped or minimally developed agricultural land in the early 1900s. Although it appears that levee construction along the Santa Ana River had begun by the 1920s, the river's west bank adjacent to the project site was still in a natural condition and bank erosion and sloughing was apparent. In 1938, a year of heavy rains and extensive flooding throughout southern California, the site was stripped of all vegetation. In 1939, on the project site's western boundary, diagonal levees or berm-like structures (denoted as "1939 Levee" on Plate 3) are observed north and south of the railroad tracks (i.e., LOSSAN railroad corridor). The 1939 Levee is approximately 50 feet wide and appears to restrict the bank sloughing to its river-side, thus protecting orchards to the west. Collins Avenue crosses the river from the east, bisecting the site and turns northward to join a road that would become the present-day South Douglass Road. Between 1955 and 1959 quarry excavation activities had begun on the project site between the railroad tracks and Collins Avenue-Douglass Road alignment. The quarry is open towards the Santa Ana River and its bottom appears to be slightly below river's bottom. The approximate extent of the quarry is shown on Plate 3. Also, during this time, bank erosion and sloughing of the project site, north of Collins Avenue, had migrated westward to the 1939 Levee. The Collins Avenue-Douglass Road alignment and the railroad tracks were largely unaffected by the quarry excavation or the sloughing of the river bank. By the late 1960s and early 1970s the Santa Ana River's current levee system has been constructed between the river and the project site. The project site, behind the current river levee (including the quarry), has been filled and, by the mid-1970s, the site has been developed with the current alignment of South Douglass Road. By the late 1970s, the SR-57 has been completed.

3.0 GEOLOGY

3.1 REGIONAL GEOLOGIC SETTING

The ARTIC site is located in the southern part of the Los Angeles Basin within the Peninsular Ranges geomorphic province. The Peninsular Ranges geomorphic province is characterized by elongate northwest-trending mountain ranges separated by sediment-floored valleys (California Geological Survey, 2002). The most dominant structural features of the province are the northwest trending fault zones, most of which die out, merge with, or are terminated by the steep reverse faults at the southern margin of the Transverse Ranges geomorphic province.

East of the site are the northwest-trending Santa Ana Mountains, a large range which has been uplifted on its eastern side along the Whittier-Elsinore Fault Zone, producing a tilted, irregular highland that slopes westward toward the sea (Schoellhamer et al., 1981). The area south and west of the Santa Ana Mountains is generally characterized as a broad, complex, alluvial fan, which receives sediments from the Santa Ana River and its tributaries draining the Santa Ana Mountains and Puente Hills. These sediments are relatively flat-lying, unconsolidated to loosely consolidated clastic deposits that are approximately 1,700 feet thick beneath the site (Metropolitan Water District of Southern California, 2007; and Orange County Water District, 2004).

3.2 SITE GEOLOGY

The ARTIC site is located adjacent to the Santa Ana River, a braided stream system with flood control measures. The surficial deposits in the vicinity of the project area consist of alluvial fan material and alluvium deposited by the Santa Ana River over the last few thousand years. These unconsolidated alluvial sediments are generally composed of flat-lying, non-marine deposits of sand and a minor amount of silt. (Morton et al., 2004). These sandy deposits become interbedded with clayey layers in the subsurface, generally at a depth of approximately 50 to 55 feet. However, due to quarrying activities and bank sloughing, most of the project site does not have alluvium at the surface, but rather an undetermined thickness of undocumented artificial fill. The site was apparently filled in to current grades during development of the property in the early 1970s.

3.3 SUBSURFACE CONDITIONS

The subsurface conditions encountered in our borings and CPTs at the site generally consist of artificial fill underlain by young alluvium. A discussion of the subsurface materials encountered is presented in the following sections. Detailed descriptions of the deposits are provided in our logs of borings and CPTs presented in Appendix A.

3.3.1 Undocumented Fill

Undocumented fill soils associated with the raising of the site were encountered in the borings recently drilled. Locally derived sand material appears to have been used as fill and compaction appears to be highly variable. This fill is considered undocumented and not suitable for structural support. The fill depth varies throughout the site and is difficult to determine due to the nature of the material. Based on our interpretation of the materials encountered, the fill depths range between about 7 and 21 feet in the vicinity of our borings. It should be noted that deeper fill may be present at other locations not explored.

The fill soils were classified mostly as poorly graded sand, poorly graded sand with silt and silty sand. The moisture contents were generally in the range of 2 to 14 percent (average about 5½ percent). The dry unit weights range between 106 and 123 pcf (average about 114 pcf).

3.3.2 Young Alluvium

Young alluvial deposits were encountered below the fill. The alluvium consists predominantly of interbedded layers and lenses of poorly graded sand, silty sand, lean clay and sandy silt. Based on the borings, the upper 10 feet of alluvium immediately below the fill consists generally of poorly graded sand (SP and SP-SM) and silty sand (SM). Groundwater appears to be perching on silt and clay soil layers. The shallowest clay layer was encountered in Boring B-2 at about 20½ feet. Gravel layers, generally ranging in thickness between about 2 and 8 feet, were identified in Borings B-1, B-2, B-4, W-1 and W-2. Borings B-3 and B-5 have sand layers containing significant amount of gravel. Generally gravel was first detected in the borings at depths between 28 and 40 feet.

With few exceptions, Kleinfelder's laboratory test data indicated moisture content of the silt and clay in the range of 14 to 27 percent (average of about 21 percent) and dry unit weights in the range of 99 to 113 pcf (average 105 pcf). For the sand and gravel materials the laboratory moistures are generally in the range of 2 to 17 percent (average of about 8 percent) and dry unit weights of 92 to 138 pcf (average of about 115 pcf). Based on field observation during sampling and blow counts recorded, the clay and silt soils are generally medium stiff or stiff, with localized soft, very stiff and hard layers. The sandy soils are generally loose to medium dense and the gravel are dense to very dense. The sand with gravel ranges from medium dense to very dense.

3.4 GROUNDWATER

The ARTIC site is located in the forebay area of Orange County Basin (Metropolitan Water District of Southern California, 2007; DWR, 2004; and OCWD, 2004). The forebay is an area consisting of coarser, interconnected deposits that allows surface water to percolate down and ultimately recharge the County's principal aquifer about 800 feet deep (DWR, 2004). The nearest aquifer beneath the site is the Talbert aquifer and it extends to a depth of approximately 150 feet below the project area (Poland, 1956). Near the site, groundwater levels in the Talbert aquifer can fluctuate substantially depending on rainfall conditions or recharge activities in the river. In 1994, wet soil samples (indication of groundwater) were logged adjacent to the site and the LOSSAN railroad corridor at a depth of approximately 50 feet (SCRRA, 1994), and in 1999 groundwater was measured at a depth of about 34 feet near the intersection of Katella Avenue and South Douglass Road (Coleman Geotechnical, 1999). In June 2006, OCWD mapped groundwater levels near the site at a depth of approximately 60 feet. However, in 2001 an evaluation of the historically shallowest groundwater levels was conducted by the CGS (Greenwood and Pridmore, 2001) for the area which included the site. They determined the highest historical groundwater to be approximately 20 feet deep for the project site.

The groundwater encountered during Kleinfelder's field exploration appears to be perched. The zones of groundwater seepage observed are presented in Table 1. It should be noted that Kleinfelder's groundwater measurements were taken during a relatively long dry period and mostly likely are not representative of the groundwater conditions during the rainy season.

Table 1
Groundwater Level Measurements

Boring No.	Location	Approximate Groundwater Depth (feet)	Approximate Groundwater Elevation (feet)	Date Measured
B-1	NE Site Portion	51	110	9/24/09
B-2	Center of Site	83	75	9/24/09
B-3	SE Corner	58	98	9/22/09
B-4	S End of Site	87	71	9/23/09
W-1	E Center of Site	23	134	10/16/09
W-2	SE Corner	50 *	--	10/16/09

Note: * Groundwater was not encountered in the well.

Fluctuations of the groundwater level, localized zones of perched water, and increased soil moisture content should be anticipated during and following the rainy season. Irrigation of landscaped areas on or adjacent to the site can also cause a fluctuation of local groundwater levels.

3.5 FAULTING

Primary ground rupture is ground deformation that occurs along the surface trace of the causative fault during an earthquake. No known active faults are mapped crossing the site, and the site is not located within a State of California, Alquist-Priolo Earthquake Fault Zone (Bryant and Hart, 2007), thus the potential for future surface fault rupture at the site is considered to be low. The closest mapped faults to the site include the Peralta-El Modeno, Puente Hill Blind Thrust, Whittier-Elsinore faults and several unnamed and buried faults to the south of the site. Table 2 summarizes the distances of the closest known faults.

Table 2
Summary of Closest Mapped Faults

Fault Name	Type	Distance, miles (km)	Magnitude, M_w
El Modeno	Reverse	2.3 (3.7)	6.5
Peralta	Reverse	3.6 (5.9)	6.5
Unnamed Buried (2)	Unknown	4.2 (6.7) and 4.9 (8.0)	Unknown
Puente Hills	Blind Thrust	5.3 (8.6)	7.1
Whittier	Strike Slip	8.5 (13.8)	6.8

The Peralta-El Modeno faults are located north and northeast of the project site. The Peralta fault outcrops along the southern edge of the Peralta Hills east of the Santa Ana River approximately 3.6 miles (5.9 kilometers) from the site (Morton et al., 2004). The Peralta fault is a reverse fault which dips north towards the Whittier fault and movement along it results in crustal shortening and uplift of the Peralta Hills (Dolan et al., 2001). The El Modeno fault could be a westward extension of the Peralta fault, but this is currently not known. The El Modeno fault is buried beneath the alluvium of the Santa Ana River and it's inferred location is about 2.3 miles (3.7 kilometers) north of the site. The CGS fault map by Jennings (1994) shows the buried El Modeno fault extending westward from Burrel Ridge to about the SR-57 freeway. Slip rates of the El Modeno and Peralta faults are not currently known; however the faults are considered potentially active capable of generating an M_w6.5 earthquake (Mualchin, 1996).

The Puente Hills Blind Thrust fault (M_w7.1 earthquake) passes approximately 5.3 miles (8.6 kilometers) from the site. This active fault consists of three segments, from west to east; the Los Angeles, Santa Fe Springs and the Coyote Hills segments. These segments shallowly dip northward toward the Puente Hills and thrusting motion along these faults has resulted in crustal shortening in the region. Slip on the three segments produced an anticlinal structure caused by the compression and folding. This has been observed in the Coyote Hills segment approximately 5.5 miles (9.1 kilometers) north-northwest of the site. Although the Puente Hills Blind Thrust is buried approximately 2 to 3 kilometers beneath the ground surface, significant seismic shaking can result from this buried fault. Displacement along a section of the Santa Fe Springs segment is believed to have caused the 1987 Whittier Narrows earthquake (M_w6.0), confirming the potential for this active fault system to cause significant seismic shaking in the Los Angeles Basin (Dolan et al., 2001; Shaw et al, 2002).

The Whittier fault is an extension of the Elsinore fault where the fault deviates from the normal northwesterly strike and turns more westward at the Santa Ana River (Morton et al., 2004). Movement along the Whittier Fault is predominantly right-lateral strike-slip at a rate of approximately 2 to 3 mm/year (Dolan et al., 2001). However, it is believed to have had some reverse movement historically causing uplift of the Puente Hills at about 0.5 mm/year (Dolan et al., 2001). The surface trace of the Whittier fault has been mapped by the State and designated as an Alquist-Priolo Earthquake Fault Zone (Bryant and Hart, 2007). The surface trace has been mapped approximately 8.5 miles (13.8 kilometers) north of the project site.

Two unnamed, buried faults are mapped to the southwest and south of the site, approximately 4.2 miles (6.7 kilometers) and 4.9 miles (8 kilometers), respectively. Both faults terminate within the Orange County Basin, however, the one to the south, is mapped trending towards the site before it ends about 4.9 miles away. No information regarding these faults is available except that they are buried beneath sediments, some older than 11,000 years (Morton et al., 2004).

3.6 OTHER GEOLOGIC HAZARDS

3.6.1 Flooding and Inundation

Flooding and inundation occurs as a result of several factors in developed areas. These factors include: rainfall rates that exceed an area's ability to absorb or control the runoff; impounded water retained behind a flood control structure (upstream-inundation); failure of a flood control structure (downstream-inundation); seiches and tsunamis (earthquake induced). Flooding of the Santa Ana River has inundated the site numerous times over the past 175 years. Channelization and flood protection levees were constructed, and following the devastating 1938 flood, Prado Dam was constructed to improve flood protection. As development of the inland empire proceeded, additional measures were soon needed. Currently, flood protection for the area is being improved with the Santa Ana River Mainstream Project. The project will increase the flood level protection along more than 75 miles of the Santa Ana River course within Orange, Riverside and San Bernardino Counties, and is scheduled to be completed by 2010.

Although the Santa Ana River Mainstream Project may reduce the risk of flood along the river, it may not prevent flood inundation at the site due to failure of the Prado Dam during an earthquake. An earthquake along the Chino Hills fault, which crosses beneath the dam near the spillway, could cause the dam to fail. A catastrophic failure of the dam with substantial water stored behind it could cause flooding at the site downstream. A flood inundation evaluation should be performed for the site during the design phase.

3.6.2 Liquefaction

Liquefaction occurs when loose, coarse-grained or silty soils are subjected to strong shaking resulting from earthquake motions. The coarse-grained or silty soils typically lose a portion or all of their shear strength, and regain strength sometime after the shaking stops. Soil movements (both vertical and lateral) have been observed under these conditions due to consolidation of the liquefied soils. The site is located within a State of California Hazard Zone for Liquefaction (CDMG, 1998). Because of the depth of historic groundwater and the soil types encountered during our investigation, the potential for liquefaction at the site is moderate to high. A more detailed description of the liquefaction analyses is provided in Section 4.2.2.

3.6.3 Lateral Spreading and Slope Stability (Santa Ana River Channel)

Lateral spreading is the term commonly used to describe the permanent deformation of sloping ground that occurs during earthquake shaking as a result of soil liquefaction. Deformations can range from inches to several feet, with the greatest displacements usually occurring near free-faces. Therefore, facilities and structures adjacent to bodies of water (e.g. ports/harbors, lakes, and rivers) are usually at the greatest risk of experiencing damage due to lateral spreading.

The portion of the site bound by the Santa Ana River has potential to be affected by slope instability and lateral spreading due to liquefaction. The top of the embankment to the channel bottom is approximately 15 to 20 feet high with an inclination of approximately 2:1 (horizontal to vertical). Preliminary analyses indicate that, due to liquefaction, the channel slope will not be stable during the design earthquake and may affect the site improvements. A detailed evaluation of the stability of the Santa Ana channel slope should be performed during the design-level geotechnical study in order to design mitigative measures to protect the site improvements.

3.6.4 Expansive Soils

The upper fill and alluvial soils are generally granular and non-cohesive in nature (sandy soil). Accordingly, the potential for expansive soils impacting the project at shallow depth is low. Subterranean parking excavations may encounter clayey soils with a medium expansion potential.

3.6.5 Subsidence

The site is not located in an area of known ground subsidence due to the withdrawal of subsurface fluids. Accordingly, the potential for subsidence occurring at the site due to the withdrawal of oil, gas, or water is considered remote.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 GENERAL

Based on the results of our field explorations performed to date, laboratory testing and geotechnical analyses conducted during this study, it is our professional opinion that the proposed project is geotechnically feasible, provided the recommendations presented in this feasibility study report and future design reports are incorporated into the project design and construction. The primary geotechnical constraints that will have a significant impact to the cost of developing the site include: 1) the compressibility of the upper alluvial soils (static settlement); 2) the potential for seismically-induced settlement and slope instability/lateral spreading due to liquefaction; 3) the presence of deep undocumented fill; and 4) the potential for shallow groundwater adversely affecting the design and construction of subterranean parking levels. Further discussion of these constraints is presented in the following sections.

The following opinions, conclusions, and preliminary recommendations are based on the properties of the materials encountered in the borings and CPTs, the results of the laboratory-testing program, and our engineering analyses performed. Our preliminary conclusions regarding the geotechnical aspects of the design and construction of the project are presented in the following sections. Any substantial changes in grades or to the proposed improvements may require a change to our preliminary recommendations.

4.2 SEISMIC DESIGN CONSIDERATIONS

The site is located in a seismically active region and the proposed development can be expected to be subjected to moderate to strong seismic shaking during its design life. The following sections discuss seismic design considerations with respect to the project site.

4.2.1 2007 CBC Seismic Design Parameters

According to the 2007 California Building Code (CBC), every structure, and portion thereof, including non-structural components that are permanently attached to structures and their supports and attachments, shall be designed and constructed to resist the effects of earthquake motions in accordance with ASCE 7-05 (ASCE, 2006), excluding Chapter 14 and Appendix 11A. The seismic design category for a structure may be determined in accordance with Section 1613 of the 2007 CBC or ASCE 7-05. According to the 2007 CBC, sites subject to liquefaction should be classified as Site Class F, which requires a site response analysis. However, ACSE7-05, which is the basis for the 2007 CBC, suggests that for a short period (less than ½ second) structure on liquefiable soils, Site Class D or E may be used instead of Site Class F to estimate design seismic loading on the structure. The selection of Site Class D or E is based on the assessment of the site soil profile assuming no liquefaction. The project structural engineer should determine if a site-specific response analysis is required during the design phase for the structural design. The 2007 CBC Seismic Design Parameters, assuming a Site Class D, are summarized in Table 3.

Table 3
2007 CBC Seismic Design Parameters

S_s (Figure 1613.5(3)) (g)	1.38
S_1 (Figure 1613.5(4)) (g)	0.50
F_a (Table 1613.5.3(1))	1.0
F_v (Table 1613.5.3(2))	1.5
S_{MS} (Equation 16-37) (g)	1.38
S_{M1} (Equation 16-38) (g)	0.75
S_{DS} (Equation 16-39) (g)	0.92
S_{D1} (Equation 16-40) (g)	0.50

4.2.2 Liquefaction and Seismic Settlement

The term liquefaction describes a phenomenon in which saturated, cohesionless soils temporarily lose shear strength (liquefy) due to increased pore water pressures induced by strong, cyclic ground motions during an earthquake. Structures founded on or above potentially liquefiable soils may experience bearing capacity failures due to the temporary loss of foundation support, vertical settlements (both total and differential), and undergo lateral spreading. The factors known to influence liquefaction potential include soil type, relative density, grain size, confining pressure, depth to groundwater, and the intensity and duration of the seismic ground shaking. The cohesionless soils most susceptible to liquefaction are loose, saturated sands and some silts.

To assess the potential for liquefaction of subsurface soils at the site, we used the simplified liquefaction analysis procedure recommended by NCEER (Youd and Idriss, 1997, 2001). For estimating the resulting ground settlements, we used the method proposed by Tokimatsu and Seed (1987). This method utilizes the standard penetration test (SPT) blow count data to estimate the amount of volumetric compaction or settlement during an earthquake.

According to the State of California (Greenwood and Pridmore, 2001), the historical high depth to groundwater beneath the site has been mapped at a depth of 20 feet below original ground surface. Following our subsurface explorations, groundwater was measured at a depth of 23 feet (Elevation 134 feet) in one of our monitoring wells (Well W-1). A groundwater level of 20 feet below the existing ground surface was used in our preliminary analyses.

According to Section 1802 of the 2007 CBC, the PGA used in the liquefaction analysis may be estimated by dividing the S_{DS} by 2.5. A PGA of 0.37g with an associated Magnitude 6.8 earthquake was used as the design-level seismic event for our liquefaction analyses.

We evaluated the liquefaction potential at the site using the SPT data. The CPTs were used to refine the soil profile of the borings because they provide a continuous measurement of the site stratigraphy. Based on the SPT data and our engineering analyses, it is our opinion that the loose to medium dense sandy silt, silty sand, and sand below the design level groundwater are subject to liquefaction in the event of a major earthquake occurring on a nearby fault. Based on our preliminary analyses, we estimate that seismically-induced settlement of saturated sandy soils due to strong ground shaking during a design-level seismic event could be on the order of 3 to 6 inches. Because of variations in distribution, density, and confining conditions of the soils, seismic settlement is generally non-uniform and severe structural damage can occur due to differential settlement. The amount of differential settlement will depend on the uniformity of the subsurface profile. For relatively uniform subsurface conditions, differential settlement on the order of 50 percent of the total seismic settlement could be expected. For highly heterogeneous sites, differential settlements on the order of 75 to 100 percent of the total seismic settlement could be expected. Differential settlement at this site is expected to be on the order of 2 to 4 inches.

4.3 PRELIMINARY DESIGN GROUNDWATER ELEVATION

As discussed above, groundwater encountered during Kleinfelder's field exploration appears to be perched. Groundwater was measured at a depth of 23 feet (Elevation 134 feet) in one of our monitoring wells (Well W-1). According to the State of California (Greenwood and Pridmore, 2001), the highest historical groundwater depth at the site has been mapped at about 20 feet below grade.

The long-term performance of the subterranean parking slab and subterranean walls will be affected by the water level if not considered in the design. Due to the potential for an increased groundwater elevation from rainfall, over-irrigation, and the proximity to the Santa Ana River, we recommend that a preliminary design groundwater elevation of 145 feet, which roughly corresponds to the adjacent river bottom, be used. We recommend that all subterranean walls and floor slabs that extend to and below Elevation 145 feet be waterproofed and designed for hydrostatic pressures. Waterproofing above this elevation may be required to prevent moisture migration through the walls.

4.4 FOUNDATIONS

The preliminary geotechnical design recommendations presented below are for project feasibility and budget-level cost estimating. These preliminary recommendations may be modified once the improvement configuration, design grades and structural loading have been finalized and after the design-level geotechnical study is completed.

As discussed above, the primary geotechnical constraints for site development are the compressibility of the upper alluvial soils (static settlement) and the potential for seismically-induced settlement and lateral spreading due to liquefaction. Several options are available for foundation support. The decision as to which option(s) to select will likely be dictated at least partially by economics, and should be made by the owner in consultation with the design team once the design-level geotechnical study is complete. Options include ground improvement, such as Stone Columns or Deep Soil

Mixing, or a deep foundation system, such as driven piles, with a structurally supported slab. Further discussion of these options is presented below.

In addition, based on our preliminary analyses, we cannot preclude the potential for lateral spreading of the Santa Ana River channel slope. Seismic deformation of the channel slope adjacent to the proposed building may need to be mitigated with ground improvement.

4.4.1 Ground Improvement

One alternative to mitigate static settlement and the potential for liquefaction at the site is to implement a properly designed ground improvement program. Once ground improvement is performed, the proposed building may be supported on a conventional shallow foundation system. Based on past experience, stone columns (vibro-replacement) or Deep Soil Mixing (DSM) may be cost effective ground improvement options.

The ground improvement program should be designed to limit total settlement (static and seismic) within tolerable levels, typically approximately ½ to 1 inch static settlement and 1 inch seismic settlement and differential settlement (static and seismic) to about ½ inch over 50 feet. At a minimum, the soils should be improved a horizontal distance of at least 15 feet beyond the edge of the building pad. Additionally, the ground improvement program should consider the impact to the surrounding roads and underground utilities.

The actual design of a ground improvement program should be performed by a design-build contractor specializing and experienced with these ground improvement methods. The contractor should provide material requirements, preliminary spacing and replacement ratios, and other design information. The ground improvement design will likely be an iterative process between the ground improvement contractor and the Geotechnical Engineer. It should be noted that ground improvement programs are typically design-build projects, and the specialty contractors are ultimately responsible for the performance of their designs. A more detailed discussion of two potential ground improvement options follows.

Stone Columns

Stone columns are formed by vibro-replacement. With vibro-replacement, a probe is advanced into the ground by means of vibration to the design treatment depth. The probe is then lifted several feet, and gravel is fed into the resulting void under pressure through a delivery tube attached to the probe. The vibrating probe is then advanced back into the deposited gravel, displacing it and compacting it. The probe is lifted and lowered again and again until a densified “stone column” is constructed to the ground surface. Ground improvement is achieved by the formation of these “stone columns” within the ground and by densifying the soil adjacent to the stone columns. The stiffer stone column matrix also helps to redistribute the shear stresses in the soil. Past experience and research have indicated that stone columns have the potential to additionally provide drainage. The inclusion of drainage assists in relieving excess pore pressures generated during an earthquake, and reducing the extent of liquefaction. Based on our experience and discussions with leading stone column installation experts, stone columns are very effective in sands and can be quite effective in silty sands and silts.

Deep Soil Mixing (DSM)

DSM is the mechanical blending of the in-situ soil with cementicious materials using a hollow auger and paddle arrangement. Soil-mixing rigs may have a single auger (about 2 to 12 feet in diameter) or several smaller-diameter augers (usually 2 to 8 augers). As the augers are advanced into the soil, grout is pumped through the stems and injected into the soil at the tips. After the design depth has been reached, the augers are withdrawn while the mixing process continues. The soil-mixing process results in a fairly uniform soil-cement column. The intent of a DSM program is to achieve increased shear strength and reduced compressibility of the soil. The DSM solidifies “columns” of soil in the treated area and the resulting soil-cement matrix helps to redistribute the shear stresses in the soil, thus, reducing the settlement of the ground surface due to liquefaction of the untreated soil. In addition, the soil-cement columns can be used as a load-bearing element to reduce static settlement.

4.4.2 Deep Foundations

As an alternative to ground improvement, the structures could be supported on deep foundation systems, such as driven piles, with structurally supported slabs (suspended slab). A properly-designed pile foundation system and structural slab would mitigate static and dynamic settlements, but would need to extend well below the depth of liquefaction due to the downdrag loads caused by the seismically-induced settlement. Deep foundations consisting of 16-inch-square precast prestressed concrete driven piles could be used at the site.

It should be noted that driven piles may encounter hard driving conditions and have difficulty penetrating interbedded dense gravel layers near the existing pile tip elevations. Pre-drilling of these dense gravel layers may be required. In addition, a vibration study should be conducted prior to final design to determine if vibrations from driving piles will have an adverse affect on existing structures. If driven piles are selected, the designer should evaluate the pile drivability and vibration concerns.

As an alternative to driven piles, Tubex Grout Injection (TGI) piles could be used at the site. A TGI pile consists of a pipe casing with an oversized drill tip that is drilled into the ground to the desired depth. The steel casing could be spliced similar to a steel H-pile. Once the pile reaches the tip elevation, grout is injected between the steel casing and the soil column, filling the void left by the oversized tip. The inside of the steel casing is then drilled out and reinforcing steel and concrete is placed. Downdrag loads can be reduced by filling a portion of the outside of the casing with bentonite.

The pile length will be significantly affected by the depth of liquefaction. Based on the available data and our preliminary analysis, liquefaction to a depth of approximately 60 feet may occur and possibly induce downdrag loads to a depth of about 55 feet. It should be noted that there are several thin soil layers below approximately Elevation 100 feet (below a depth of 60 feet) that could potentially be susceptible to liquefaction; however, the existing data was not sufficient to positively determine that liquefaction was of concern in these layers. As a result, these recommendations may require revision during the final design phase once additional data is available. Preliminary pile tip elevations based on extrapolating the existing soil data and assuming liquefaction will occur to an approximate elevation of 100 feet are presented in Table 4. The preliminary capacities provided in Table 4 assume the piles will tip into dense gravelly sand or gravel at around Elevation 70 feet.

Table 4
Summary of Preliminary Axial Pile Capacities

Type of Pile	Preliminary Pile Tip Elevation (feet)	Allowable Capacity [1] (kips) Compression
16-ich-square PCPS Driven	70	200
Tubex Grout Injection pile	70	175 [2]
	60	250 [2]

Notes: [1] A one-third increase may be used when considering wind loads, but not seismic loads.
[2] The preliminary pile capacities do not consider a reduction in the downdrag loads due to filling the annulus around the casing with bentonite.

4.5 EXCAVATION CONSIDERATIONS

4.5.1 General

While the details of site excavation (i.e., depth and lateral extent) are not known at this time, the proposed excavation will require temporary shoring around the perimeter of the site during construction. Underpinning may also be required for adjacent improvements and potentially for any power poles or utilities affected by the planned excavations.

The actual shoring design should be provided by a registered civil engineer in the State of California experienced in the design and construction of shoring under similar conditions. Once the final excavation and shoring plans are complete, the plans and design should be reviewed by the Geotechnical Engineer for conformance with the design intent and geotechnical recommendations.

4.5.2 Dewatering

Due to the depth of the anticipated excavation, dewatering may be required during construction depending on when construction takes place. The owner or contractor should retain an experienced engineer for design of a dewatering system. The dewatering system should be installed by a contractor specializing in dewatering under similar soil conditions. It has been our experience that improperly designed or constructed dewatering systems can significantly impact project schedule and cost.

The dewatering system will likely consist of deep wells with localized well points. If sump pumping is used to remove accumulated surface water in trenches or excavations, the gravel filled trenches and sump pits should be lined with filter fabric to reduce the potential of pumping out fines. The County of Orange, Division of Environmental Health (OCDEH), will likely restrict the discharge of water removed from excavations. Water mostly likely will need to be treated to discharge it to either the storm drain or sewer systems.

4.5.3 Shoring

Conventional shoring consisting of closely-spaced soldier piles and wooden lagging is commonly used. Due to the potential depth of the proposed excavation, several rows of tie-back anchors may be needed. Tie-backs may be installed by using hollow-stem auger drilling equipment. The tendon (high strength steel bar or cable) would be inserted into the hollow stem, the anchor drilled to its full length, and grout pumped through the stem while retracting the auger.

For preliminary cost estimating purposes, the unit friction between the grout and the soil (ultimate bond stress) for post-grouted anchors may be assumed to be on the order of 3,000 psf. Only the resistance developed beyond the failure wedge should be used in resisting lateral loads. The minimum bonded length should not solely be based on the required anchor capacity; the global stability of the shored wall should also be checked. In addition, due to the reduced overburden and cover depth, the Santa Ana River channel slope will need to be considered in the tie-back anchor design.

For preliminary design, braced excavations (including those using tie-back anchors) should be designed to resist a uniform horizontal soil pressure of at least 24H (in psf), where H is the wall height (feet). Forty five percent of any areal surcharge adjacent to the shoring (including existing structures and soil stockpiles) may be assumed to act as a uniform horizontal pressure against the shoring. A uniform horizontal surcharge pressure of 120 psf should be used for tieback walls adjacent to vehicular traffic.

Plate 4 presents the recommended preliminary lateral earth pressures for temporary shoring.

The pressures presented on Plate 4 do not include hydrostatic pressures; it is assumed that any temporary shoring will not be subject to hydrostatic pressures because construction dewatering will remove water before it accumulates behind the wall. If shoring or soldier piles extend below the water table, the effects of groundwater should be accounted for in the design of shoring.

4.6 PERMANENT SUBTERRANEAN WALLS

We anticipate that the permanent restrained retaining walls for the subterranean parking level will predominantly be constructed directly against the temporary shoring. The walls should be properly waterproofed and should have drainage system extending to the elevation of about 145 feet to collect surface water. We have assumed that the remainder of the wall will be designed for full hydrostatic pressure. We recommend that permanent walls be designed for the preliminary lateral earth pressures presented on Plate 5.

4.7 EARTHWORK

4.7.1 General

The earthwork recommendations that follow are based on the evaluation of widely spaced borings and CPTs. As soil conditions can vary, sometimes significantly, across short distances, earthwork recommendations may need to be modified based on the results of the future design-level geotechnical study. The recommendations that follow provide our estimate of remedial grading based on the limited data available. Once the final proposed grades and building configurations are established and the design-level geotechnical study is complete, we can modify the remedial grading recommendations, as appropriate.

All site preparation and earthwork operations should be performed in accordance with applicable codes, safety regulations and other local, state or federal specifications. All references to maximum unit weights are established in accordance with the latest version of ASTM Standard Test Method D1557. Site preparation will vary depending on the foundation support selected.

- Structural Areas (Building Pads) supported on Piles with a Structural Slab: Any disturbed soil below the bottom of the floor slab should be overexcavated and replaced as engineered fill.

- Structural Areas (Building Pads) supported on Shallow Foundation on Improved Ground: After ground improvement is performed, the upper few feet of the existing soils will be disturbed and some remedial grading will be required. In addition, there may be bulking of the upper soils from the ground improvement process. We recommend that the improvement area be overexcavated to a depth of at least 3 feet below the pre-improved grade. Depending on the amount of disturbance, the overexcavation may have to be deepened. This overexcavation should extend the full width of the improved area and at least of 5 feet outside the building pad, whichever is greater, where possible.

- Non-Structural Areas: For non-structural areas, such as equipment pads, pavements, sidewalks and other flatwork, etc., we recommend that the existing soils be overexcavated a minimum of 30 inches below existing grade or finished subgrade, whichever is greater, and be replaced as engineered fill. Depending on the observed condition of the existing soils, deeper overexcavation may be required in some areas. The overexcavation should extend beyond the proposed improvements a horizontal distance of at least two feet.

4.7.2 Wet Soils and Subgrade Stabilization

Depending on the location and depth of the earthwork at the site, wet soils should be anticipated and significant processing of these materials will likely be required (moisture reduction) prior to placement as engineered fill. Processing may require ripping the material, discing to break up clumps, and blending to attain uniform moisture contents necessary for compaction. Also, additional overexcavation and recompaction or replacement and/or cement treatment may be necessary to stabilize the bottom of deep excavations. Processing of wet soils and subgrade stabilization should be accounted for in cost estimates.

4.7.3 Temporary Excavations

Temporary cut slopes may be sloped back at an inclination of no steeper than 1.5:1 (horizontal to vertical) in the existing site soils and newly placed fill. Where space for sloped embankments is not available, shoring will be necessary. Shoring and/or underpinning of existing improvements that are to remain may be required. Excavations within a 1.5:1 plane extending downward from a horizontal distance of 2 feet beyond the bottom outer edge of existing improvements should not be attempted without bracing and/or underpinning the footings. All applicable excavation safety requirements and regulations, including OSHA requirements, should be met.

4.8 SUBTERRANEAN PARKING SLAB-ON-GRADE

With the ground improvement option, we recommend that a reinforced concrete slab be used to support the slab loads on the subgrade. Because the anticipated foundation level may be below the design groundwater level, the effects of uplift by hydrostatic pressure will likely control the design of the slab-on-grade. A design groundwater elevation of 145 feet is recommended for uplift of the slab areas. A thickened slab or permanent tie-down anchors may be utilized to resist uplift pressures.

4.9 SOIL CORROSION

The corrosion potential of the on-site materials to steel and buried concrete was preliminarily evaluated. Laboratory testing was performed on three representative soil samples to evaluate pH, minimum resistivity, chloride and soluble sulfate content. The test results are presented in Table 5.

Table 5
Corrosion Test Results

Component Analyzed	Method	Unit	Boring No. And Depth (feet)		
			B-1 @ 4'	**B-4 @ 4.5'**	**B-4 @35'**
Sulfate (SO_4)	375.4/9038	Mg/kg	8	11	34
Chloride Cl	325.3/9253	Mg/kg	56	46	66
pH	9045C/150.1	pH Unit	8.3	7.8	6.9
Minimum Resistivity	120.1	Ω-cm	23,200	19,600	3,710

These tests are only an indicator of soil corrosivity for the samples tested. Other soils found on site may be more, less, or of a similar corrosive nature. Imported fill materials should be tested to confirm that their corrosion potential is not more severe than those noted.

Although Kleinfelder does not practice corrosion engineering, based on the minimum resistivity results from the soil tested, the near-surface site soils may be considered to be moderately corrosive towards buried ferrous metals. The concentrations of soluble sulfates indicate that the potential of sulfate attack on concrete in contact with the on-site soils is “negligible” based on ACI 318 Table 4.3.1 (ACI, 2004). Accordingly, a concrete mix with Type II cement may be used. Maximum water-cement ratios are not specified for these sulfate concentrations.

We recommend that a competent corrosion engineer be retained to evaluate the corrosion potential of the on-site soils to the proposed improvements, to recommend further testing as required, and to provide specific corrosion mitigation methods appropriate for the project, if desired.

5.0 ADDITIONAL SERVICES

This report presents conclusions and preliminary recommendation related to foundation type, earthwork, pavements and other pertinent topics for a feasibility study. A design-level geotechnical study will need to be performed to develop final recommendations for the proposed development.

The recommendations provided in this report are based on our understanding of the described project information and on our interpretation of the data. We have made our recommendations based on 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.

6.0 LIMITATIONS

This geotechnical feasibility study has been prepared for the exclusive use of Jones and Stokes, OCTA, and their agents for specific application to the proposed ARTIC Phase I project in support of the project's Environmental Documents. It may not contain sufficient information for other uses or purposes of other parties. It is not considered sufficient for final design or construction of the project. The findings, conclusions and recommendations presented in this report were prepared in accordance with generally accepted geotechnical engineering practice. No other warranty, express or implied, is made.

The scope of services was limited to the field exploration program described in Section 1.2. It should be recognized that definition and evaluation of subsurface conditions are difficult. Judgments leading to conclusions and recommendations are generally made with incomplete knowledge of the subsurface conditions present due to the limitations of data from field studies. The conclusions of this assessment are based on our field exploration, laboratory testing programs, and engineering analyses.

Kleinfelder offers various levels of investigative and engineering services to suit the varying needs of different clients. Although risk can never be eliminated, more detailed and extensive studies yield more information, which may help understand and manage the level of risk. Since detailed study and analysis involves greater expense, our clients participate in determining levels of service, which provide information for their purposes at acceptable levels of risk. The client and key members of the design team should discuss the issues covered in this report with Kleinfelder, so that the issues are understood and applied in a manner consistent with the owner's budget, tolerance of risk and expectations for future performance and maintenance.

Recommendations contained in this report are based on our field observations and subsurface explorations, limited laboratory tests, and our present knowledge of the proposed construction. It is possible that soil or groundwater conditions could vary between or beyond the points explored. If soil or groundwater conditions are encountered during construction that differ from those described herein, the client is responsible for ensuring that Kleinfelder is notified immediately so that we may reevaluate the recommendations of this report. If the scope of the proposed construction, including the estimated Traffic Index or locations of the improvements, changes from that described in this report, the conclusions and recommendations contained in this report are not considered valid until the changes are reviewed, and the conclusions of this report are modified or approved in writing, by Kleinfelder.

The scope of services for this geotechnical report did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous substances in the soil, surface water, or groundwater at this site.

This report, and any future addenda or reports regarding this site, may be made available to bidders to supply them with only the data contained in the report regarding subsurface conditions and laboratory test results at the point and time noted. Bidders may not rely on interpretations, opinion, recommendations, or conclusions contained in the report. Because of the limited nature of any subsurface study, the contractor may encounter conditions during construction which differ from those presented in this report. In such event, the contractor should promptly notify the owner so that Kleinfelder's Geotechnical Engineer can be contacted to confirm those conditions. We recommend the contractor describe the nature and extent of the differing conditions in writing and that the construction contract include provisions for dealing with differing conditions. Contingency funds should be reserved for potential problems during earthwork and foundation construction.

This report may be used only by the client and only for the purposes stated, within a reasonable time from its issuance, but in no event later than one year from the date of the report. Land use, site conditions (both on site and off site) or other factors may change over time, and additional work may be required with the passage of time. Any party, other than the client who wishes to use this report shall notify Kleinfelder of such intended use. Based on the intended use of this report and the nature of the new project, Kleinfelder may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the client or anyone else will release Kleinfelder from any liability resulting from the use of this report by any unauthorized party and the client agrees to defend, indemnify, and hold harmless Kleinfelder from any claims or liability associated with such unauthorized use or non-compliance.

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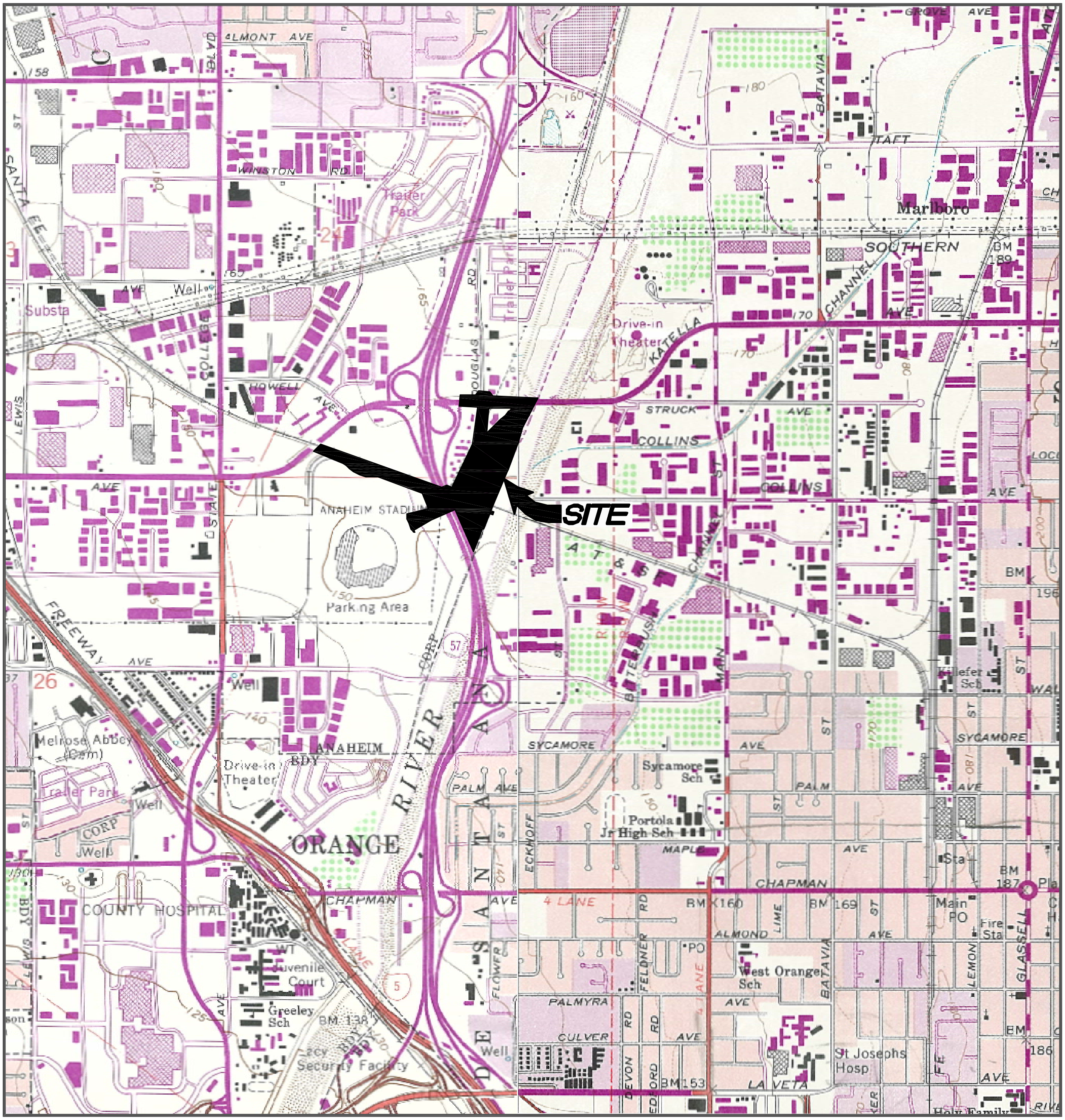
AERIAL PHOTOGRAPHS REVIEWED

Date	Type	Flight	Frames	Approximate Scale	Source
2-28-1929	B&W	C-287 #3	A1, A2; and B2, B3	1:18,000	Fairchild Aerial Collection
1931	B&W	C-1780	C-1	1:15,600	Fairchild Aerial Collection
3-4-1938	B&W	C-5029	66-68	1:32,000	Fairchild Aerial Collection
6-24-1939	B&W	C-5925	120-122	1:24,000	Fairchild Aerial Collection
6-17-1947	B&W	C-11351-7	54-56	1:24,000	Fairchild Aerial Collection
8-1947	B&W	C-113730A-11	155X-157X	1:7,200	Fairchild Aerial Collection
8-1947	B&W	C-113730A-12	102-104	1:7,200	Fairchild Aerial Collection
8-1947	B&W	C-113730A-14	4-6	1:7,200	Fairchild Aerial Collection
8-31-1947	B&W	C-113730D-14	48-50	1:14,400	Fairchild Aerial Collection
12-26-1952	B&W	5K	84-86	1:20,000	Continental Aerial Surveys
2-11-1953	B&W	C-18785-1	100	1:14,400	Fairchild Aerial Collection
5-2-1953	B&W	C-19400-V11-LA	1-33, 2-28	1:63,360	Fairchild Aerial Collection
3-7-1955	B&W	C-21678-2	23-25	1:18,000	Fairchild Aerial Collection
1-17-1958	B&W	C-23023-V11-ORA 5	82, 83	1:36,000	Fairchild Aerial Collection
3-25-1959	B&W	261-3-14	66-68	1:12,000	Continental Aerial Surveys
3-25-1959	B&W	261-3-15	110-112	1:12,000	Continental Aerial Surveys
6-3-1961	B&W	C-24129	10	1:24,000	Fairchild Aerial Collection

AERIAL PHOTOGRAPHS REVIEWED (Continued)

Date	Type	Flight	Frames	Approximate Scale	Source
3-1-1967	B&W	1	32, 33	1:24,000	Continental Aerial Collection
2-18-1970	B&W	61-6	270	1:48,000	Continental Aerial Surveys
10-29-1973	B&W	132-6	6-8	1:24,000	Continental Aerial Surveys
1-13-1975	B&W	157-7	14, 15	1:24,000	Continental Aerial Surveys
12-28-1976	B&W	181-7	12-14	1:24,000	Continental Aerial Surveys
12-10-1978	B&W	203-7	15, 16	1:24,000	Continental Aerial Surveys
2-25-1980	B&W	80033	75, 76	1:32,000	Continental Aerial Surveys
4-2-1983	B&W	218-7	13-15	1:24,000	Continental Aerial Surveys
1-9-1987	B&W	F	232, 233	1:34,300	Continental Aerial Surveys
1-29-1992	B&W	C-85-7	16, 17	1:25,800	Continental Aerial Surveys
6-9-1993	B&W	C-93-13	176, 177	1:25,800	Continental Aerial Surveys
1-29-1995	B&W	C-103-35	115, 116	1:24,000	Continental Aerial Surveys
10-15-1997	B&W	C-117-35	230, 231	1:24,000	Continental Aerial Surveys
2-24-1999	B&W	C-134-35	121, 122	1:24,000	Continental Aerial Surveys

PLATES



SOURCE: U.S.G.S. 7.5' topographic series, Anaheim and Orange, California quadrangle dated 1965 (1964), photorevised 1981.

N

2,000 1,000 0 2,000

APPROXIMATE SCALE (feet)

PLOTTED: 26 Oct 2009, 1:39pm, MGriffin

CAD FILE: L:\2009\CADD\103567\10-15-09_GEO\ LAYOUT: 1

Images: Topo-plate1_1.JPG Images: Topo-plate1_2.JPG

ATTACHED IMAGES:
ATTACHED XREFS:
DIAMOND BAR, CA

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PROJECT NO.	103567
DRAWN:	10/26/09
DRAWN BY:	MRG
CHECKED BY:	PJM
FILE NAME:	103567p1.dwg

SITE VICINITY MAP

GEOTECHNICAL FEASIBILITY STUDY
PROPOSED ANAHEIM REGIONAL TRANSPORTATION
INTERMODAL CENTER (ARTIC) - PHASE 1 REPORT
ANAHEIM, CALIFORNIA

PLATE

1

EXPLANATION

W-2 ⊕ APPROXIMATE WELL LOCATION

C-7 ▲ APPROXIMATE CPT LOCATION

B-5 ● APPROXIMATE BORING LOCATION

REFERENCE: BASE MAP IS PHOTOGRAMMETRIC SURVEY BY DMI, DATED JULY 2008.

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EXPLORATION MAP

GEOTECHNICAL FEASIBILITY STUDY
PROPOSED ANAHEIM REGIONAL TRANSPORTATION
INTERMODAL CENTER (ARTIC) - PHASE 1
ANAHEIM, CALIFORNIA

PLATE
2

PLOTTED: 26 Oct 2009, 1:41pm, MGriffin
LAYOUT: 2
ATTACHED IMAGES: Images: ARTIC - Boring Location Map.jpg Images: IMAGE-Plate2.JPG
ATTACHED XREFS: XRef: 108816JMPR1: XRef: 108816JAT01: XRef: 108816JAL01MOD
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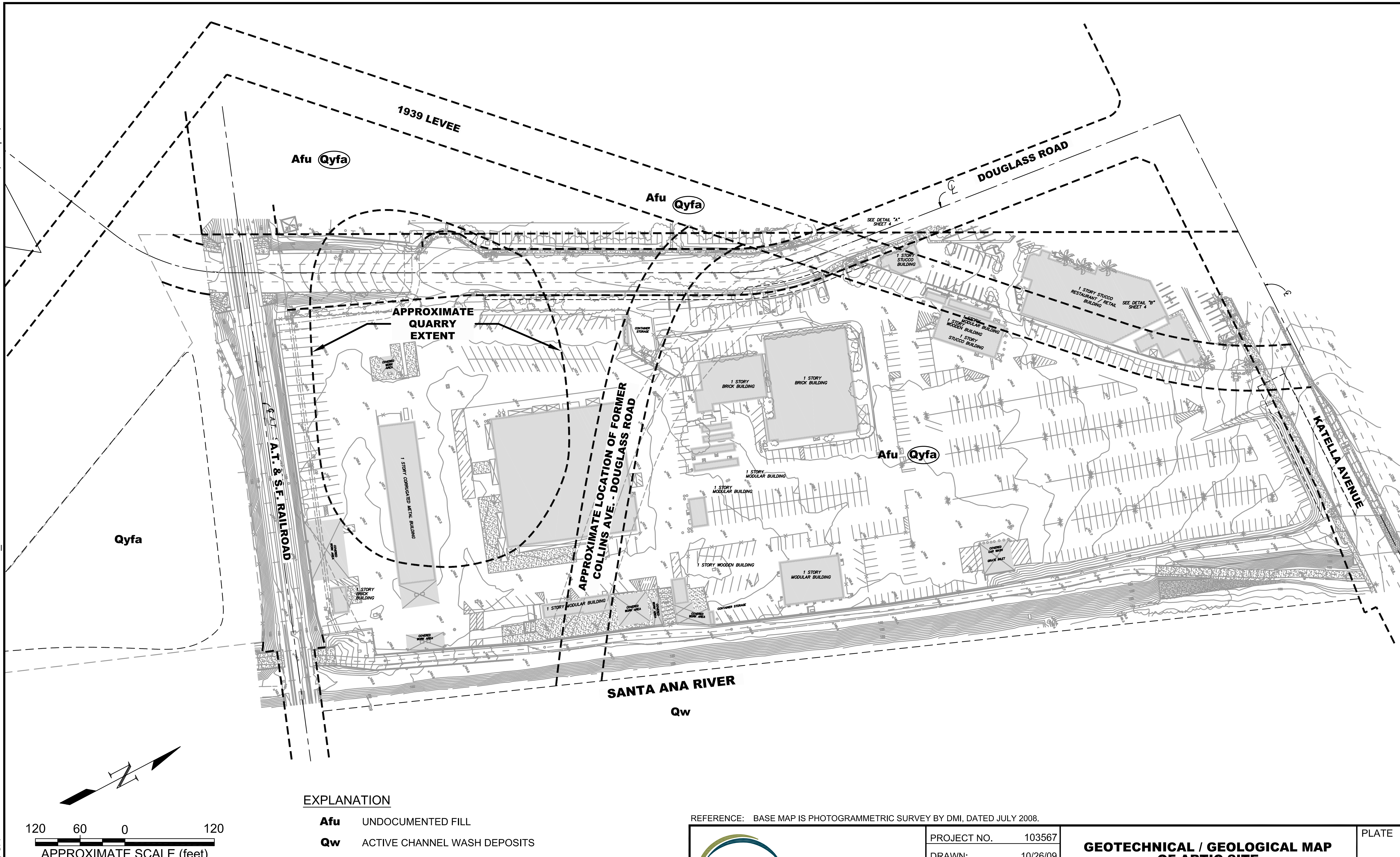

EXPLANATION

Afu	UNDOCUMENTED FILL
Qw	ACTIVE CHANNEL WASH DEPOSITS
Qyfa	ALLUVIAL FAN DEPOSITS OF SAND (CIRCLED WHERE BURIED)
— — —	APPROXIMATE GEOLOGIC CONTACT
— — —	APPROXIMATE FEATURE BOUNDARY

REFERENCE: BASE MAP IS PHOTOGRAMMETRIC SURVEY BY DMI, DATED JULY 2008.

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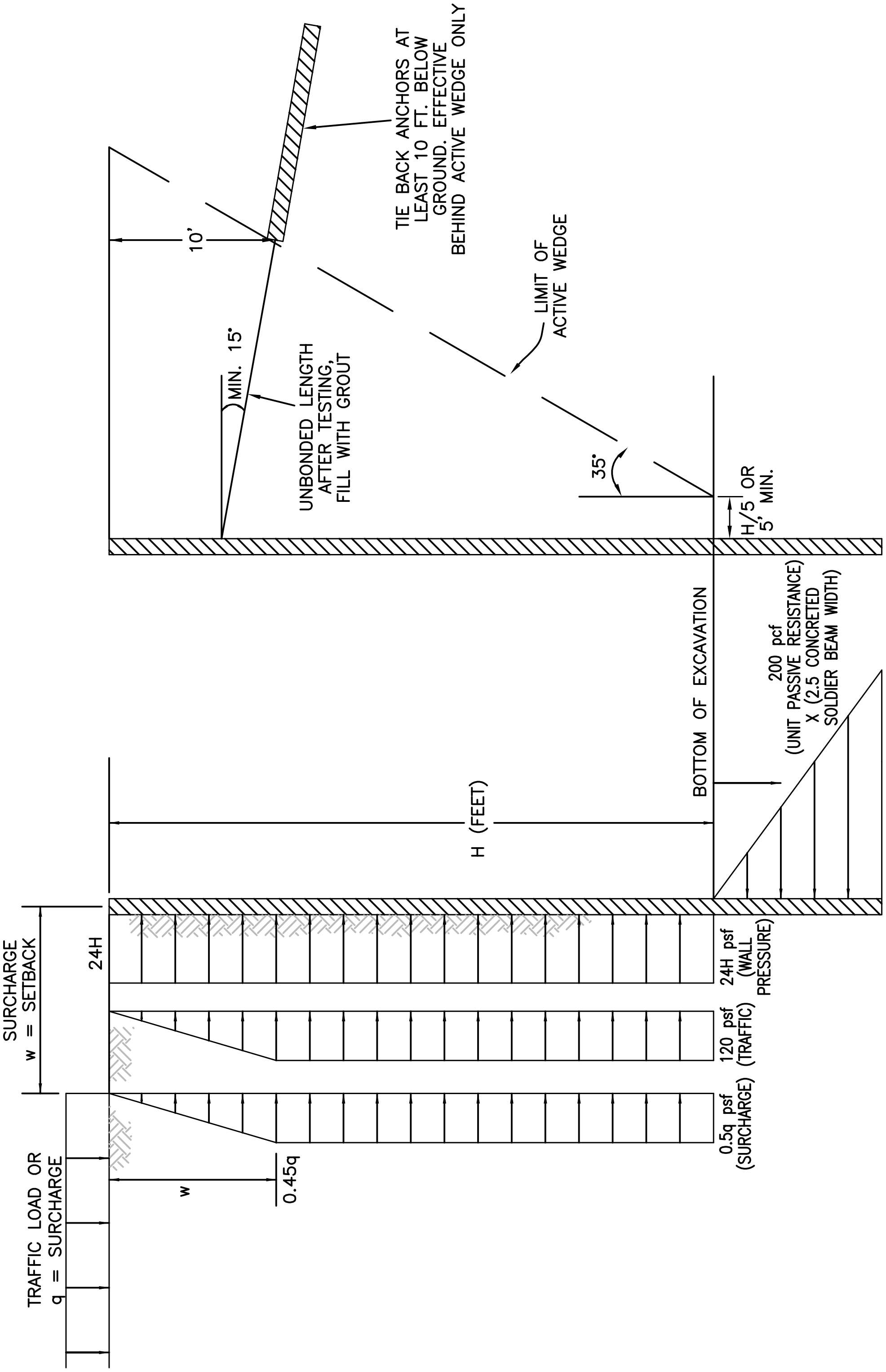
GEOTECHNICAL / GEOLOGICAL MAP OF ARTIC SITE

GEOTECHNICAL FEASIBILITY STUDY
ABUTMENT AND DOUGLASS RD. MODIFICATIONS
INTERMODAL CENTER (ARTIC) - PHASE 1 REPORT
ANAHEIM, CALIFORNIA

PLATE

3



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FILE NAME:	103567p4.dwg

RECOMMENDED LATERAL EARTH PRESSURES FOR TEMPORARY SHORING

GEOTECHNICAL FEASIBILITY STUDY
PROPOSED ANAHEIM REGIONAL TRANSPORTATION INTERMODAL CENTER (ARTIC) - PHASE 1
ANAHEIM, CALIFORNIA

PLATE

4

PLOTTED: 26 Oct 2009, 1:43pm, MGriffin

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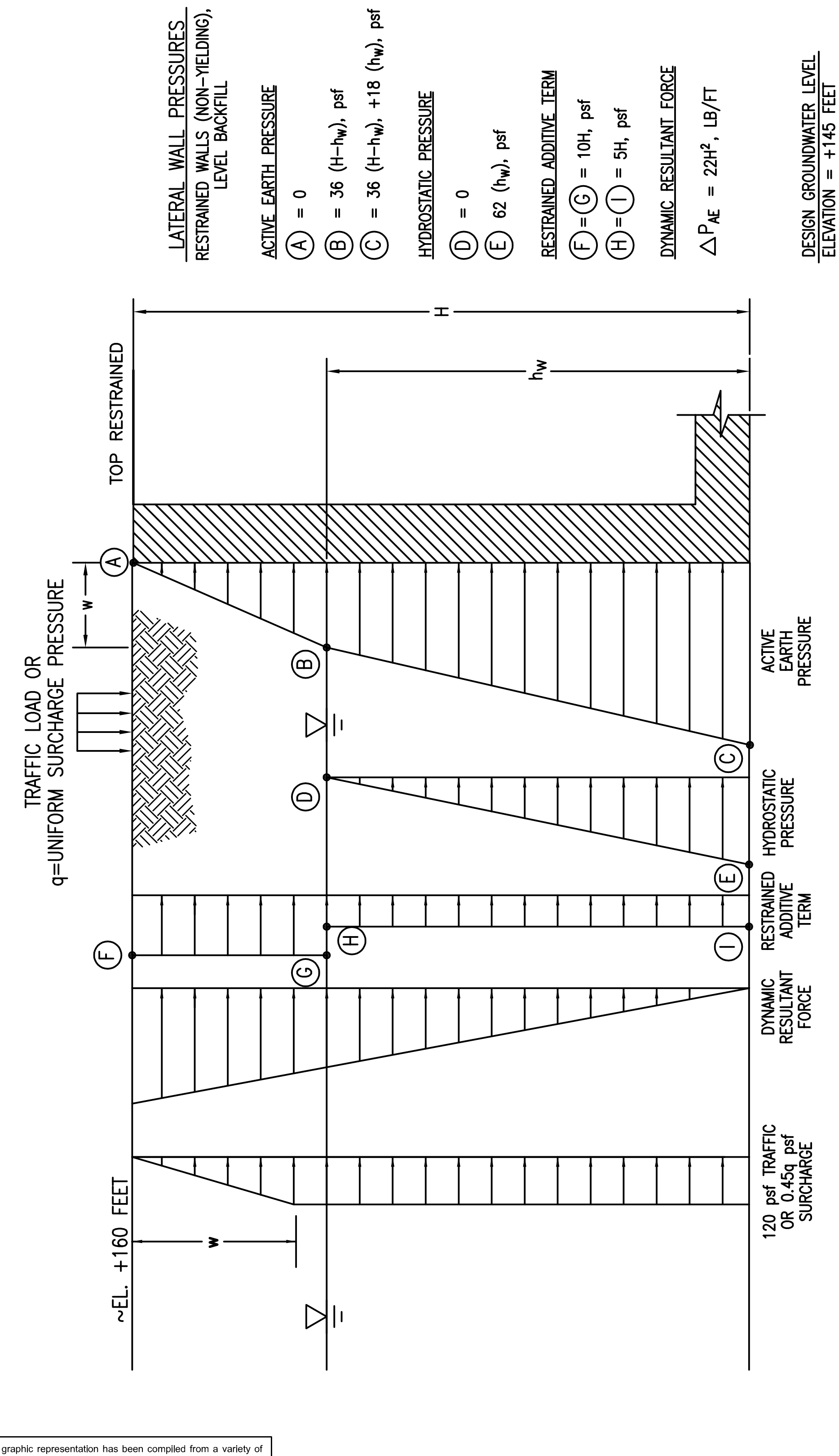

ATTACHED IMAGES:
ATTACHED XREFS:
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DRAWN BY:	MRG
CHECKED BY:	PJM
FILE NAME:	103567p5.dwg

RECOMMENDED LATERAL EARTH PRESSURES FOR PERMANENT BASEMENT WALLS

GEOTECHNICAL FEASIBILITY STUDY
PROPOSED ANAHEIM REGIONAL TRANSPORTATION INTERMODAL CENTER (ARTIC) - PHASE 1
ANAHEIM, CALIFORNIA

PLATE
5

APPENDIX A
FIELD EXPLORATIONS

APPENDIX A
FIELD EXPLORATIONS

GENERAL

Our field exploration program consisted of a site reconnaissance and drilling five borings, installing two groundwater monitoring wells, and advancing seven Cone Penetration Tests (CPTs). The borings were drilled to depths between approximately 51½ and 101½ feet below the existing ground surface (bgs). The CPTs were advanced to depths between approximately 38 and 94 feet bgs. The approximate locations of the borings, wells and CPTs are presented on Plate 2.

Prior to commencement of the fieldwork, various geophysical techniques were used at each boring, well and CPT location in order to identify potential conflicts with subsurface structures. Each of our proposed field exploration locations were also cleared for buried utilities through Underground Service Alert (USA).

BORINGS AND MONITORING WELLS

The borings and monitoring wells were drilled on September 22 through 25 by Cal Pac Drilling of Calimesa, California with a truck-mounted, hollow-stem-auger drilling rig equipped with an auto-hammer (Mobile B61). After completion, the borings were backfilled using bentonite grout and bentonite chips upon completion of the drilling. The borings were then capped with quickset concrete. The monitoring wells were constructed in two boreholes after completion of drilling. The well construction is presented on the logs.

A Modified California sampler was used to obtain drive samples of the soil encountered. This sampler consists of a 3-inch O.D., 2.4-inch I.D. split barrel shaft that is pushed or driven a total of 18-inches into the soil at the bottom of the boring. The soil was retained in six-inch long metal sleeve and in six 1-inch brass rings for laboratory testing. An additional 2 inches of soil from each drive remained in the cutting shoe and was usually discarded after visually classifying the soil. The sampler was driven using a 140-pound hammer falling 30 inches. The total number of blows required to drive the sampler the final 12 inches is termed blow count and is recorded on the Logs of Borings.

Samples were also obtained using a Standard Penetration Sampler (SPT). This sampler consists of a 2-inch O.D., 1-inch I.D. split barrel shaft that is advanced into the soils at the bottom of the drill hole a total of 18 inches. The sampler was driven using a 140-pound hammer falling 30-inches. The total number of hammer blows required to drive the sampler the final 12 inches is termed the blow count (N) and is recorded on the Logs of Borings. The procedures we employed in the field are generally consistent with those described in ASTM Standard Test Method D1586-84. Bulk samples of the near-surface soils were directly retrieved from the auger cuttings.

The Logs of Borings are presented as Plates A-2 through A-6 and the Well Logs are on Plates A-7 and A-8. An explanation to the logs is presented as Plates A-1a and A-1b. The Logs of Borings describe the earth materials encountered, samples obtained and show field and laboratory tests performed. The logs also show the location, boring number, drilling date and the name of the drilling subcontractor. The borings were logged by a Kleinfelder engineer using the Unified Soil Classification System. The boundaries between soil types shown on the logs are approximate because the transition between different soil layers may be gradual.

CPT SOUNDINGS

The CPTs were advanced by Kehoe Testing and Engineering of Huntington Beach, California using a truck-mounted rig. The CPT involves pushing a conical-shaped probe into a soil deposit and recording the resistance of the soil to penetration. Test equipment consists of a cone assembly, a series of hollow sounding rods, a hydraulic frame to push the cone and rods into the soil, an electronic data processing unit, and a truck to transport the test equipment and provide thrust resistance.

The cone penetrometer consists of a conical tip with a 60-degree apex angles and a cylindrical friction sleeve. The interior of the device is instrumented with strain gauges allowing simultaneous measurements of cone penetration resistance and sleeve friction during testing. Electric signals from the strain gauges are transmitted by cable through the hollow sounding rods to a data processing unit. The cone assembly used on this project had a cross-sectional area of 15-square centimeters and a friction sleeve surface area of 225 square centimeters. Plots of the tip resistance (tip bearing) and friction ratio for each CPT performed during this investigation are provided in this Appendix.

CPT data can be used to derive several significant soil parameters related to foundation design and performance. The end bearing resistance of the cone tip (generally referred to as the tip resistance) is an indicator of both in-situ bearing capacity and compressibility. Indirectly, tip resistance can also be an indicator of soil type, since a fine-grained soil typically has a lower tip resistance than a coarse-grained soil.

The sleeve friction resistance is an indirect indicator of in-situ shear strength. In addition, the friction ratio (expressed as a percentage), is an indicator of soil behavior types. Sands typically have low friction ratios (0 to 2½ percent) while clays have higher friction ratios (typically more than 4 percent).

The combination of CPT data defining soil behavior type and penetration resistance allows rapid interpretation of subsurface stratigraphy. A general classification of soil strata can be obtained from the data using the CPT Classification Chart provided in the attached CPT report in this Appendix. Since the CPT provides near-continuous information throughout the stratigraphy penetrated, it is possible to identify thinner soil units that could go undetected in selectively sampled boring.

Date Drilled:
Drilled By:
Drilling Method:
Logged By:

Water Depth:
Date Measured:
Reference Elevation:
Datum:

Elevation (feet) Depth	Sample	Sample No.	Blow Count (Blows/ft.)	Graphic Log	GEOTECHNICAL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
		1	6			108	10	DS, SE
		2	12					GS
5								
	(1)	(2)	(3)	(4)	(5)	(6)	(6)	(7)

NOTES ON FIELD INVESTIGATION

1. SAMPLE – Graphical representation of sample type as shown below.
   - Split Spoon – Standard Penetration Test Sample (SPT)
   - Drive Sample – California Sample (Cal)
   - Bulk Sample – Obtained by collecting cuttings in a plastic bag
   - Tube Sample – Shelby/Pitcher Tube Sample
2. SAMPLE NO. – Sample Number
3. BLOWS/FT – Number of blows required to advance sampler 1 foot (unless a lesser distance is specified). Samplers in general were driven into the soil at the bottom of the hole with a standard (140 lb) hammer dropping a standard 30 inches. Drive samples collected in bucket auger borings may be obtained by dropping non-standard weight from variable heights. When a SPT sampler is used the blow count conforms to ASTM D-1586.

   SCR/RQD – Sample Core Recovery (SCR) in percent (%) and Rock Quality Designation (RQD) in percent (%). RQD is defined as the percentage of core in each run which the spacing between natural fractures is greater than 4 inches. Mechanical breaks of the core are not considered.
4. GRAPHIC LOG – Standard symbols for soil and rock types, as shown on plate A-1b.
5. GEOTECHNICAL DESCRIPTION

   Soil – Soil classifications are based on the United Soil Classification System per ASTM D-2987, and designations include consistency, moisture, color and other modifiers. Field descriptions have been modified to reflect results of laboratory analyses where deemed appropriate.

   Rock – Rock classifications generally include a rock type, color, moisture, mineral constituents, degree of weathering, alteration, and the mechanical properties of the rock. Fabric, lineations, bedding spacing, foliations, and degree of cementation are also presented where appropriate.

   Description of soil origin or rock formation is placed in brackets at the beginning of the description where applicable, for example, Residual Soil.
6. DRY DENSITY, MOISTURE CONTENT: As estimated by laboratory or field testing.
7. ADDITIONAL TESTS – (Indicates sample tested for properties other than the above):

MAX – Maximum Dry Density	SG – Specific Gravity	PP – Pocket Penetrometer
GS – Grain Size Distribution	HA – Hydrometer Analysis	WA – Wash Analysis
SE – Sand Equivalent	AL – Atterberg Limits	DS – Direct Shear
EI – Expansion Index	RV – R-Value	CP – Collapse Potential
CHEM – Sulfate and Chloride Content, pH, Resistivity	CN – Consolidation	UC – Unconfined Compression
PM – Permeability	CU – Consolidation Undrained Triaxial	T – Torvane
UU – Unconsolidated Undrained Triaxial	CD – Consolidated Drained Triaxial	

8. ATTITUDES – Orientation of rock discontinuity observed in bucket auger boring or rock core, expressed in strike/dip and dip angle, respectively, preceeded by a one-letter symbol denoting nature of discontinuity as shown below.

   B: Bedding Plane J: Jointing C: Contact F: Fault S: Shear

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EXPLANATION OF LOGS

PLATE
A-1a

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

PRIMARY DIVISIONS			GROUP SYMBOLS	SECONDARY DIVISIONS
COURSE GRAINED SOILS MORE THAN HALF OF MATERIALS IS LARGER THAN #200 SIEVE SIZE	GRAVELS MORE THAN HALF OF COURSE FRACTION IS LARGER THAN #4 SIEVE	CLEAN GRAVELS (LESS THAN) 5% FINES	GW	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
			GP	POORLY GRADED GRAVELS OR GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
		GRAVEL WITH FINES	GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SANDS MORE THAN HALF OF COURSE FRACTION IS SMALLER THAN #4 SIEVE	CLEAN SANDS (LESS THAN) 5% 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	SM	SILTY SANDS, SAND-SILT MIXTURES
			SC	CLAYEY SANDS, SAND-CLAY MIXTURES
FINE GRAINED SOILS MORE THAN HALF OF MATERIALS IS SMALLER THAN #200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT IS LESS THAN 50	ML	INORGANIC SILTS, VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC SILTS AND ORGANIC SILT-CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT IS GREATER THAN 50	MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDS OR SILTS, 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, MUCK AND OTHER HIGHLY ORGANIC SOILS
TYPICAL FORMATIONAL MATERIALS	SANDSTONES		SS	
	SILTSTONES		SH	
	CLAYSTONES		CS	
	LIMESTONES		LS	
	SHALE		SL	

CONSISTENCY CRITERIA BASED ON FIELD TESTS

RELATIVE DENSITY – COARSE – GRAIN SOIL

RELATIVE DENSITY	SPT * (# blows/ft)	RELATIVE DENSITY (%)
Very Loose	<4	0 – 15
Loose	4 – 10	15 – 35
Medium Dense	10 – 30	35 – 65
Dense	30 – 50	65 – 85
Very Dense	>50	85 – 100

CONSISTENCY– FINE–GRAIN SOIL

CONSISTENCY	SPT (# blows/ft)	TORVANE UNDRAINED SHEAR STRENGTH (tsf)	POCKET ** PENETROMETER UNCONFINED COMPRESSIVE STRENGTH (tsf)
Very Soft	<2	<0.13	<0.25
Soft	2 – 4	0.13 – 0.25	0.25 – 0.5
Medium Stiff	4 – 8	0.25 – 0.5	0.5 – 1.0
Stiff	8 – 15	0.5 – 1.0	1.0 – 2.0
Very Stiff	15 – 30	1.0 – 2.0	2.0 – 4.0
Hard	>30	>2.0	>4.0

* NUMBER OF BLOWS OF 140 POUND HAMMER FALLING 30 INCHES TO DRIVE A 2 INCH O.D. (1 3/8 INCH I.D.) SPLIT BARREL SAMPLER (ASTM–1586 STANDARD PENETRATION TEST)

** UNCONFINED COMPRESSIVE STRENGTH IN TONS/SQ.FT. READ FROM POCKET PENETROMETER

MOISTURE CONTENT

DESCRIPTION	FIELD TEST
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

CEMENTATION

DESCRIPTION	FIELD TEST
Weakly	Crumbles or breaks with handling or slight finger pressure
Moderately	Crumbles or breaks with considerable finger pressure
Strongly	Will not crumble or break with finger pressure

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EXPLANATION OF LOGS

PLATE
A-1b

Date Drilled:	9/24/09	Water Depth:	Not Encountered
Drilled By:	Cal Pac Drilling	Date Measured:	9/24/2009
Drilling Method:	Hollow Stem Auger	Elevation:	161 feet (approx.)
Logged By:	K. S.	Datum:	NAVD 88

Elevation (feet)	Depth	Sample Number	Blows per Foot	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
160				**Artificial Fill:** **Pavement:** approx. 2.75 inches of asphalt over 16.5 inches of base.			
		1, 2		**Poorly Graded Sand (SP):** olive gray to light brownish gray, moist, fine to medium grained, trace of fine gravel.		4.3	
		3		-- brown		4.7	CHEM
	5			**Poorly Graded Sand with Silt (SP-SM):** olive brown, medium dense to dense, moist, fine to coarse grained.			
155		4	38		114	6.5	
		5	8	-- loose, fine to medium grained, trace fine gravel.		5.4	
	10			**Alluvium:** **Poorly Graded Sand (SP):** light brownish gray, loose, moist, fine to medium grained, pocket of sandy clay, layers of sand with silt.			
150		6	5		104	5.8	
		7	9	**Sandy Silt (ML):** olive gray to light brownish gray, medium stiff, fine to medium grained.		17.5	
	15			**Poorly Graded Sand (SP):** pink, olive, yellow, loose, moist, fine grained.			
145		8	10			4.1	
	20						
140		9	26	-- light brown, gray, pink, medium dense, fine to medium grained, micaceous.	101	4.1	
	25						
135		10	14			3.6	WA (3% fines)
				Poorly Graded Sand with Silt (SP-SM): light gray, medium dense, moist, fine to medium grained.			
	30						
130		11	30		97	5.2	

GEOTECH DB 103567 ARTIC BORINGS, WELLS, CPTS.GPJ KA_RDLND.GDT 10/25/09

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LOG OF BORING B-1

PLATE A-2a

Drafted By: Reviewed By:

Legend To Logs On Plate A-1

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Elevation (feet)	Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION *(Continued From Previous Page)*	Dry Density (pcf)	Moisture Content (%)	Additional Tests
125			12	17		**Poorly Graded Sand (SP):** olive brown, medium dense, moist, fine to medium grained, -- increase coarse sand		3.5	
120	40		13	20 50/5"		**Poorly Graded Sand with Gravel (SP):** olive brown, medium dense, moist, fine to coarse grained. -- olive yellow, very dense, with silt and gravel.	110	3.8	
115	45		14	38		**Poorly Graded Gravel (GP):** brown, dense, fine to coarse grained, broken gravel.		4.5	
						Lean Clay with Sand (CL): yellowish brown, hard, moist.			
110	50		15	32			111	19.1	
105	55		16	9				21.8	
						Poorly Graded Gravel with Sand (GP): dry, dense, fine to medium grained.			
100	60		17	39		**Lean Clay with Sand (CL):** yellowish brown, hard, moist.	124	2.2	
95	65		18	18				17.0	
						Silty Sand (SM): yellowish brown, medium dense, moist, fine grained, pockets of clean sand.			
90	70		19	28			110	13.9	
						Sandy Silt (ML): yellowish brown, stiff, moist.			

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LOG OF BORING B-1

PLATE
A-2b

Legend To Logs On Plate A-1

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Elevation (feet)	Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION *(Continued From Previous Page)*	Dry Density (pcf)	Moisture Content (%)	Additional Tests
85			20	9		**Sandy Silt (ML):** yellowish brown, stiff, moist. *(continued)*		17.8	
						Silty Sand (SM): olive brown, dense, moist, fine to medium grained.			
80	80		21	41			111	8.6	

Total depth: 81.5 feet.
Free water encountered on top of sample at 51 feet.
Boring backfilled with bentonite slurry and capped with quickset conrete.

Legend To Logs On Plate A-1
Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Date Drilled:	9/24/09	Water Depth:	83 feet
Drilled By:	Cal Pac Drilling	Date Measured:	9/24/2009
Drilling Method:	Hollow Stem Auger	Elevation:	158 feet (approx.)
Logged By:	K. S.	Datum:	NAVD 88

Elevation (feet)	Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
						Artificial Fill:			
						Pavement: approx 5 inches of asphalt over 5 inches of base			
			1			**Poorly Graded Sand (SP):** pink, moist, fine to coarse grained, trace fine gravel, layers of sand with silt.		3.1	
155			2						
			3			-- olive yellow, fine to medium grained.		3.5	DS
	5								
			4	26		-- medium dense, fine to coarse grained.	109	3.3	
150			5	8		-- pink, loose, fine to medium grained, trace fine gravel.		3.9	
	10								
			6	18		-- olive brown, medium dense.	106	4.6	
						Alluvium:			
145			7	20		**Poorly Graded Sand (SP):** pink, loose to medium dense, dry, fine to medium grained.	100	2.8	
	15		8	7				3.1	
140									
	20								
			9	7		**Lean Clay (CL):** greenish black, medium stiff, moist.	69	51.5	WA (95% fines)
135						**Silty Sand (SM):** brown, loose, moist.			
	25		10	9				7.9	WA (32% fines)
130						**Sandy Silt (ML):** yellowish brown, stiff, moist, fine grained sand.			
	30		11	21			107	16.1	WA (56% fines)
125									

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LOG OF BORING B-2

PLATE
A-3a

Drafted By: Reviewed By:

Legend To Logs On Plate A-1

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

GEOTECH DB 103567 ARTIC BORINGS, WELLS, CPTS.GPJ KA_RDLND.GDT 10/25/09

Elevation (feet)	Depth	Sample Number	Blows per Foot	SOIL DESCRIPTION AND CLASSIFICATION *(Continued From Previous Page)*	Dry Density (pcf)	Moisture Content (%)	Additional Tests
		12	12	**Sandy Lean Clay (CL):** yellowish brown, stiff, moist, layers of clayey sand. *(continued)*		16.7	
120				**Poorly Graded Gravel with Sand (GP):** olive gray, very dense.			
	40	13	74		129	3.5	
115							
	45	14	9	**Lean Clay with Sand (CL):** yellowish brown, stiff, moist, gravel inclusions.		14.2	AL LL = 30 PL = 16
110							
	50	15	15		105	20.5	CN
105				**Sandy Silt (ML):** yellowish brown, stiff, moist, calcium stringers.			
	55	16	11			15.3	
100				**Lean Clay with Sand (CL):** yellowish brown, very stiff, moist.			
	60	17	28		115	16.9	WA (81% fines)
				Sandy Silt (ML): yellowish brown, stiff, moist.			
95							
	65	18	13			15.8	
90							
				Lean Clay with Sand (CL): yellowish brown, stiff, moist. layers of sandy silt.			
	70	19	21		100	26.4	
85							
				Sandy Silt (ML): yellowish brown, very stiff, moist.			

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LOG OF BORING B-2

PLATE

A-3b

Legend To Logs On Plate A-1
Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

GEOTECH DB 103567 ARTIC BORINGS, WELLS, CPTS.GPJ KA_RDLND.GDT 10/25/09

Elevation (feet)	Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION *(Continued From Previous Page)*	Dry Density (pcf)	Moisture Content (%)	Additional Tests
			20	19		**Sandy Silt (ML):** yellowish brown, very stiff, moist. *(continued)*		16.7	
80									
	80					**Sandy Lean Clay (CL):** yellowish brown, very stiff, moist, sandy silt and silty sand layers.			
			21	25				19.3	WA (53% fines)
						Poorly Graded Sand with Silt (SP-SM): yellowish brown, loose, wet, fine to medium grained, trace subrounded gravel.			
75 ▼									
	85		22					16.9	
70									
	90		23	38 50/2"		**Poorly Graded Sand with Silt and Gravel (SP-SM):** olive brown, very dense, moist, fine to coarse grained.	133	8.3	
65						**Poorly Graded Gravel with Sand (GP):** olive brown, very dense, wet, fine to coarse grained sand.			
	95		24	36				10.7	
60									
	100					**Poorly Graded Sand with Gravel (SP):** olive brown, very dense, wet.			
			25	92			127	9.4	

Total depth: 101.5 feet.
Groundwater encountered at approximately 83 feet.
Boring backfilled with bentonite slurry and capped with quickset concrete.

GEOTECH DB 103567 ARTIC BORINGS, WELLS, CPTS.GPJ KA_RDLND.GDT 10/25/09

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ARTIC
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LOG OF BORING B-2

PLATE
A-3c

Legend To Logs On Plate A-1
Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Date Drilled:	9/22/09	Water Depth:	58 feet
Drilled By:	Cal Pac Drilling	Date Measured:	9/22/2009
Drilling Method:	Hollow Stem Auger	Elevation:	156 feet (approx.)
Logged By:	K. S. and F. J.	Datum:	NAVD 88

Elevation (feet)	Depth	Sample Number	Blows per Foot	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
				Artificial Fill:			
155				**Pavement:** approx 2.75 inches of asphalt over 6.5 inches of base			
		1		**Poorly Graded Sand (SP):** olive brown, dense, slightly moist, trace fine to coarse gravel, layers of sand with silt.		2.0	
		2					
		3					
		4				4.8	
	5	5					
150		6	36	-- brown, moist, small clay pockets.	118	4.6	
		7	50 / 5"	--brown with light brown inclusions, moist, fine to medium grained.	118	4.5	
145	10	8	70		123	7.3	
				Silty Sand (SM): olive brown, medium dense, fine to coarse grained.			
		9	16			10.1	
	15			**Alluvium:**			
140		10	12	**Sand with Silt (SP-SM):** light brownish gray, medium dense, slightly moist, moderate iron oxide staining, lumps of clay.		6.0	
				Poorly Graded Sand (SP): pink, medium dense, moist, micaceous, fine to medium grained.			
135	20	11	28		105	2.2	
130	25	12	7	**Silty Sand (SM):** olive brown to yellowish brown, loose, very moist, fine grained.		11.3	WA (30% fines)
				Sandy Silt (ML)			
				Silty Sand (SM): olive brown to yellowish brown, loose, moist, fine grained.			
125	30	13	6		101	22.3	WA (39% fines)
				Sandy Lean Clay (CL): yellowish brown, very stiff, moist, layers of clayey sand.			

GEOTECH DB 103567 ARTIC BORINGS, WELLS, CPTS.GPJ KA_RDLND.GDT 10/25/09

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PROJECT NO. 103567	**LOG OF BORING B-3**	

Drafted By: Reviewed By:

Legend To Logs On Plate A-1

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Elevation (feet)	Depth	Sample Number	Blows per Foot	SOIL DESCRIPTION AND CLASSIFICATION *(Continued From Previous Page)*	Dry Density (pcf)	Moisture Content (%)	Additional Tests
120		14	15	**Sandy Lean Clay (CL):** yellowish brown, very stiff, moist, layers of clayey sand. *(continued)*		16.7	WA (72% fines) AL LL = 30 PL = 15
115	40	15	11	-- lense of yellowish brown silty sand, trace fine gravel, very moist.	105	21.9	WA (63 % fines)
110	45	16	9			19.9	WA (73% fines)
				Lean Clay (CL): yellowish brown, stiff, moist.			
105	50	17	9		101	25.5	WA (89% fines)
	55					31.6	
100		18	33	**Silty Sand with Gravel (SM):** yellowish brown, medium dense, fine to coarse grained gravel.		10.8	
				Poorly Graded Sand with Gravel (SP): gray, medium dense, wet, fine to coarse sand.			
95	60	19	36		121	13.8	
				Lean Clay with Sand (CL): yellowish brown, stiff, moist.			
90	65	20	11			24.7	
85	70	21	16		103	23.6	
						22.2	

GEOTECH DB 103567 ARTIC BORINGS, WELLS, CPTS.GPJ KA_RDLND.GDT 10/25/09

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PROJECT NO. 103567

ARTIC
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LOG OF BORING B-3

PLATE
A-4b

Legend To Logs On Plate A-1
Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Elevation (feet)	Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION *(Continued From Previous Page)*	Dry Density (pcf)	Moisture Content (%)	Additional Tests
80			22	13		**Clayey Sand (SC):** yellowish brown, medium dense, very moist, fine grained.		16.2	
	80					**Silty Sand (SM):** yellowish brown, medium dense, moist to very moist, trace gravel.			
75			23	30				16.8	

Total depth: 81.5 feet.
Groundwater encountered at approximately 58 feet.
Boring backfilled with bentonite slurry and capped with quickset concrete.

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ARTIC
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Anaheim, California

LOG OF BORING B-3

PLATE
A-4c

Legend To Logs On Plate A-1
Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Date Drilled:	9/23/09	Water Depth:	87 feet
Drilled By:	Cal Pac Drilling	Date Measured:	9/23/2009
Drilling Method:	Hollow Stem Auger	Elevation:	158 feet (approx.)
Logged By:	F. J.	Datum:	NAVD 88

SOIL DESCRIPTION AND CLASSIFICATION

Elevation (feet)	Depth	Sample Type	Sample Number	Blows per Foot	Soil Description and Classification	Dry Density (pcf)	Moisture Content (%)	Additional Tests
	0				**Artificial Fill:** **Pavement:** approx. 2.5 inches of asphalt over 5.25 inches of base			
			2		**Poorly Graded Sand (SP):** yellowish brown, moist, fine to medium grained, layers of sand with silt, layers of silty sand.		3.1	
155			3					
	5		4				6.0	CHEM
					Poorly Graded Sand with Silt (SP-SM): yellowish brown, dense, moist, fine to medium grained, layers of silty sand.			
			5	41		122	5.4	
					Poorly Graded Sand (SP): yellowish brown, medium dense, dry, fine to medium grained.			
150			6	24	-- brown, lumps of clay, rounded gravel.		3.9	
	10							
			7	38	-- darker, dense, inclusions of silty sand, moist.	112	5.8	
145			8	16		107	7.4	
					Alluvium: **Silty Sand (SM):** olive brown, medium dense, moist, fine to medium grained.			
	15		9	3	-- thin sandy clay layer at 16 feet.		7.0	
					Poorly Graded Sand with Silt (SP-SM): brown, medium dense, moist, fine to medium grained.			
140					-- olive brown sandy clay			
					-- poorly graded sand			
	20		10	25	-- light brown, dry, fine to coarse grained.	104	2.5	
					Silty Sand (SM): brown, loose, moist.			
135								
	25		11	10			9.1	WA (26% fines)
130								
	30		12	13		104	8.9	WA (34% fines)
					Sandy Lean Clay (CL): yellowish brown, medium stiff, wet, trace subrounded gravel.			
125								

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PROJECT NO. 103567

LOG OF BORING B-4

PLATE A-5a

Drafted By: Reviewed By:

Legend To Logs On Plate A-1

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Elevation (feet)	Depth	Sample Type	Sample Number	Blows per Foot	SOIL DESCRIPTION AND CLASSIFICATION *(Continued From Previous Page)*	Dry Density (pcf)	Moisture Content (%)	Additional Tests
			13	9	**Sandy Lean Clay (CL):** yellowish brown, medium stiff, wet, trace subrounded gravel. *(continued)*		17.4	WA (57% fines) CHEM
120					**Sand with Silt and Gravel (SW-SM):** olive brown, very dense, moist, fine to coarse grained, moderate iron oxide staining.			
	40		14	22 43 50/5"		134	5.5	
115								
	45		15	67	**Poorly Graded Gravel with Sand (GP):** brown, very dense, moist, medium to coarse grained sand, layers of sand.		4.6	
110								
	50		16	17	**Lean Clay with Sand (CL):** yellowish brown, stiff, moist.	107	19.0	
105								
	55		17	7			23.3	
100					**Silty Sand (SM):** yellowish brown, medium dense, moist, fine to medium grained.			
	60		18	13		113	16.5	WA (46% fines)
95								
	65		19	8	**Lean Clay with Sand (CL):** yellowish brown, medium stiff, moist, fine grained sand.		18.5	
90					**Silty Sand (SM):** yellowish brown, medium dense, moist.			
	70		20	34		115	10.7	
85								

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ARTIC
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LOG OF BORING B-4

PLATE A-5b

Legend To Logs On Plate A-1

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Elevation (feet)	Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION *(Continued From Previous Page)*	Dry Density (pcf)	Moisture Content (%)	Additional Tests
						Silty Sand (SM): yellowish brown, medium dense, moist. *(continued)*		15.0	
			21	11		**Sandy Lean Clay (CL):** yellowish brown, stiff, moist.		20.9	
80									
	80								
			22	77		**Sand with Silt (SP-SM):** yellowish brown, medium dense, moist, fine to coarse grained -- decrease silt.	110	8.2	
75									
	85								
			23	42		-- dense, wet, with gravel.		7.4	
70									
	90								
			24	23		**Well Graded Sand with Gravel (SW):** gray, medium dense, wet, fine to coarse grained, trace of yellowish brown silty sand and sandy clay.	122	10.7	
65									
	95								
			25	36		**Poorly Graded Gravel with Sand (GP):** gray, very dense, wet, fine to medium grained, fine to coarse grained sand.		11.2	
60									
	100								
			26	90		-- olive brown	138	8.3	

Total depth: 101.5 feet.
Groundwater encountered at approximately 87 feet.
Boring backfilled with bentonite slurry and capped with quickset concrete.

GEOTECH DB 103567 ARTIC BORINGS, WELLS, CPTS.GPJ KA RDLND.GDT 10/25/09

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PROJECT NO. 103567

ARTIC
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Anaheim, California

LOG OF BORING B-4

PLATE
A-5c

Legend To Logs On Plate A-1
Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Date Drilled:	9/22/09	Water Depth:	Not Encountered
Drilled By:	Cal Pac Drilling	Date Measured:	9/22/2009
Drilling Method:	Hollow Stem Auger	Elevation:	157 feet (approx.)
Logged By:	F. J. and K. S.	Datum:	NAVD 88

Elevation (feet) Depth	Sample Number	Blows per Foot	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
			Artificial Fill:			
			Pavement: approx 3.5 inches of asphalt over 5 inches of base			
155	1, 2		**Poorly Graded Sand with Silt (SP-SM):** brown, dry, fine to coarse grained, trace gravel, layers of clean sand.		3.1	
	3					GS (8% fines)
5	4					
	5, 6	34	-- medium dense, fine to medium grained.	114	2.8	
150	7	32	-- light brown, slightly moist.		3.8	
10	8	72	-- brown, very dense.	107	4.3	
145	9	30	**Silty Sand (SM):** dark olive brown, medium dense, moist, fine to coarse grained, trace fine gravel, pockets of lean clay.	118	12.3	
15	10	16	-- dark brown with light brown inclusion.		13.5	
140						
20	11	20	-- layer of sandy lean clay.	111	4.2	
135			**Alluvium:**			
			Poorly Graded Sand (SP): light brown, moist, layers of silty sand.			
			Silty Sand (SM): brown, loose, moist, fine grained.			
25	12	8	-- loose, fine to medium grained, trace fine gravel, iron oxide staining.		6.4 12.7	
130						
30	13	14		114	16.2	WA (40% fines)
125						

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ARTIC
S. Douglass Road and Katella Avenue
Anaheim, California

LOG OF BORING B-5

PLATE
A-6a

Drafted By: Reviewed By:

Legend To Logs On Plate A-1

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Elevation (feet)	Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION *(Continued From Previous Page)*	Dry Density (pcf)	Moisture Content (%)	Additional Tests
			14	34		**Sand with Gravel (SP):** brown, dense, moist, fine to medium grained gravel, fine to coarse grained sand, layers of gravel. *(continued)*		5.8	
120									
						Well Graded Sand with Gravel (SW): brown, very dense, moist, fine to coarse grained gravel.			
	40		15	64			134	5.5	
115									
						Poorly Graded Sand with Gravel (SP): olive brown, medium dense, moist, fine to medium grained, layers of gravel.			
	45		16	23				5.1	
110									
						Well Graded Sand with Gravel (SW): brown, medium dense, moist, fine to coarse grained, silty sand and sandy silt inclusion.			
	50		17	34			117	4.7	
105									
	55		18	18		-- layer of sandy lean clay.		19.0	
						Sand with Silt (SP-SM): yellowish brown, medium dense, moist, fine to medium grained.		5.1	
100									
						Lean Clay (CL): yellowish brown, stiff, very moist.			
	60		19	18			113	17.9	WA (84% fines)
95									
	65		20	8				17.7	
90									
	70		21	10			101	25.6	
						Silty Sand (SM): yellowish brown, loose, moist, lense of clayey sand.			
85									

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PROJECT NO. 103567

ARTIC
S. Douglass Road and Katella Avenue
Anaheim, California

LOG OF BORING B-5

PLATE
A-6b

Legend To Logs On Plate A-1

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Elevation (feet)	Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION *(Continued From Previous Page)*	Dry Density (pcf)	Moisture Content (%)	Additional Tests
80			22	18		**Silty Sand (SM):** yellowish brown, loose, moist, lense of clayey sand. *(continued)*		14.0	
	80		23	43		**Sand with Silt and Gravel (SP-SM):** olive yellow, dense, moist, fine to medium grained, some fine gravel.	107	6.9	
						Total depth: 81.5 feet. Groundwater not encountered. Boring backfilled with bentonite slurry and capped with quickset concrete.			

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LOG OF BORING B-5

PLATE
A-6c

Legend To Logs On Plate A-1

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Date Drilled:	9/25/09	Water Depth:	25 feet
Drilled By:	Cal Pac Drilling	Date Measured:	9/25/09
Drilling Method:	Hollow Stem Auger	Elevation:	159 feet (approx.)
Logged By:	F. J.	Datum:	NAVD 88

Elevation (feet)	Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
						Artificial Fill:			
			1			**Pavement:** approx 2.75 inches of asphalt over 5 inches of base			
			2			**Poorly Graded Sand (SP):** pink, dry, fine to coarse grained.		2.6	
			3						
155	5		4						
			5	8		-- layer of sand with silt, mottled brown, lumps of lean clay. -- loose, yellowish brown, moist.	101	9.2	
						Alluvium:			
150			6	7		**Poorly Graded Sand (SP):** pink, loose to medium dense, dry, fine to medium grained.		4.5	
	10		7	31		-- layer of lean clay with sand -- layer of sand with silt, mottled brown, moist, fine to coarse sand.	118	7.2	
145			8	19			108	2.5	
	15		9	8		-- loose		7.0	
140						**Silty Sand (SM):** brown, medium dense, moist, fine to medium sand.			
	20					-- coarse gravel			
			10	27		-- darker, silty sand inclusions.	117	11.7	WA (20%)
135						**Sandy Silt (ML):** olive brown, soft, wet, layers of lean clay.			
	▼25		11	5				26.0	
130						**Clayey Sand (SC):** yellowish brown, medium dense, moist, fine to coarse sand, lenses and layers of lean clay.			
	30		12	14		-- layer of gravel	124	14.0	WA (38%)
125						**Poorly Graded Gravel with Sand (GP):** olive brown, very dense, moist.			

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Anaheim, California

LOG OF BORING W-1

PLATE

A-7a

Drafted By: Reviewed By:

Legend To Logs On Plate A-1

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Elevation (feet)	Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION *(Continued From Previous Page)*	Dry Density (pcf)	Moisture Content (%)	Additional Tests
			13	72		**Poorly Graded Gravel with Sand (GP):** olive brown, very dense, moist. *(continued)*		4.3	
120	40		14	56		**Poorly Graded Sand (SP):** olive brown, dense, moist, trace iron oxide staining.		5.3	
115	45		15	13		**Lean Clay with Sand (CL):** yellowish brown, stiff, moist. -- layer of silty sand, gray to olive gray, moist, fine to coarse grained.		19.2 8.6	
110	50		16	14		**Sandy Silt (ML):** yellowish brown, stiff, moist.	106	19.8	
105	55		17	15		-- wet, trace gravel.		21.6	
100	60		18	17		-- moist, some fine sand.	105	21.9	

Total depth: 61.5 feet.
Seepage at approximately 25 feet.
Two inch well constructed.

GEOTECH DB 103567 ARTIC WELLS.GPJ KA_RDLND.GDT 10/25/09

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Anaheim, California

LOG OF BORING W-1

PLATE

A-7b

Legend To Logs On Plate A-1
Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Date Drilled:	9/23/09	Water Depth:	None
Drilled By:	Cal Pac Drilling	Date Measured:	9/23/09
Drilling Method:	Hollow Stem Auger	Elevation:	156 feet (approx.)
Logged By:	F. J.	Datum:	NAVD 88

Elevation (feet)	Depth	Sample Number	Blows per Foot	SOIL DESCRIPTION AND CLASSIFICATION	Dry Density (pcf)	Moisture Content (%)	Additional Tests
				Artificial Fill:			
155		1		**Pavement:** approx 3 inches of asphalt over 6.25 inches of base			
		2		**Poorly Graded Sand with Silt (SP-SM):** brown, moist -- lumps of sandy clay		8.4	
		3					
	5	4					
150		5	47	-- olive gray to olive brown, dense, fine to medium grained.	121	7.2	
		6	28	-- medium dense, iron oxide staining, trace clay nodules,			
145	10	7	24	-- layer of sand with silt, sandy lean clay, olive brown.	114	7.2	
				Alluvium:			
		8	13	**Poorly Graded Sand (SP):** gray to pink, medium dense, dry, fine to medium grained.		2.8	
140	15	9	12		92	3.4	
135	20	10	20	-- layer of silty sand, moist. -- olive brown, sandy clay intrusions.	113	12.5	
				Silty Sand (SM): olive gray, loose, moist, fine to medium grained.			
130	25	11	7			12.1	WA (33%)
				Sandy Silt (ML): yellowish brown, medium stiff, moist.			
125	30	12	10	**Lean Clay with Sand (CL):** yellowish brown, stiff, moist.	99	26.0	WA (64%)

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Anaheim, California

PROJECT NO. 103567

LOG OF BORING W-2

PLATE
A-8a

Drafted By: Reviewed By:

Legend To Logs On Plate A-1

Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

Elevation (feet)	Depth	Sample Type	Sample Number	Blows per Foot	Graphic Log	SOIL DESCRIPTION AND CLASSIFICATION *(Continued From Previous Page)*	Dry Density (pcf)	Moisture Content (%)	Additional Tests
120			13	10		**Lean Clay with Sand (CL):** yellowish brown, stiff, moist. *(continued)*		16.5	
115	40		14	42		**Well Graded Sand with Gravel (SW):** mottled olive brown and olive gray, medium dense, moist, fine to coarse grained, fine to coarse gravel.	120	7.2	
						Lean Clay with Sand (CL): yellowish brown, medium stiff, moist, layers of clayey sand and sandy silt.			
110	45		15	8		**Poorly Graded Gravel with Sand (GP):** olive brown, loose, fine to coarse sand.		19.5	
105	50		16	11		**Lean Clay with Sand (CL):** yellowish brown, medium stiff, moist.	101	24.7	

Total depth: 51.5 feet.
Groundwater not encountered.
Two inch well constructed.

GEOTECH DB 103567 ARTIC WELLS.GPJ KA RDLND.GDT 10/25/09

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ARTIC
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Anaheim, California

LOG OF BORING W-2

PLATE A-8b

Legend To Logs On Plate A-1
Note: The boundaries between soil types shown on the logs are approximate as the transition between different soil layers may be gradual.

SUMMARY

OF

CONE PENETRATION TEST DATA

1. INTRODUCTION

This report presents the results of a Cone Penetration Test (CPT) program carried out for the project located at E. Katella Avenue & Douglass Road in Anaheim, California. The work was performed by Kehoe Testing & Engineering (KTE) on September 23, 2009. The scope of work was performed as directed by Kleinfelder, Inc. personnel.

2. SUMMARY OF FIELD WORK

The fieldwork consisted of performing CPT soundings at seven 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	46	Refusal, hole open to 40 ft (dry)
CPT-2	94	Refusal, groundwater @ 20 ft
CPT-3	38	Refusal, hole open to 32 ft (dry)
CPT-4	103	Groundwater @ 96 ft
CPT-5	46	Refusal, hole open to 43 ft (dry)
CPT-6	44	Refusal, hole open to 30 ft (dry)
CPT-7	57	Refusal, hole open to 54 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)

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,

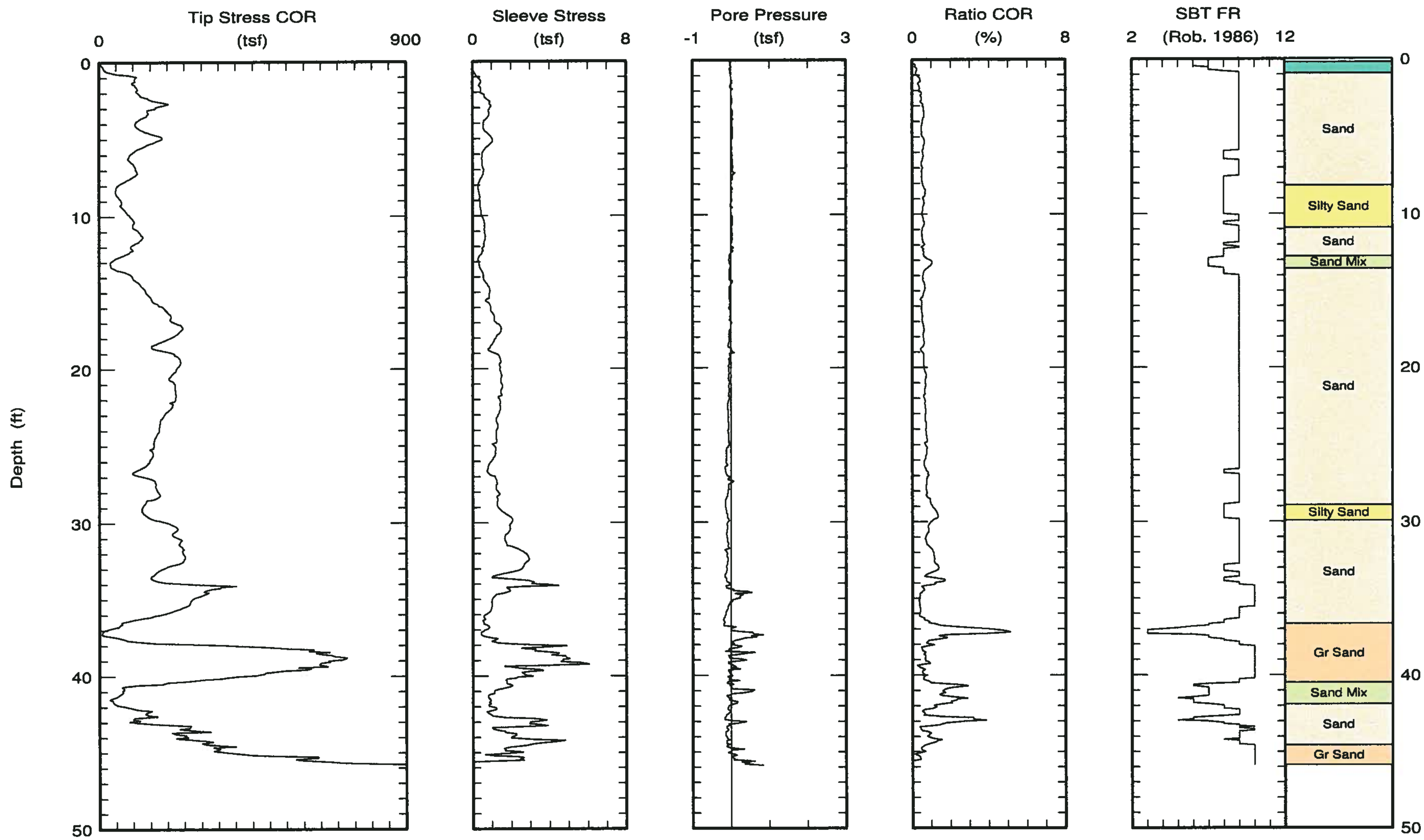
KEHOE TESTING & ENGINEERING

Richard W. Koester, Jr.
General Manager

09/28/09-jk-81-9955

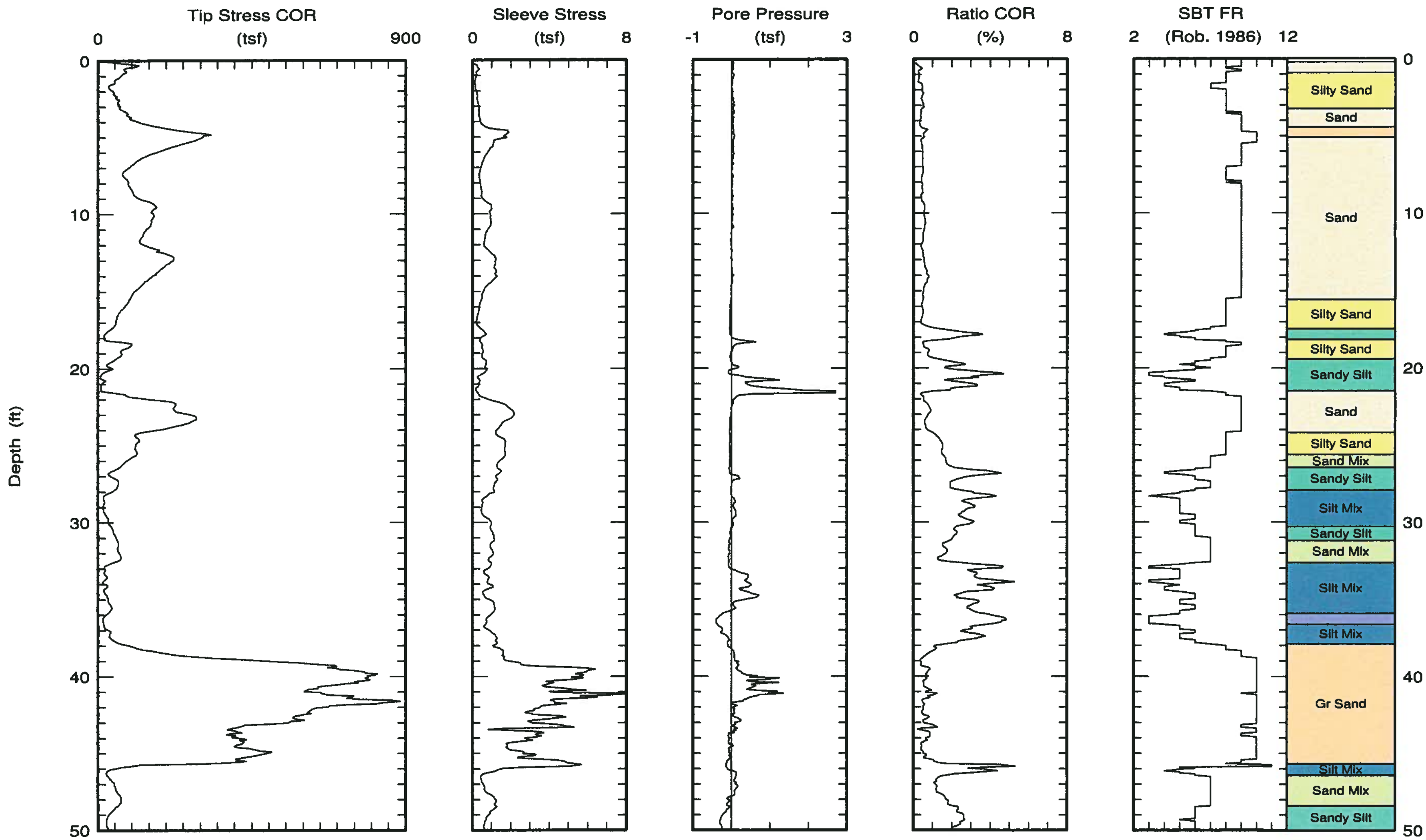
APPENDIX

Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 rich@kehoetesting.com skehoe@msn.com	CPT Data 30 ton rig	Date: 23/Sep/2009 Test ID: CPT-1 Project: Anaheim
	Customer: Kleinfelder Job Site: E. Katella Ave & Douglass Rd	



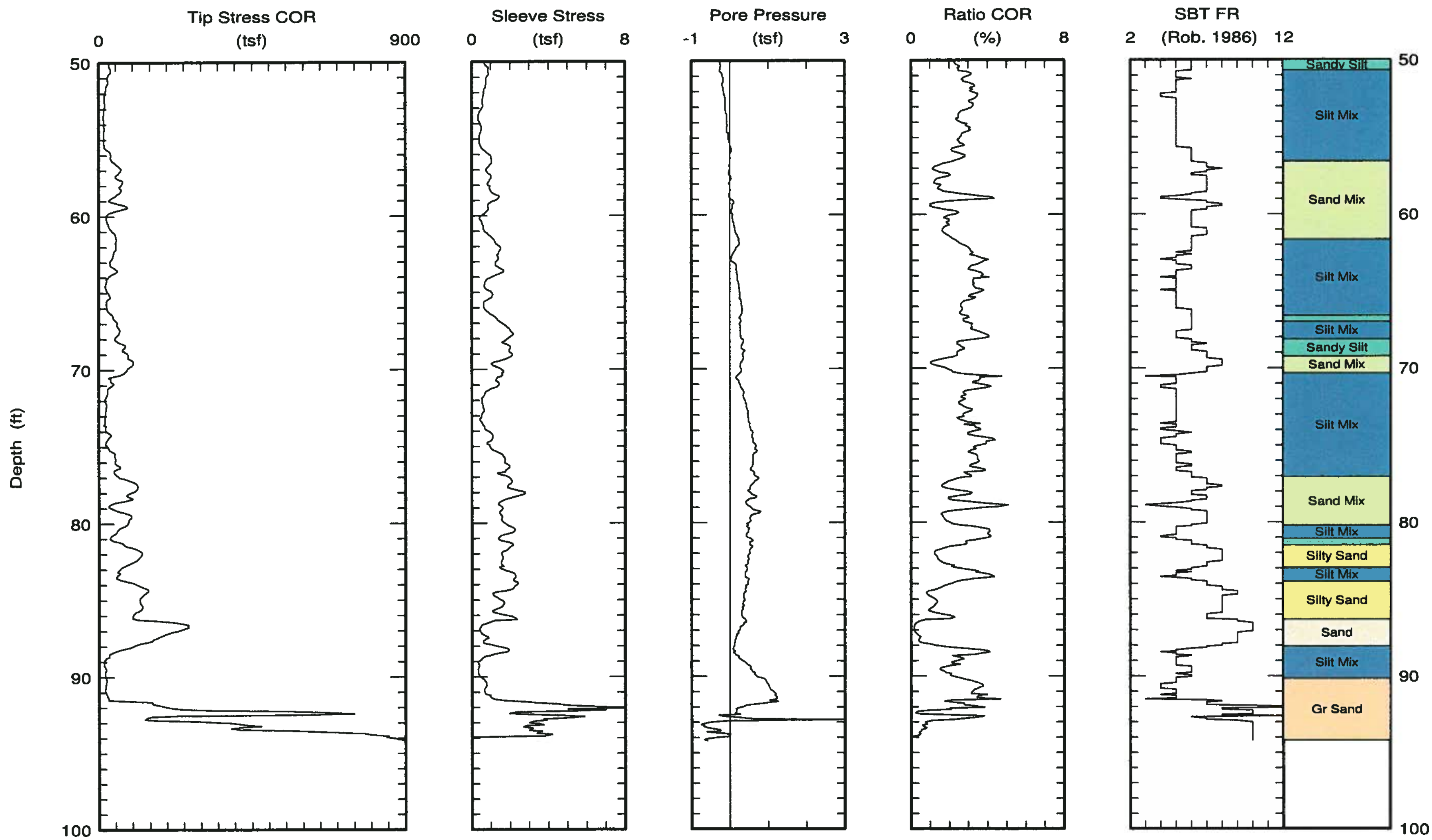
Maximum depth: 45.89 (ft)

Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 rich@kehoetesting.com skehoe@msn.com	CPT Data 30 ton rig	Date: 23/Sep/2009 Test ID: CPT-2 Project: Anaheim
	Customer: Kleinfelder Job Site: E. Katella Ave & Douglass Rd	



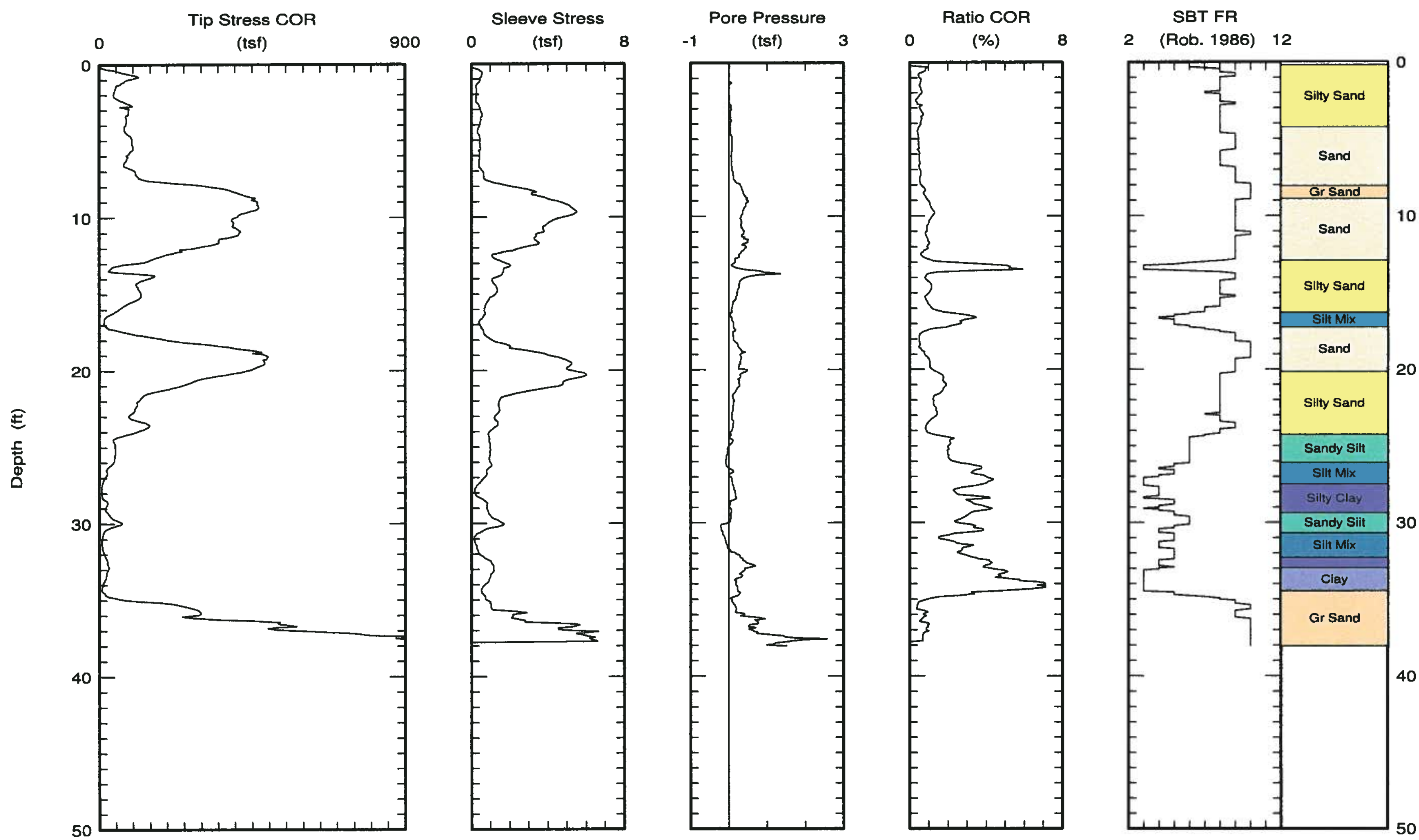
Maximum depth: 94.23 (ft)

Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 rich@kehoetesting.com skehoe@msn.com	CPT Data 30 ton rig	Date: 23/Sep/2009 Test ID: CPT-2 Project: Anaheim
	Customer: Kleinfelder Job Site: E. Katella Ave & Douglass Rd	



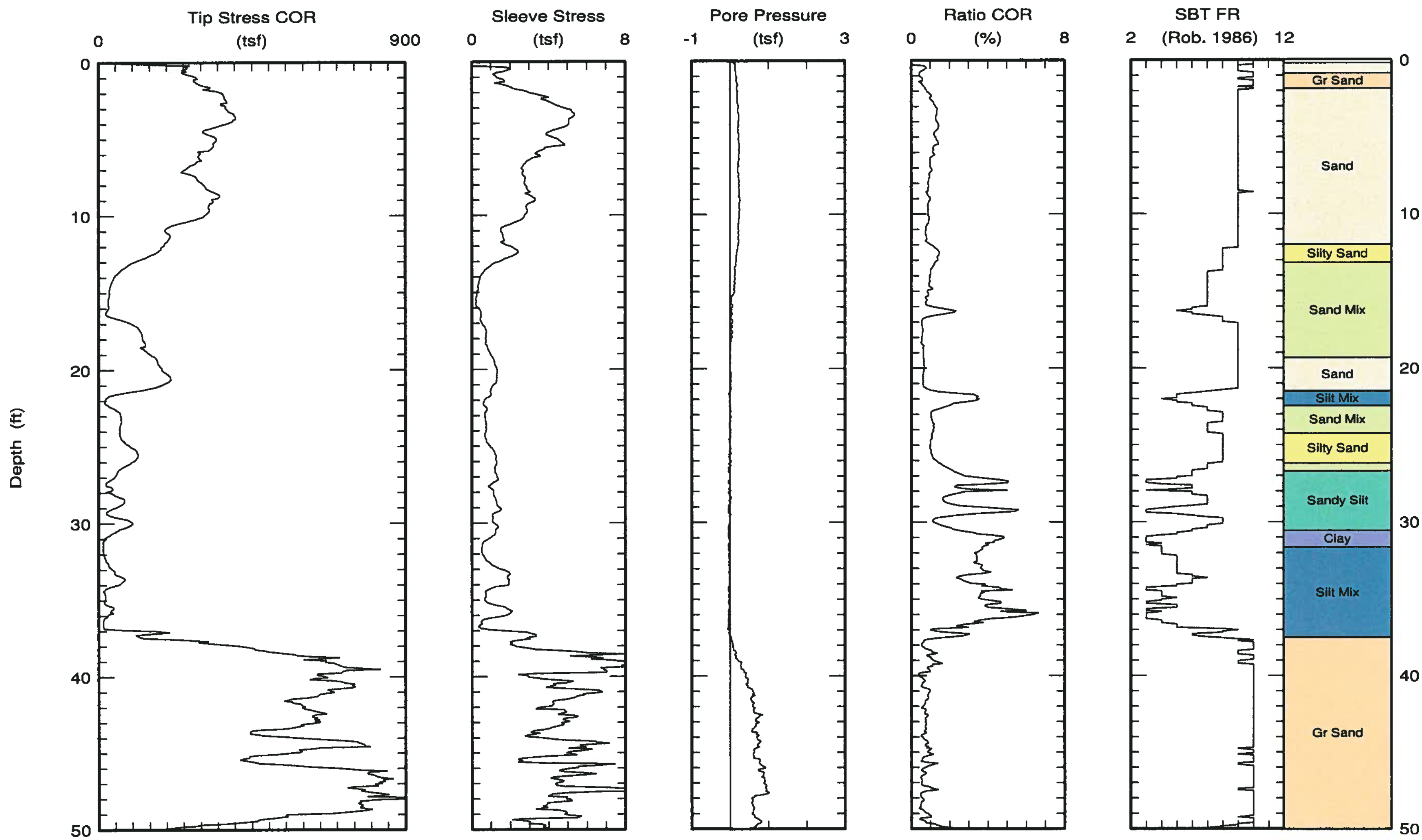
Maximum depth: 94.23 (ft)

Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 rich@kehoetesting.com skehoe@msn.com	CPT Data 30 ton rig	Date: 23/Sep/2009 Test ID: CPT-3 Project: Anaheim
	Customer: Kleinfelder Job Site: E. Katella Ave & Douglass Rd	



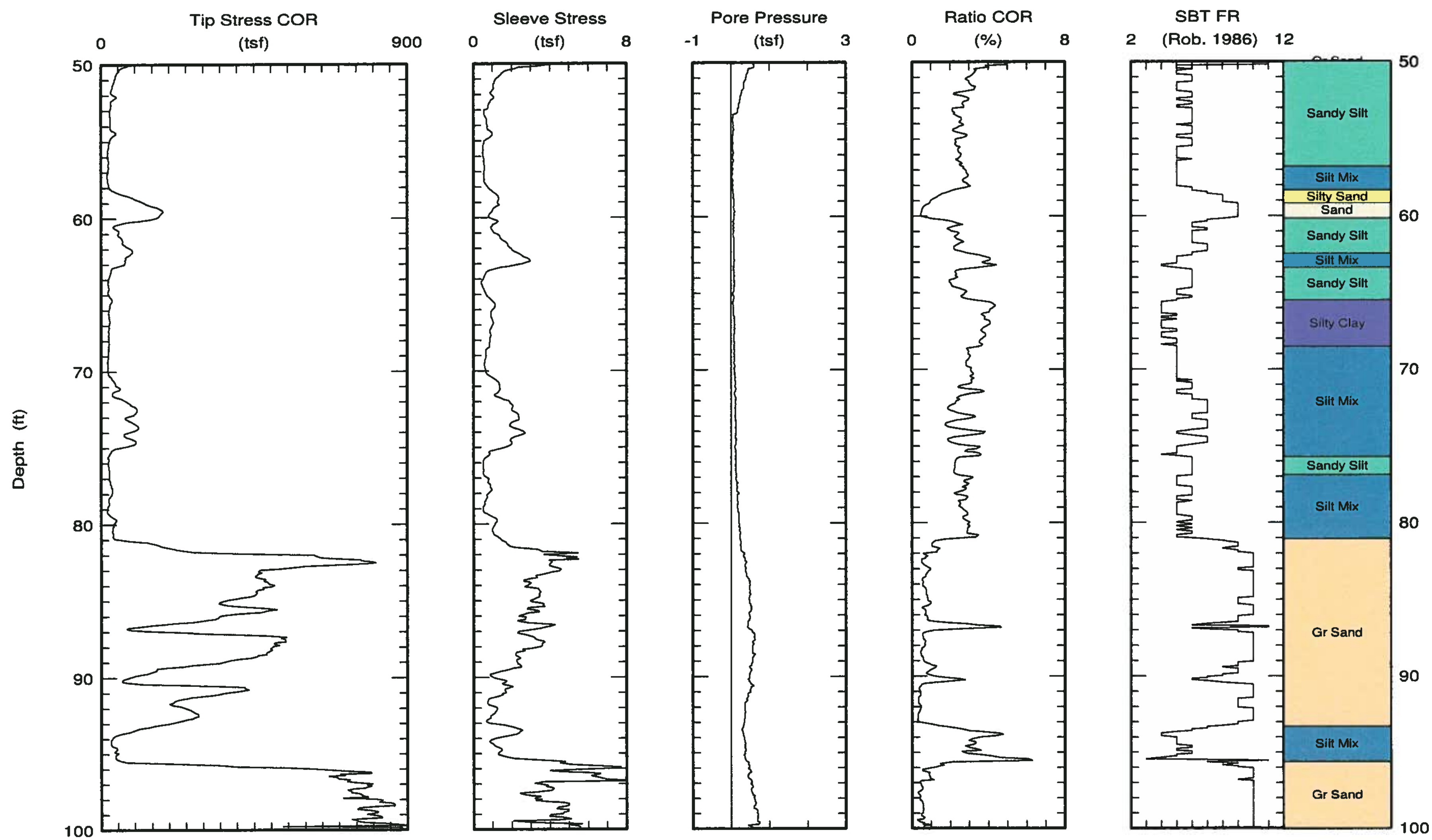
Maximum depth: 38.07 (ft)

Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 rich@kehoetesting.com skehoe@msn.com	CPT Data 30 ton rig	Date: 23/Sep/2009 Test ID: CPT-4 Project: Anaheim
	Customer: Kleinfelder Job Site: E. Katella Ave & Douglass Rd	



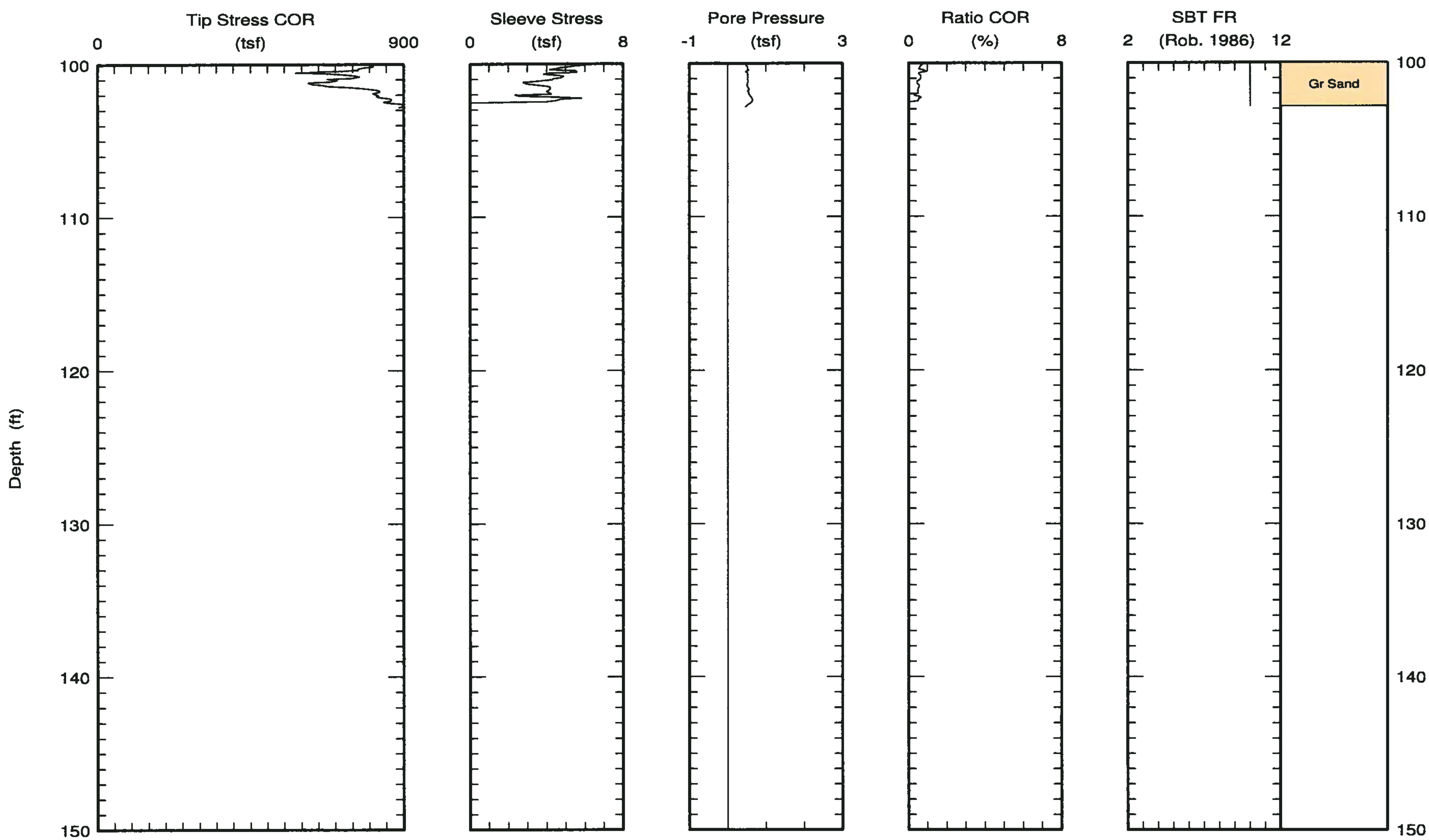
Maximum depth: 102.84 (ft)

Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 rich@kehoetesting.com skehoe@msn.com	CPT Data 30 ton rig	Date: 23/Sep/2009 Test ID: CPT-4 Project: Anaheim
	Customer: Kleinfelder Job Site: E. Katella Ave & Douglass Rd	



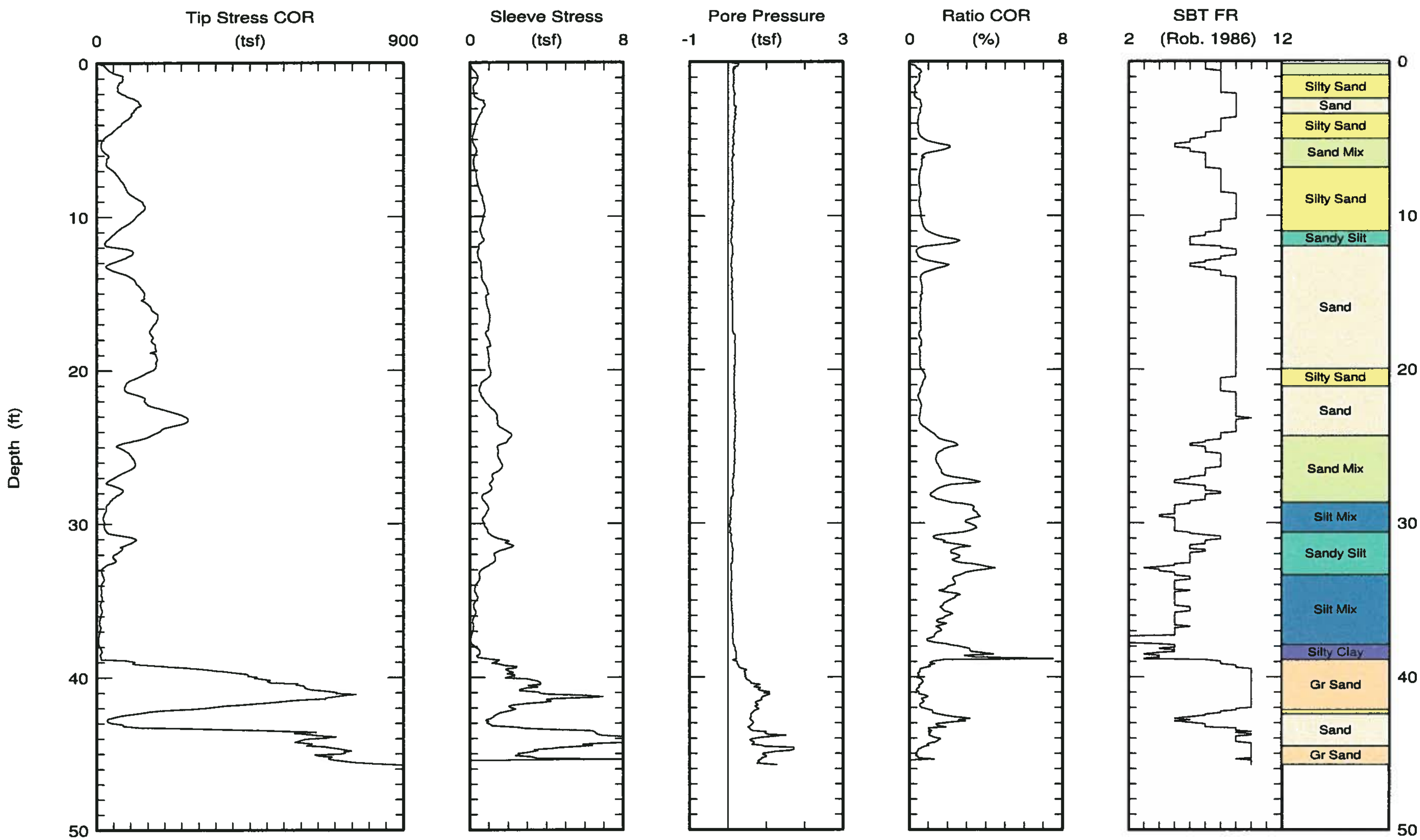
Maximum depth: 102.84 (ft)

Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 rich@kehoetesting.com skehoe@msn.com	CPT Data 30 ton rig	Date: 23/Sep/2009 Test ID: CPT-4 Project: Anaheim
	Customer: Kleinfelder Job Site: E. Katella Ave & Douglass Rd	



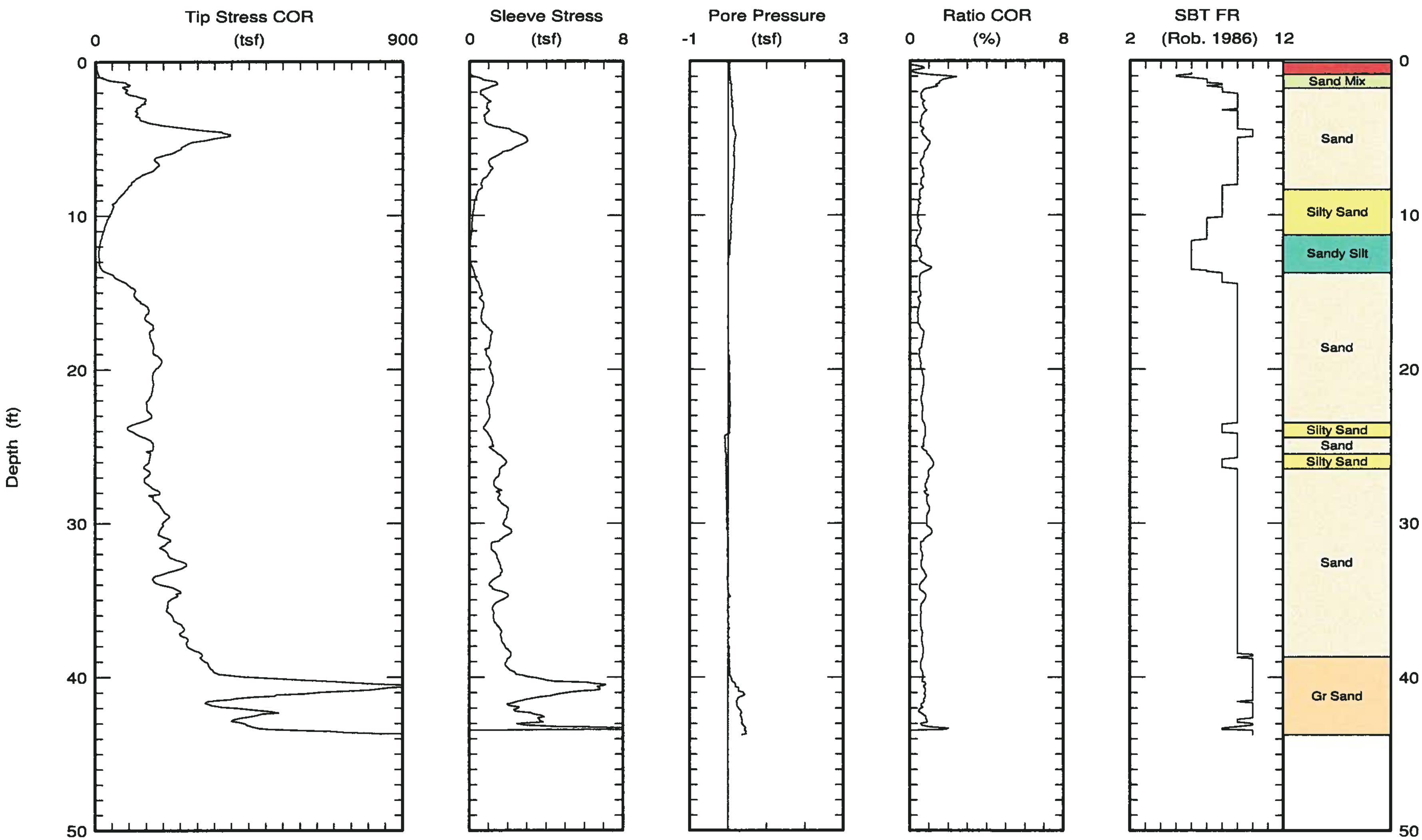
Maximum depth: 102.84 (ft)

Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 rich@kehoetesting.com skehoe@msn.com	CPT Data 30 ton rig	Date: 23/Sep/2009 Test ID: CPT-5 Project: Anaheim
	Customer: Kleinfelder Job Site: E. Katella Ave & Douglass Rd	



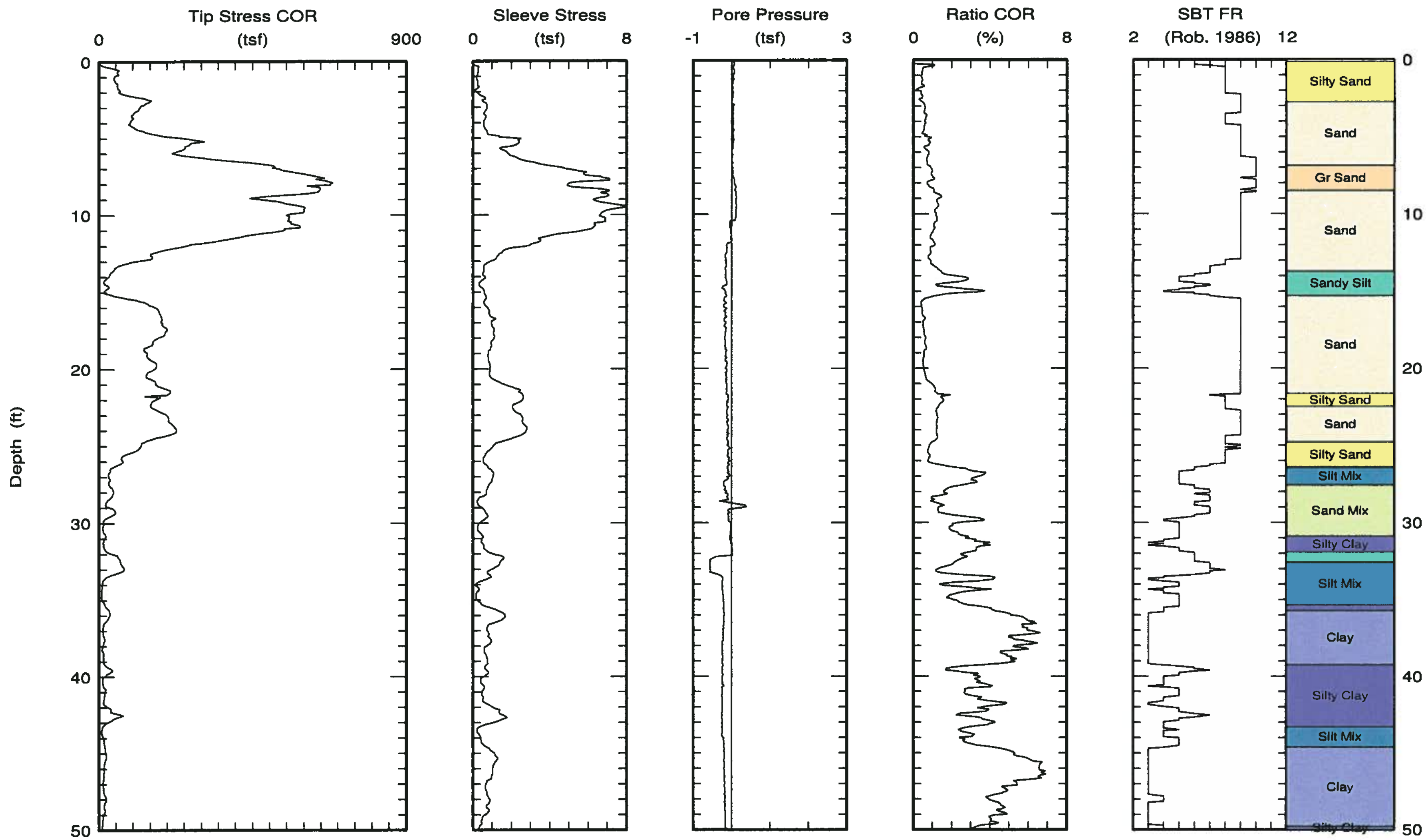
Maximum depth: 45.74 (ft)

Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 rich@kehoetesting.com skehoe@msn.com	CPT Data 30 ton rig	Date: 23/Sep/2009 Test ID: CPT-6 Project: Anaheim
	Customer: Kleinfelder Job Site: E. Katella Ave & Douglass Rd	



Maximum depth: 43.74 (ft)

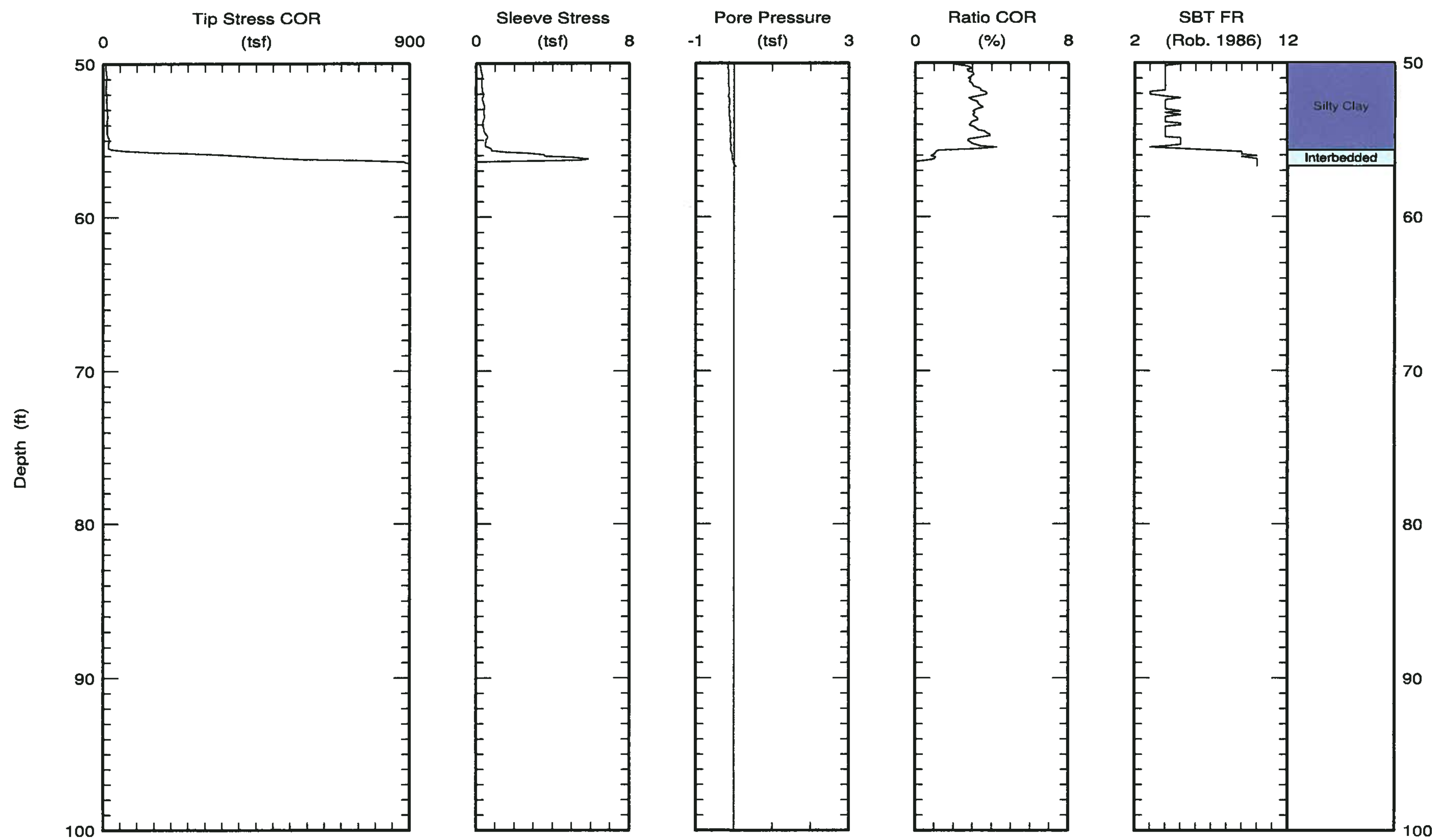
Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 rich@kehoetesting.com skehoe@msn.com	CPT Data 30 ton rig	Date: 23/Sep/2009 Test ID: CPT-7 Project: Anaheim
	Customer: Kleinfelder Job Site: E. Katella Ave & Douglass Rd	



Maximum depth: 56.72 (ft)

Page 1 of 2

Kehoe Testing & Engineering Office: (714) 901-7270 Fax: (714) 901-7289 rich@kehoetesting.com skehoe@msn.com	CPT Data 30 ton rig	Date: 23/Sep/2009 Test ID: CPT-7 Project: Anaheim
	Customer: Kleinfelder Job Site: E. Katella Ave & Douglass Rd	

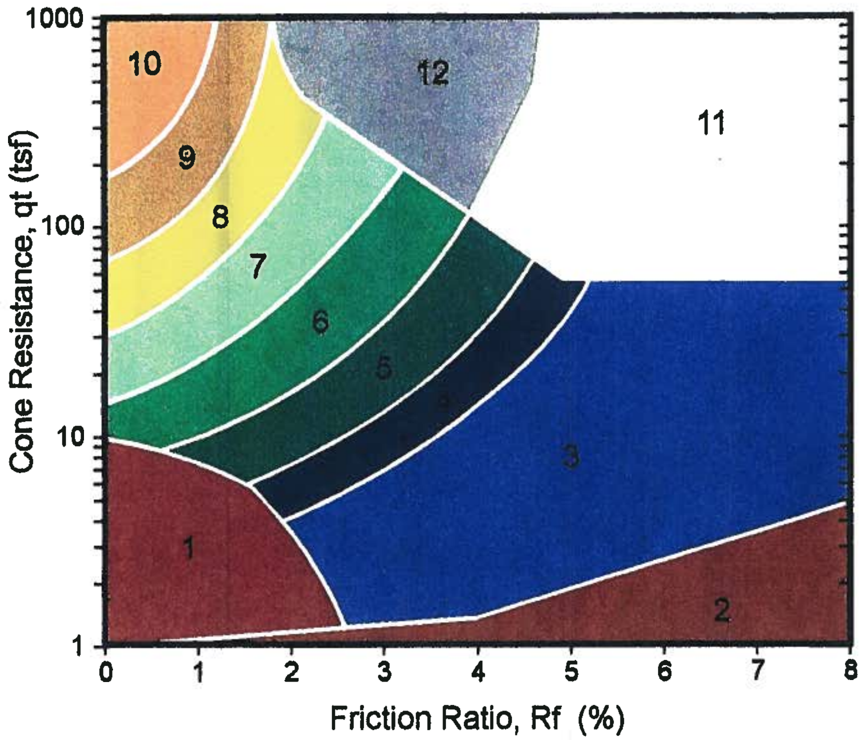


Maximum depth: 56.72 (ft)

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

APPENDIX B
LABORATORY TESTING

APPENDIX B
LABORATORY TESTING

GENERAL

Laboratory tests were performed on selected, representative samples as an aid in classifying the soils and to evaluate physical properties of the soils that may affect foundation design and construction procedures. The tests were performed in general conformance with the current ASTM or California Department of Transportation (Caltrans) standards. A description of the laboratory-testing program is presented below.

MOISTURE AND UNIT WEIGHT

Moisture content and dry unit weight tests were performed on a number of samples recovered from the borings. Moisture contents were determined in general accordance with ASTM Test Method D 2216; dry unit weight was calculated using the entire weight of the samples collected. Results of these tests are presented on the logs of borings in Appendix A.

WASH SIEVE

The percent passing the No. 200 sieve of selected soil samples was performed by wash sieving in accordance with ASTM Standard Test Method D1140. The results of the tests are presented on the boring logs in Appendix A.

ATTERBERG LIMITS

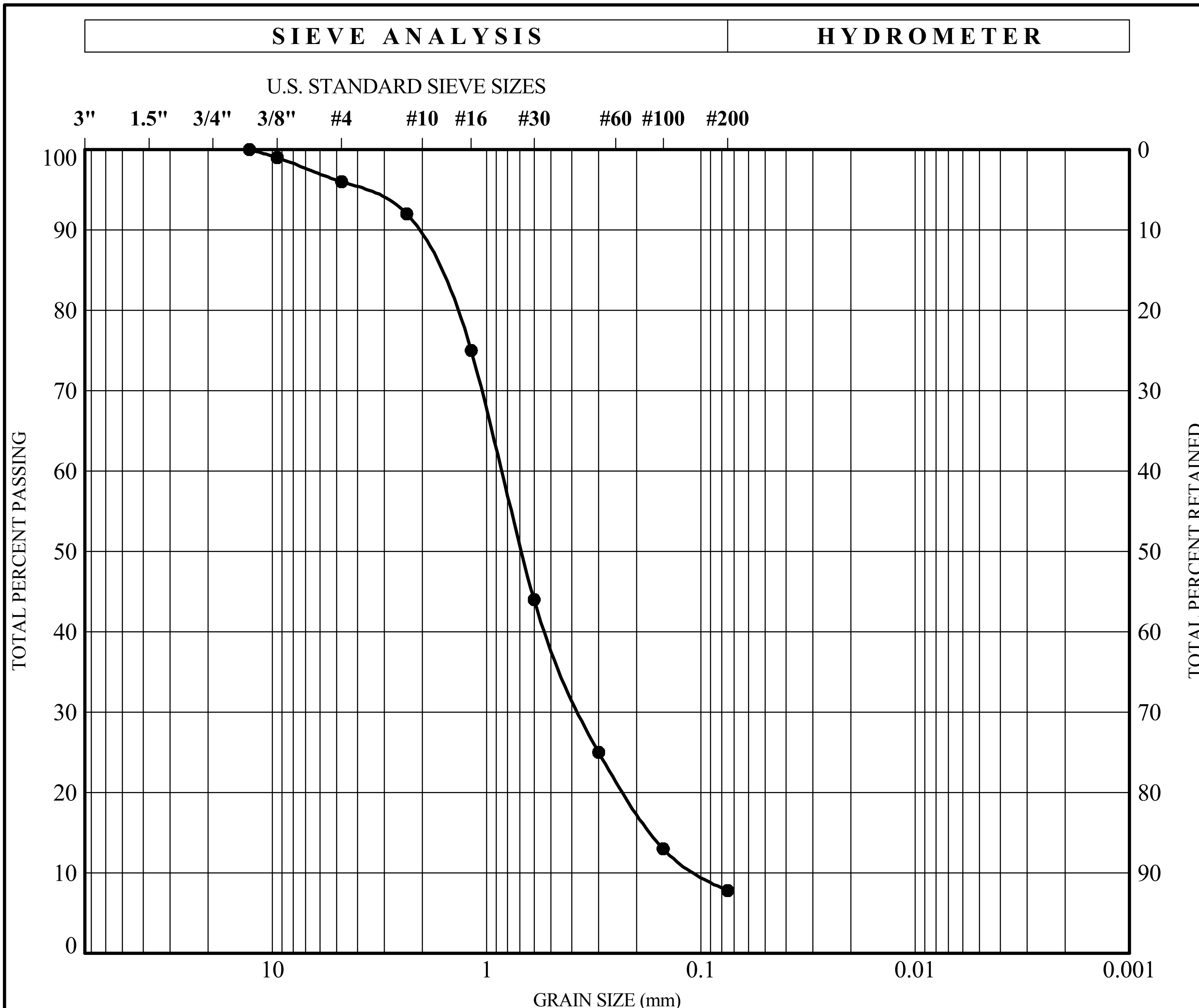
Two Atterberg limits tests were performed on soil samples to aid in classification and to evaluate the plasticity characteristics of the materials. The testing was performed in general accordance with ASTM Test Method D4318. The test results are presented on the logs of Boring B-2 and B-3 in Appendix A.

CONSOLIDATION TESTS

One consolidation test was performed on a relatively undisturbed sample of Boring B-2 in accordance with ASTM D2435. The tests was performed on 1.0-inch-high and 2.42-inch diameter sample. After trimming the ends, the sample was placed in the consolidometer and initial reading was recorded. The sample was incrementally loaded and submerged with water at a pressure of 5 ksf. The test results are presented on Plates B-3, Consolidation Test.

SOIL CORROSIVITY TESTS

A series of chemical tests were performed on three selected samples of the near-surface soils to estimate pH, resistivity and sulfate and chloride contents. Test results may be used by a qualified corrosion engineer to evaluate the general corrosion potential with respect to construction materials. The tests were performed by Enviro-Chem, Inc. of Pomona, California. The results of the tests are presented in the Table 5 in the body of the text.



GRAVEL		SAND			SILT	CLAY
coarse	fine	coarse	medium	fine		

Symbol	Boring	Depth (ft)	Description	Classification
●	B-5	3	Poorly Graded Sand with Silt	SP-SM

GRAIN SIZE 103567 ARTIC BORINGS, WELLS, CPTS.GPJ KA_RDLND.GDT 10/25/09

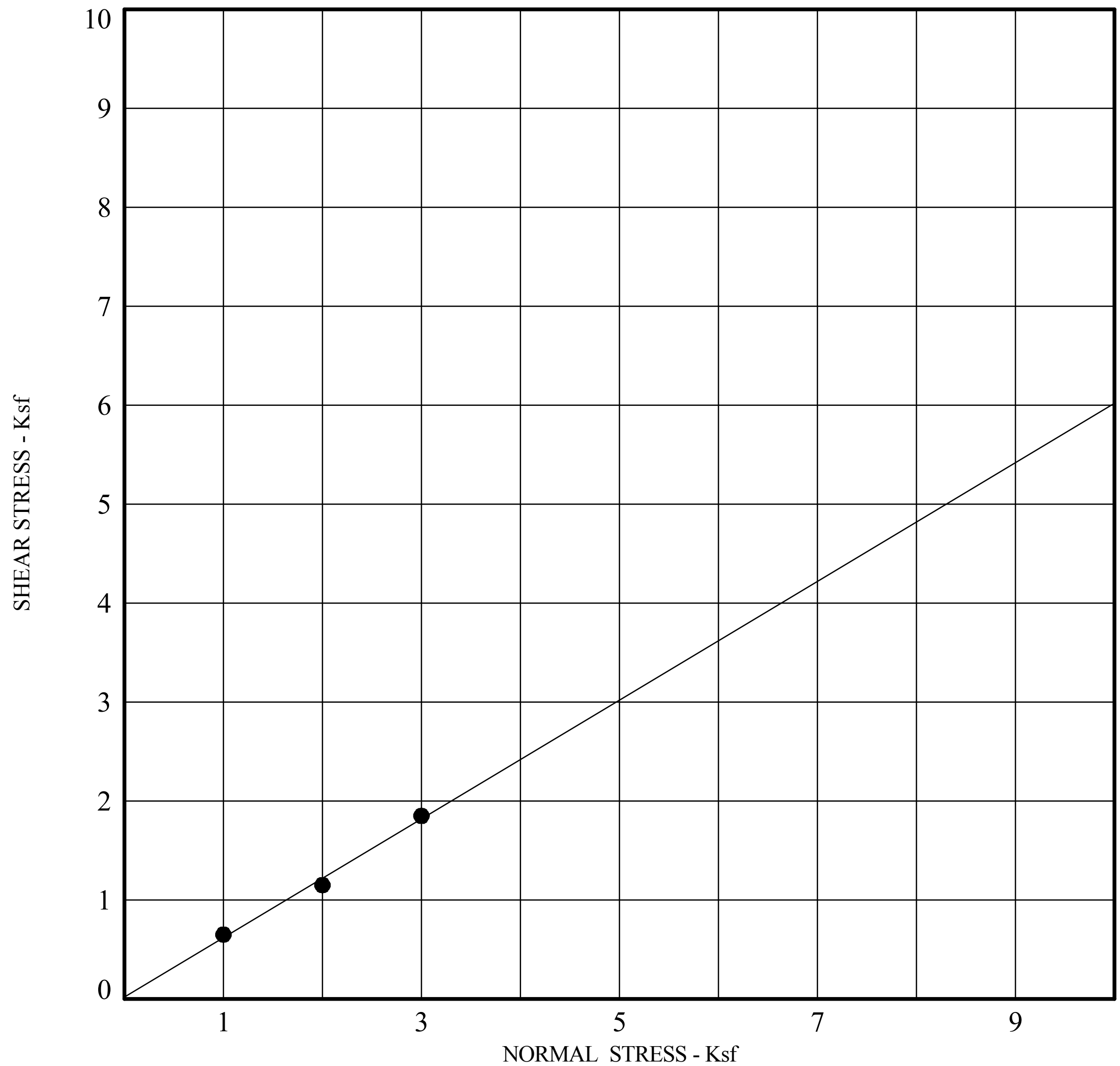
ARTIC
S. Douglass Road and Katella Avenue
Anaheim, California

PROJECT NO. 103567

GRAIN SIZE DISTRIBUTION

PLATE

B-1



Boring	**B-2**
Depth (ft)	**11**
Friction Angle - deg	**31**
Cohesion (Ksf)	**0.02**
Moisture Content (%)	**107**
Dry Density (pcf)	**4.9**
Description	**Poorly Graded Sand**
Classification	**SP**

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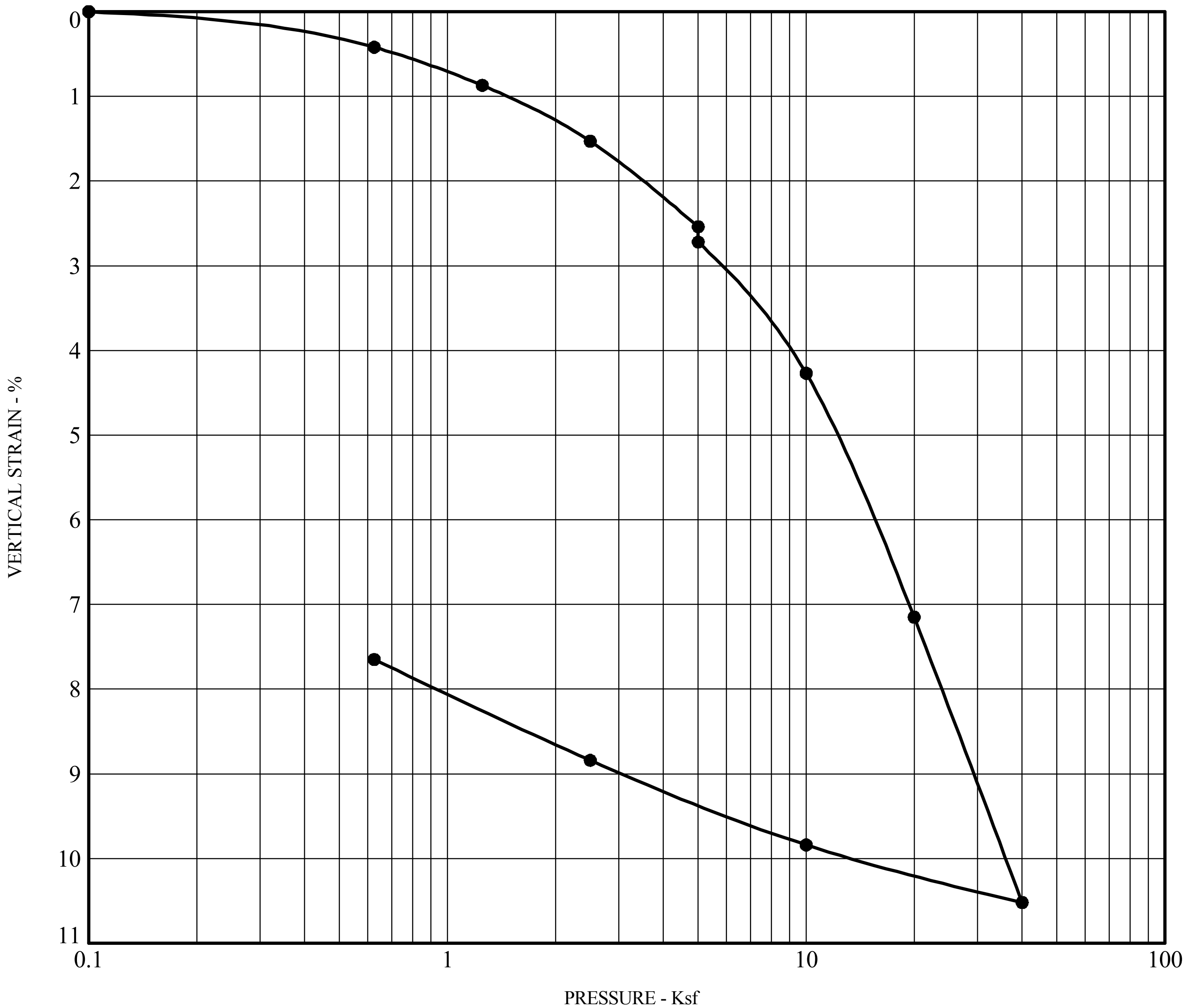
PLATE

B-2

PROJECT NO. 103567

DIRECT SHEAR TEST

DIRECT SHEAR 103567 ARTIC BORINGS, WELLS, CPTS.GPJ KA RDLND.GDT 10/25/09



Boring	**B-2**
Depth (feet)	**51**
Moisture Content (%)	**20.5**
Dry Density (pcf)	**105**
Description	**Lean Clay with Sand**
Classification	**CL**
Compression Ratio	
Recompression Ratio	

ARTIC
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Anaheim, California

PLATE

B-3

PROJECT NO. 103567

CONSOLIDATION TEST

8 Pasteur, Suite 190
Irvine, CA
92618
p| 949.727.4466
f| 949.727.9242
kleinfelder.com

June 3, 2009
Project No. 103567

Jones and Stokes
1 Ada, Suite 100
Irvine, California 92618

Attention: Ms. Donna McCormick
Principal

Subject: DRAFT Technical Memorandum
Preliminary Geotechnical Data Report
Proposed ARTIC – Phase 1
Anaheim, California

Dear Ms. McCormick:

Kleinfelder West, Inc. (Kleinfelder) is pleased to present this draft technical memorandum summarizing our findings related to geologic hazards and hydrogeologic conditions at the proposed Anaheim Regional Transportation Intermodal Center (ARTIC) - Phase 1 project site. The scope of our services was presented in our proposal titled, "Proposal for Geotechnical and Environmental Services, 30 Percent Design Level Submittal, Proposed ARTIC – Phase 1, Anaheim, California," dated February 25, 2009 (Proposal No. IRV9P031). This memorandum summarizes the work performed, data acquired, and our findings and conclusions.

INTRODUCTION

Kleinfelder understands that the Orange County Transportation Authority (OCTA) and the City of Anaheim plan to develop a major transit facility, known as the Anaheim Regional Transportation Intermodal Center (ARTIC). This proposed facility will serve Metrolink, Amtrak, fixed-route buses, and serve as a regional terminal for the future California High Speed Train. The ARTIC facility will be located southeast of the intersection of Katella Avenue and Douglass Road; bounded by the Santa Ana River, Los Angeles to San Diego (LOSSAN) railroad corridor, Douglass Road and Katella Avenue (see Plate 1, Site Vicinity Map).

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In addition to replacing the existing Anaheim station, the proposed construction for the ARTIC project site will include replacement of the existing railroad bridge crossing over Douglass Road, lowering and widening of Douglass Road, and modifying the existing crash walls to the support columns/foundations beneath State Route 57 (SR-57). Also, we understand that the proposed ARTIC development may include parking structure with two subterranean levels.

SCOPE OF SERVICES

The scope of our services consisted of an evaluation of the potential geologic and seismic hazards, which may affect the project site. The evaluation included analyzing for expected ground shaking, determining the site's potential exposure to fault rupture, landslides, liquefaction, lateral spreading and other geologic hazards including unstable soil conditions. An evaluation of existing and historical groundwater conditions was also performed. A site reconnaissance was performed; however, no fieldwork or subsurface investigation was conducted for this geologic hazards assessment. More specifically, Kleinfelder performed the following.

- Review of available geotechnical/geologic reports and maps collected from the U.S. Geological Survey (USGS), California Geological Survey (CGS) and other available sources, of the site and surrounding area. A list of the reports and documents utilized can be found in the Bibliography provided at the end of this report;

- Research at the City of Anaheim and the Orange County Water District (OCWD) offices to view geotechnical and hydrogeological reports, maps and as-built plans for existing structures, and any available preliminary studies for the site and vicinity that may be available;

- Review of historical, stereo-paired aerial photographs for the area available at Whittier College (Fairchild Collection) and Continental Aerial Services. The Fairchild Collection provided a detailed photographic coverage of the site from 1929 to 1961, while those provided by Continental Aerial Services spanned the years from 1952 to 1999. The aerial photographs reviewed are listed at the end of the Bibliography of this report;

- A field reconnaissance to observe the existing site conditions;

- A discussion of design and construction considerations that influence preliminary engineering; and

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- Preparation of this report which summarizes the work performed, data acquired, and our findings and conclusions for the proposed ARTIC development.

SITE CONDITION

The ARTIC project site includes approximately 13.5 acres located north of the existing LOSSAN corridor, and extending westward from the Santa Ana River to, and including, Douglas Road. Within the project area, Douglass Road is currently a 4-lane road, which crosses under the LOSSAN railroad corridor and SR-57 bridges. Current surface elvations of Douglass Road are approximately 165 feet (NAVD 88) near Katella Avenue and dropping to about 146 feet beneath the LOSSAN corridor bridge. The remainder of the project site is developed with several single-story office and maintenance buildings, and covered with asphalt for surface parking. The site ranges in elevation between approximately 165 feet in the northeast corner near Katella Avenue to 156 feet at the southern end near the LOSSAN corridor. The LOSSAN railroad corridor is about 10 feet above the site at an elevation of 166 feet. To the east of the site is the Santa Ana River with a bottom elevation estimated to be approximately 140 to 145 feet. The river is separated from the site by an improved embankment (levee or berm), which rises to an elevation of about 165 to 168 feet. The levee crest is paved and currently used as an Orange County bike path and maintenance access to the river.

SITE HISTORY

Historical aerial photography (see the Bibliography for a complete list) and vintage topographic maps (Plate II of Mendenhall, 1905) show that the project site and general vicinity was largely undeveloped or minimally developed agricultural land in the early 1900s. Although it appears that levee construction along the Santa Ana River had begun by the 1920s, the river's west bank adjacent to the project site was still in a natural condition and bank erosion and sloughing was apparent. In 1938, a year of heavy rains and extensive flooding throughout southern California, the site was stripped of all vegetation. In 1939, on the project site's western boundary, diagonal levees or berm-like structures (denoted as "1939 Levee" on Plate 2) are observed north and south of the railroad tracks (i.e., LOSSAN railroad corridor). The 1939 Levee is approximately 50 feet wide and appears to restrict the bank sloughing to its river-side, thus protecting orchards to the west. Collins Avenue crosses the river from the east, bisecting the site and turns northward to join a road that would become the present-day Douglass Road. Between 1955 and 1959 quarry excavation activities had begun on the project site between the railroad tracks and Collins Avenue-Douglass Road alignment. The quarry is open towards the Santa Ana River and its bottom appears to be slightly below river's bottom. The

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approximate extent of the quarry is shown on Plate 2. Also, during this time, bank erosion and sloughing of the project site, north of Collins Avenue, had migrated westward to the 1939 Levee. The Collins Avenue-Douglass Road alignment and the railroad tracks were largely unaffected by the quarry excavation or the sloughing of the river bank. By the late 1960s and early 1970s the Santa Ana River's current levee system has been constructed between the river and the project site. The project site behind the current river levee (including the quarry) is filled and, by the mid-1970s, the site has been developed and the current alignment of Douglass Road is completed. By the late 1970s, the SR-57 is completed.

GEOLOGY

Regional Geologic Setting

The ARTIC site is located in the southern part of the Los Angeles Basin within the Peninsular Ranges geomorphic province. The Peninsular Ranges geomorphic province extends 900 miles (1,450 kilometers) southward from the Los Angeles Basin to the tip of Baja California and is characterized by elongate northwest-trending mountain ranges separated by sediment-floored valleys (California Geological Survey, 2002). The most dominant structural features of the province are the northwest trending fault zones, most of which die out, merge with, or are terminated by the steep reverse faults at the southern margin of the Transverse Ranges geomorphic province.

East of the site are the northwest-trending Santa Ana Mountains, a large range which has been uplifted on its eastern side along the Whittier-Elsinore Fault Zone, producing a tilted, irregular highland that slopes westward toward the sea (Schoellhamer et al., 1981). The area south and west of the Santa Ana Mountains is generally characterized as a broad, complex, alluvial fan, which receives sediments from the Santa Ana River and its tributaries draining the Santa Ana Mountains and Puente Hills, and to a lesser extent the San Bernardino Mountains. These sediments are relatively flat-lying, unconsolidated to loosely consolidated clastic deposits that are approximately 1,700 feet thick beneath the site (Metropolitan Water District of Southern California, 2007; and Orange County Water District, 2004).

General Site Geology

The ARTIC site is located adjacent to the Santa Ana River, a braided stream system which has had significant flood control measures constructed along its course over the past 100 years. However, prior to flood control, deposition and erosion, primarily during flood events, contributed to the general geology of the project site and vicinity. The surficial deposits in the vicinity of the project area

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consist of alluvial fan material and alluvium deposited by the Santa Ana River (denoted as Qyfa on Plate 2, Geotechnical Map) over the last few thousand years. These unconsolidated alluvial sediments are generally composed of flat-lying, non-marine deposits of sand and a minor amount of silt. (Morton et al., 2004). South of Ball Road these sandy deposits become interbedded with clayey layers in the subsurface, generally at a depth of approximately 50 to 55 feet (OCWD, 2004; Southern California Regional Rail Authority [SCRRA], 1994). However, due to quarrying activities and bank sloughing, most of the project site is not underlain by alluvium, but rather an undetermined thickness of undocumented artificial fill (denoted Afu on Plate 2). The site was filled in to current grades during development of the property in the early 1970s. Although the bottom elevation of the fill is most likely equal to the river's elevation in the northern part of the site, in the southern part (quarry area) aerial photography indicate that fill depth may be about 5 to 10 feet deeper than the river's bottom elevation. The source for, or composition of, the fill material is not known. Underlying the undocumented fill throughout the project site is alluvial sand to silty sand. Plate 2 reflects this mapping and utilizes similar nomenclature (e.g., Qw and Qyf) presented by the USGS (Morton et al., 2004) and CGS (Greenwood and Pridmore, 2001).

GROUNDWATER

The ARTIC site is located in the forebay area of Orange County Basin (Metropolitan Water District of Southern California, 2007; DWR, 2004; and OCWD, 2004). The forebay is an area consisting of coarser, interconnected deposits that allows surface water to percolate down and ultimately recharge the County's principal aquifer about 800 feet deep (DWR, 2004). In other areas, the aquifer is under hydrostatic pressure and recharge from the surface is not possible. Most of the basin's recharge occurs north of Ball Road in lakes, ponds, pits and the river's main channel bottom. Here the alluvial deposits are sandier with few clay/silt layers to impede the downward movement of the recharge water. South of Ball Road clay layers become present and are interbedded with the sandy deposits. The clay layers are laterally discontinuous, thereby slowing, but not restricting, recharge from the surface. Adjacent to the site, sand levees are constructed in the bottom of the Santa Ana River to capture runoff and allow it to percolate into the groundwater system (OCWD, 2004).

The nearest aquifer beneath the site is the Talbert aquifer and it extends to a depth of approximately 150 feet below the project area (Poland, 1956). Near the site, groundwater levels in the Talbert aquifer can fluctuate substantially depending on rainfall conditions or recharge activities in the river. In 1994, wet soil samples (indication of groundwater) were logged adjacent to the site and the LOSSAN railroad corridor at a depth of approximately 50 feet (SCRRA, 1994), and in 1999 groundwater was measured at a depth of about 34 feet near the

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intersection of Katella Avenue and Douglass Road (Coleman Geotechnical, 1999). In June 2006, OCWD mapped groundwater levels near the site at a depth of approximately 60 feet. However, in 2001 an evaluation of the historically shallowest groundwater levels was conducted by the CGS (Greenwood and Pridmore, 2001) for the area which included the site. They determined the highest historical groundwater to be approximately 20 feet deep for the project site. Although no site-specific groundwater data are available at this time, utilizing the depth of 20 feet reported by the CGS would appear to be the most prudent. A depth of 20 feet at the project site the groundwater elevation would be roughly equal to the bottom elevation of the Santa Ana River adjacent to the site.

Fluctuations of the groundwater level, localized zones of perched water, and soil moisture content should be anticipated during and following the rainy season. Irrigation of landscaped areas on or immediately adjacent to the site can also cause a fluctuation of local groundwater levels.

GEOLOGIC SITE HAZARDS

Geologic and seismic hazards are those that could impact the site due to the surrounding geologic and seismic conditions. Potential geologic/seismic hazards include phenomena that occur during an earthquake such as ground rupture, liquefaction, lateral spreading, lurching, landslides, settlement and expansive soils. The geologic and seismic hazards have been evaluated in terms of their potential impact on the proposed project.

The most significant geologic hazard to the project is the potential for moderate to strong ground shaking resulting from earthquakes generated on the faults within the seismically active southern California region. Active or potentially active surface faults are not known to exist on the site. An active fault is defined as one that has moved within Holocene time (about the last 11,000 years). However, for the purposes of the Alquist-Priolo Earthquake Fault Zoning Act (Act), an active fault is defined as a fault that has exhibited surface displacement within Holocene time (Bryant and Hart, 2007). A potentially active fault is defined by the State as a fault with a history of movement within Pleistocene time (between 11,000 and 1.8 million years ago). These active and potentially active faults are capable of producing potentially damaging seismic shaking at the site. It is anticipated that the project site will periodically experience ground acceleration as the result of earthquakes. Active faults without surface expression (buried faults) and other potentially active seismic sources which are capable of generating earthquakes are not currently zoned by the Act, and are known to be locally present under the region.

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Surface Fault Rupture

Primary ground rupture is ground deformation that occurs along the surface trace of the causative fault during an earthquake. No known active faults are mapped crossing the site, and the site is not located within a State of California, Alquist-Priolo Earthquake Fault Zone (Bryant and Hart, 2007), thus the potential for future surface fault rupture at the site is considered to be low. The closest mapped faults to the site include the Peralta-El Modeno, Puente Hill Blind Thrust, Whittier-Elsinore faults and several unnamed and buried faults to the south of the site. Table 1 summarizes the distances of the closest known faults. Further discussion follows.

Table 1
Summary of Closest Mapped Faults

Fault Name	Type	Distance, miles (km)	Magnitude, M_w
El Modeno	Reverse	2.3 (3.7)	6.5
Peralta	Reverse	3.6 (5.9)	6.5
Unnamed Buried (2)	Unknown	4.2 (6.7) and 4.9 (8.0)	Unknown
Puente Hills	Blind Thrust	5.3 (8.6)	7.1
Whittier	Strike Slip	8.5 (13.8)	6.8

The Peralta-El Modeno faults are located north and northeast of the project site. The Peralta fault outcrops along the southern edge of the Peralta Hills east of the Santa Ana River approximately 3.6 miles (5.9 kilometers) from the site (Morton et al., 2004). The Peralta fault is a reverse fault which dips north towards the Whittier fault and movement along it results in crustal shortening and uplift of the Peralta Hills (Dolan et al., 2001). The El Modeno fault could be a westward extension of the Peralta fault, but this is currently not known. The El Modeno fault is buried beneath the alluvium of the Santa Ana River and it's inferred location is about 2.3 miles (3.7 kilometers) north of the site. The CGS fault map by Jennings (1994) shows the buried El Modeno fault extending westward from Burrel Ridge to about the SR-57 freeway. Slip rates of the El Modeno and Peralta faults are not currently known; however the faults are considered potentially active capable of generating an M_w6.5 earthquake (Mualchin, 1996).

The Puente Hills Blind Thrust fault (M_w7.1 earthquake) extends approximately 40 kilometers from downtown Los Angeles to near Brea in northern Orange County, and passes approximately 5.3 miles (8.6 kilometers) from the site. This active fault consists of three segments, from west to east; the Los Angeles,

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Santa Fe Springs and the Coyote Hills segments. These segments shallowly dip northward toward the Puente Hills and thrusting motion along these faults have resulted in crustal shortening in the region. Slip on the three segments produced an anticlinal structure caused by the compression and folding. This has been observed in the Coyote Hills segment approximately 5.5 miles (9.1 kilometers) north-northwest of the site. Although the Puente Hills Blind Thrust is buried approximately 2 to 3 kilometers beneath the ground surface, significant seismic shaking can result from this buried fault. Displacement along a section of the Santa Fe Springs segment is believed to have caused the 1987 Whittier Narrows earthquake (M_w6.0), confirming the potential for this active fault system to cause significant seismic shaking in the Los Angeles Basin (Dolan et al., 2001; Shaw et al, 2002).

The Whittier fault is an extension of the Elsinore fault where the fault deviates from the normal northwesterly strike and turns more westward at the Santa Ana River (Morton et al., 2004). Movement along the Whittier Fault is predominantly right-lateral strike-slip at a rate of approximately 2 to 3 mm/year (Dolan et al., 2001). However, it is believed to have had some reverse movement historically causing uplift of the Puente Hills at about 0.5 mm/year (Dolan et al., 2001). The surface trace of the Whittier fault has been mapped by the State and designated as an Alquist-Priolo Earthquake Fault Zone (Bryant and Hart, 2007). The surface trace has been mapped approximately 8.5 miles (13.8 kilometers) north of the project site.

Two unnamed, buried faults are mapped to the southwest and south of the site, approximately 4.2 miles (6.7 kilometers) and 4.9 miles (8 kilometers), respectively. Both faults terminate within the Orange County Basin, however, the one to the south, is mapped trending towards the site before it ends about 4.9 miles away. No information regarding these faults is available except that they are buried beneath sediments, some older than 11,000 years (Morton et al., 2004).

Liquefaction and Lateral Spreading

Seismically induced soil liquefaction generally occurs in loose, saturated, cohesionless soil when pore pressures within the soil increase during ground shaking. The increase in pore pressure transforms the soil from a solid to a semi-liquid state. The primary factors affecting the liquefaction potential of a soil deposit are: 1) intensity and duration of earthquake shaking, 2) soil type and relative density, 3) overburden pressures, and 4) depth to groundwater. Soils most susceptible to liquefaction are clean, loose, uniformly graded, fine-grained sands, and non-plastic silts that are saturated. Silty sands have also been shown to be susceptible to liquefaction. These soils typically lose a portion or all of their shear strength and regain strength sometime after shaking stops. Soil

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movements (both vertical and lateral) have been observed under these conditions due to consolidation of the liquefied soils and the reduced shear resistance of slopes.

According to the State (California Division of Mines and Geology [CDMG], 1998), the site is located within a liquefaction hazard zone. An evaluation of the liquefaction potential at the project site is required and should be performed following the collection of site-specific information from the field exploration and laboratory testing program.

The potential for lateral spreading should also be evaluated along the site's eastern boundary with the Santa Ana River. Seismically induced lateral spreading involves primarily lateral movement of earth materials due to ground shaking. Liquefaction-induced lateral displacement usually occurs on gently sloping ground, and results in near-vertical cracks with predominantly horizontal movement of the soil mass involved towards a free face (i.e., the river's bank to the east). Estimating the magnitude of lateral spreading depends on the site's regional topography and continuity of the liquefiable layer(s); therefore, an accurate estimate of lateral spreading magnitude is complicated and should be completed at a site-specific level following subsurface exploration program.

Seismically-Induced Settlement and Differential Compaction

Seismically-induced settlement and differential compaction occurs when relatively soft or loose soils experience a reduction in volume (compaction) caused by strong ground motion. Soil conditions subject to these include unconsolidated soil or areas where weak soils of variable thickness overlie firm soil or bedrock. The type of materials that would be more likely to experience seismically-induced settlement and differential compaction are deposits of alluvium and loosely compacted man-made fill, both of which underlie most of the project site. Any structures built on such soils could be damaged during settlement. Due to the possible high ground shaking levels and the unknown thickness and composition of the undocumented fill the seismically-induced settlement and differential compaction hazard is considered high.

Lurching

Lurching is the relative displacement of adjacent land surfaces during an earthquake. As the seismic motion encounters a cliff, bluff, stream bank, or even a fill slope at nearly right angles it may cause displacement of the material in the unsupported direction. Lurching may also be caused by liquefaction of a zone beneath the otherwise intact surface. Visible evidence of lurching includes ground cracking and fissuring generally in a relatively parallel fashion to a stream

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bank or slope face. Due to the expected high ground motion, potential for lurching exists at the site, especially along the Santa Ana River bank.

Slope Failure

Landslides and other forms of mass wasting, including mud flows, debris flows, soil slips, and rock falls occur as soil or rock moves down slope under the influence of gravity. Landslides are frequently triggered by intense rainfall or seismic shaking. The site is not within a State or County designated hazard zone for landslides (CDMG, 1998). Although the project site is relatively flat, the risk of landslides and other forms of slope failure could occur along the bank of the Santa Ana River, or the foundation slope beneath the LOSSAN railroad corridor, thus impacting the proposed project.

Flooding and Inundation

Flooding and inundation occurs as a result of several factors in developed areas. These factors include: rainfall rates that exceed an area's ability to absorb or control the runoff; impounded water retained behind a flood control structure (upstream-inundation); failure of a flood control structure (downstream-inundation); seiches and tsunamis (earthquake induced). Flooding of the Santa Ana River has inundated the site numerous times over the past 175 years. Channelization and flood protection levees were constructed, and following the devastating 1938 flood, Prado Dam was constructed in to improve flood protection. As development of the inland empire proceeded, additional measures were soon needed. Currently, flood protection for the area is being improved with the Santa Ana River Mainstem Project. The project will increase the flood level protection along more than 75 miles of the Santa Ana River course within Orange, Riverside and San Bernardino Counties, and is scheduled to be completed by 2010.

Although the Santa Ana River Mainstem Project may reduce the risk of flood along the river, it may not prevent flood inundation at the site due to failure of the Prado Dam during an earthquake. An earthquake along the Chino Hills fault, which crosses beneath the dam near the spillway, could cause the dam to fail. A catastrophic failure of the dam with substantial water stored behind it could cause flooding at the site downstream. A flood inundation evaluation should be performed for the site during the next phase.

DESIGN AND CONSTRUCTION CONSIDERATIONS

Based on our review of readily-available geologic, geotechnical, and seismologic reports and publications covering the site and general vicinity, it is our professional opinion that the proposed project is geotechnically feasible. The

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primary geotechnical constraints that could have a significant impact to the cost of developing the site include: 1) the potential for seismically-induced settlement and lateral spreading due to liquefaction; 2) the presence of deep undocumented fill that was placed in the early 1970s; and 3) the potential for high groundwater potentially affecting the design and construction of subterranean structures. More detailed discussion of each potential geotechnically constraint is presented below.

Liquefaction Potential

The potential for seismically-induced settlement and lateral spreading due to liquefaction could have a significant impact to the ARTIC development. Depending on the severity of the liquefaction potential, ground improvement and/or alternative foundation systems, such as piles, may be necessary for the proposed structures. Current standard of practice dictates that seismic settlement greater than about 2 inches is excessive for a conventional spread footing foundation system. In addition, ground improvement along the river channel side of the site may be necessary to mitigate lateral spreading. The potential for liquefaction and its adverse affects, seismically-induced settlement and lateral spreading, will need to be evaluated in detail as part of the design-level geotechnical study for the ARTIC Development.

Undocumented Fill

An undetermined thickness of undocumented artificial fill is present at the site due to quarry activities in the late 1950s and infilling the site in the early 1970s. This material is mostly likely not suitable for support of settlement sensitive structures. Due to the anticipated depths of the undocumented fill, complete removal and recompaction may not be practical. Therefore, ground improvement and/or alternative foundation systems, such as piles, may be necessary for the proposed structures. The depth and composition of the undocumented fill, along with its adverse affects, will need to be evaluated in detail as part of the design-level geotechnical study for the ARTIC Development.

High Groundwater

Although the current groundwater levels beneath the site are likely below the historic high groundwater levels, fluctuations of the groundwater level, localized zones of perched water, and increased soil moisture content should be anticipated during and following the rainy season, especially since sand levees exist in the bottom of the Santa Ana River adjacent to the site to capture runoff and allow it to percolate into the subsurface. High groundwater will need to be considered when designing all subterranean walls and floor slabs that extend to

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and below groundwater. In addition, increased soil moisture contents and localized zones of perched water will need to be considered during construction.

LIMITATIONS

This technical memorandum has been prepared for the exclusive use of Jones and Stokes, OCTA, and their agents for specific application to the subject project. This work was performed in a manner consistent with that level of care and skill ordinarily exercised by other members of Kleinfelder's profession practicing in the same locality, under similar conditions and at the date the services are provided. Our conclusions, opinions and recommendations are based on a limited number of observations and data. It is possible that conditions could vary between or beyond the data evaluated or that others may develop different opinions based on the available data. Kleinfelder makes no other representation, guarantee or warranty, express or implied, regarding the services, communication (oral or written), report, opinion, or instrument of service provided.

The scope of services was based on the data collected, as described above. It should be recognized that definition and evaluation of subsurface conditions are difficult. Judgments leading to conclusions and recommendations are generally made with incomplete knowledge of the subsurface conditions present due to the limitations of data from field studies. The conclusions of this assessment are based on our background data research.

Kleinfelder offers various levels of investigative and engineering services to suit the varying needs of different clients. Although risk can never be eliminated, more detailed and extensive studies yield more information, which may help understand and manage the level of risk. Since detailed study and analysis involves greater expense, our clients participate in determining levels of service, which provide information for their purposes at acceptable levels of risk. The client and key members of the design team should discuss the issues covered in this memorandum with Kleinfelder, so that the issues are understood and applied in a manner consistent with the owner's budget, tolerance of risk and expectations for future performance and maintenance.

This report may be used only by the client and only for the purposes stated, within a reasonable time from its issuance, but in no event later than two (2) years from the date of the report. Land use, site conditions (both on site and off site) or other factors may change over time, and additional work may be required with the passage of time. Any party, other than the client who wishes to use this report shall notify Kleinfelder of such intended use. Based on the intended use of this report and the nature of the new project, Kleinfelder may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the client or anyone else will

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release Kleinfelder from any liability resulting from the use of this report by any unauthorized party and the client agrees to defend, indemnify, and hold harmless Kleinfelder from any claims or liability associated with such unauthorized use or non-compliance.

CLOSURE

We appreciate the opportunity to provide geotechnical engineering services to you on this project. If you have any questions regarding this report, or if we can be of further service, please do not hesitate to contact the undersigned.

Respectfully submitted,

KLEINFELDER WEST, INC.

Robert Lemmer, C.E.G., C.H.G. Senior Engineering Geologist	Brian E. Crystal, P.E., G.E. Geotechnical Group Manager

Attachments: Bibliography
Plate 1 - Site Location Map
Plate 2 – Geotechnical Map

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U.S. Geological Survey, 1965 (photorevised 1981), 7.5-minute Topographic map of the Anaheim, California Quadrangle, scale 1:24,000.

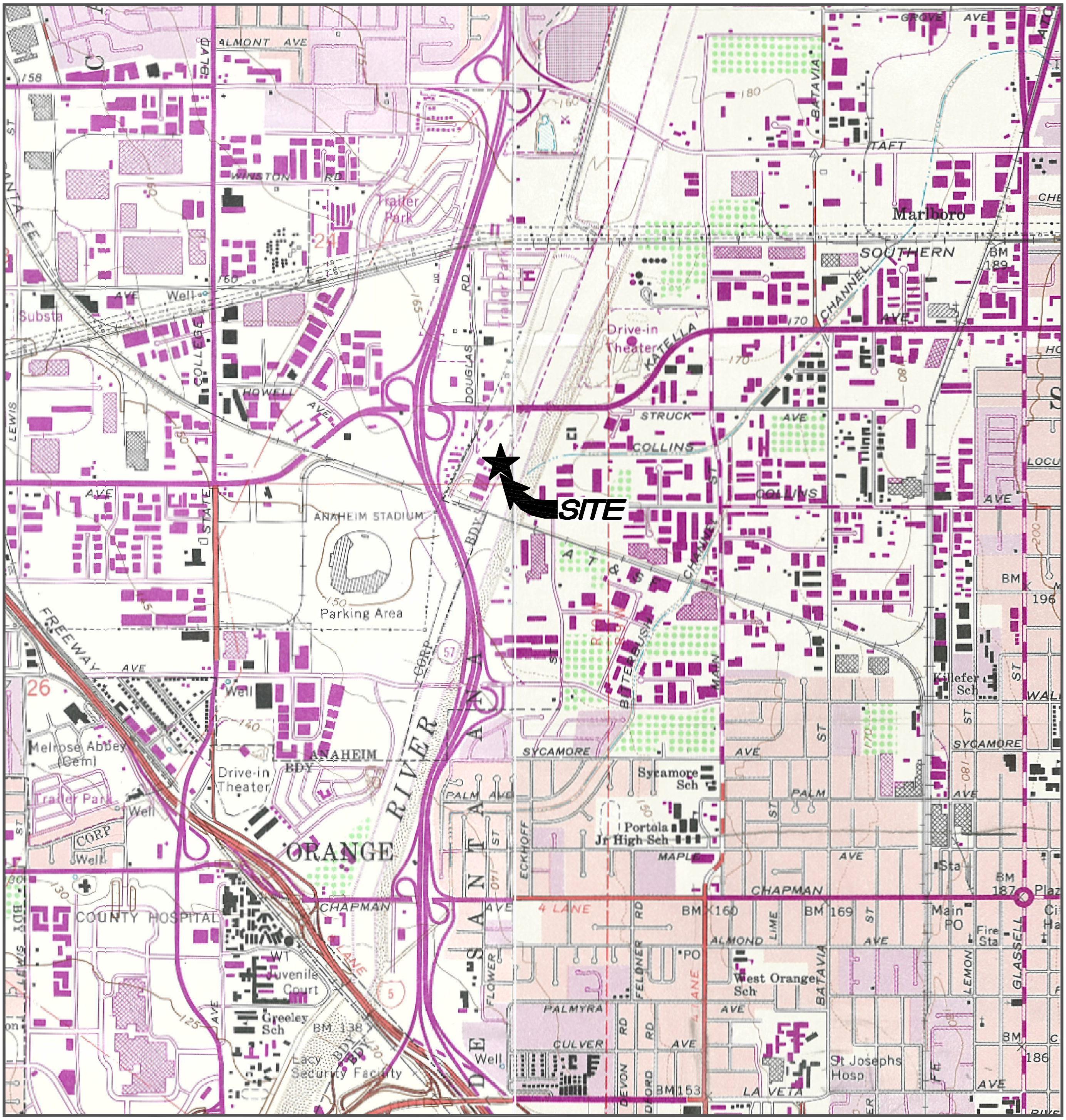
AERIAL PHOTOGRAPHS REVIEWED

Date	Type	Flight	Frames	Approximate Scale	Source
2-28-1929	B&W	C-287 #3	A1, A2; and B2, B3	1:18,000	Fairchild Aerial Collection
1931	B&W	C-1780	C-1	1:15,600	Fairchild Aerial Collection
3-4-1938	B&W	C-5029	66-68	1:32,000	Fairchild Aerial Collection
6-24-1939	B&W	C-5925	120-122	1:24,000	Fairchild Aerial Collection
6-17-1947	B&W	C-11351-7	54-56	1:24,000	Fairchild Aerial Collection
8-1947	B&W	C-113730A-11	155X-157X	1:7,200	Fairchild Aerial Collection
8-1947	B&W	C-113730A-12	102-104	1:7,200	Fairchild Aerial Collection
8-1947	B&W	C-113730A-14	4-6	1:7,200	Fairchild Aerial Collection
8-31-1947	B&W	C-113730D-14	48-50	1:14,400	Fairchild Aerial Collection
12-26-1952	B&W	5K	84-86	1:20,000	Continental Aerial Surveys
2-11-1953	B&W	C-18785-1	100	1:14,400	Fairchild Aerial Collection
5-2-1953	B&W	C-19400-V11-LA	1-33, 2-28	1:63,360	Fairchild Aerial Collection
3-7-1955	B&W	C-21678-2	23-25	1:18,000	Fairchild Aerial Collection
1-17-1958	B&W	C-23023-V11-ORA 5	82, 83	1:36,000	Fairchild Aerial Collection
3-25-1959	B&W	261-3-14	66-68	1:12,000	Continental Aerial Surveys
3-25-1959	B&W	261-3-15	110-112	1:12,000	Continental Aerial Surveys
6-3-1961	B&W	C-24129	10	1:24,000	Fairchild Aerial Collection

AERIAL PHOTOGRAPHS REVIEWED (Continued)

Date	Type	Flight	Frames	Approximate Scale	Source
3-1-1967	B&W	1	32, 33	1:24,000	Continental Aerial Collection
2-18-1970	B&W	61-6	270	1:48,000	Continental Aerial Surveys
10-29-1973	B&W	132-6	6-8	1:24,000	Continental Aerial Surveys
1-13-1975	B&W	157-7	14, 15	1:24,000	Continental Aerial Surveys
12-28-1976	B&W	181-7	12-14	1:24,000	Continental Aerial Surveys
12-10-1978	B&W	203-7	15, 16	1:24,000	Continental Aerial Surveys
2-25-1980	B&W	80033	75, 76	1:32,000	Continental Aerial Surveys
4-2-1983	B&W	218-7	13-15	1:24,000	Continental Aerial Surveys
1-9-1987	B&W	F	232, 233	1:34,300	Continental Aerial Surveys
1-29-1992	B&W	C-85-7	16, 17	1:25,800	Continental Aerial Surveys
6-9-1993	B&W	C-93-13	176, 177	1:25,800	Continental Aerial Surveys
1-29-1995	B&W	C-103-35	115, 116	1:24,000	Continental Aerial Surveys
10-15-1997	B&W	C-117-35	230, 231	1:24,000	Continental Aerial Surveys
2-24-1999	B&W	C-134-35	121, 122	1:24,000	Continental Aerial Surveys

PLATES



SOURCE: U.S.G.S. 7.5' topographic series, Anaheim and Orange, California quadrangle dated 1965 (1964), photorevised 1981.

N

2,000 1,000 0 2,000

APPROXIMATE SCALE (feet)

PLOTTED: 03 Jun 2009, 12:08pm, dfahrney

CAD FILE: L:\2009\CADD\103567\ LAYOUT: 1

Images: Topo-plate1_1.JPG Images: Topo-plate1_2.JPG

ATTACHED IMAGES: ATTACHED XREFS: DIAMOND BAR, CA

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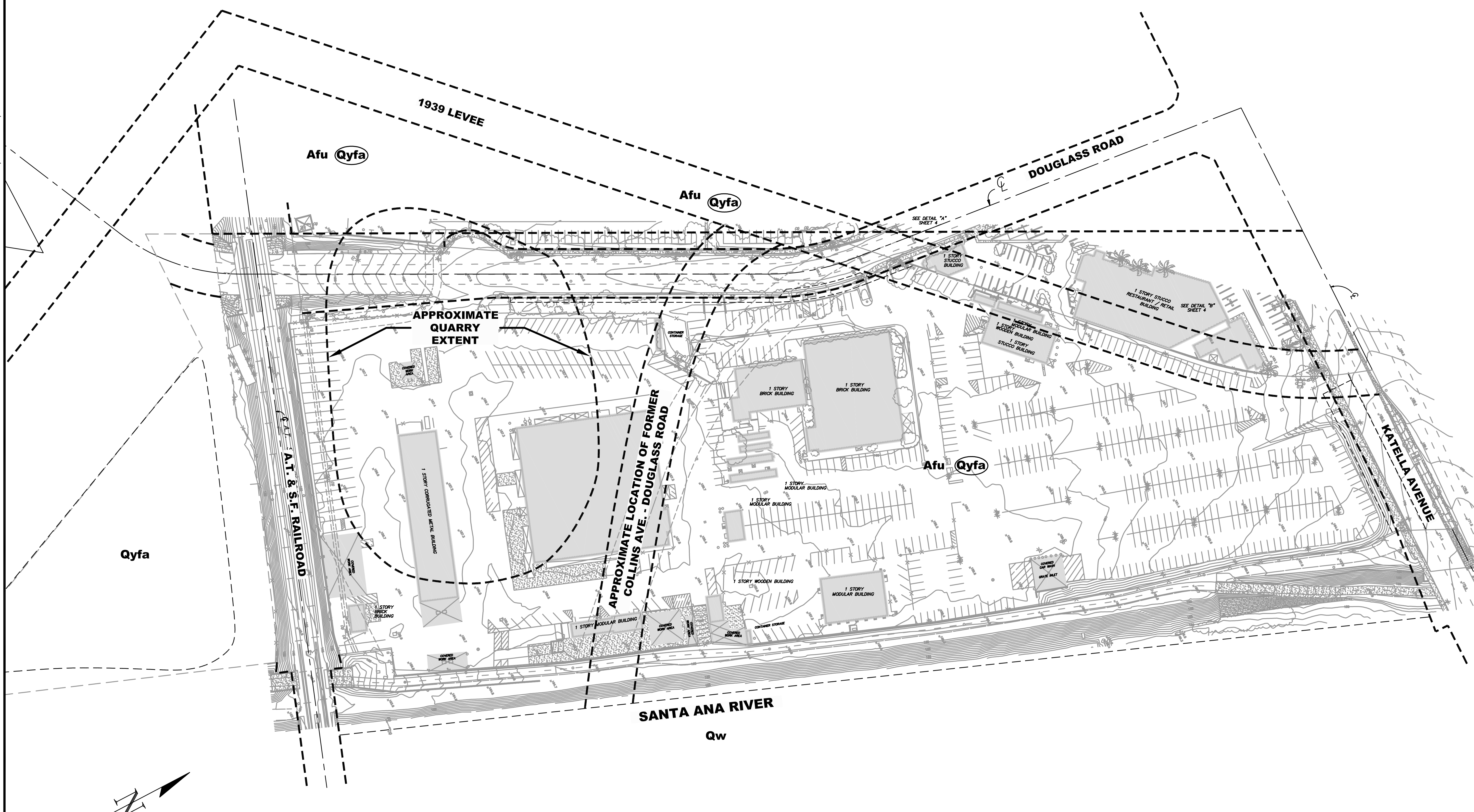
DRAWN:	6/02/09
DRAWN BY:	MRG
CHECKED BY:	MG
FILE NAME:	103567p1.dwg

SITE VICINITY MAP

PROPOSED ANAHEIM REGIONAL TRANSPORTATION INTERMODAL CENTER (ARTIC) - PHASE 1
ANAHEIM, CALIFORNIA

PLATE

1



120 60 0 120

APPROXIMATE SCALE (feet)

EXPLANATION

Afu	UNDOCUMENTED FILL
Qw	ACTIVE CHANNEL WASH DEPOSITS
Qyfa	ALLUVIAL FAN DEPOSITS OF SAND (CIRCLED WHERE BURIED)
— — —	APPROXIMATE GEOLOGIC CONTACT
— — —	APPROXIMATE FEATURE BOUNDARY

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DRAWN:	6/02/09
DRAWN BY:	MRG
CHECKED BY:	MG
FILE NAME: 103567p2.dwg	

GEOTECHNICAL MAP

PROPOSED ANAHEIM REGIONAL TRANSPORTATION INTERMODAL CENTER (ARTIC) - PHASE 1
ANAHEIM, CALIFORNIA

PLATE
2

PLOTTED: 03 Jun 2009, 11:16am, MGriffin
LAYOUT: 2
ATTACHED IMAGES: Images: IMAGE-Plate2.JPG
ATTACHED XREFS: XRef: 108816JMPR1: XRef: 108816JAT01
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