

Appendix F: Preliminary Geotechnical Investigation

**PRELIMINARY
GEOTECHNICAL INVESTIGATION
CENTERPOINT INTERMODAL CENTER
AIRPORT WAY, SOUTH OF ROTH ROAD
LATHROP, CALIFORNIA**

**REPORT PREPARED FOR:
CENTERPOINT PROPERTIES**

OUR PROJECT NUMBER: LGE090554

SEPTEMBER 15, 2009

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GEOTECHNICAL
ENVIRONMENTAL
INSPECTIONS & TESTING
LABORATORY SERVICES
POOL ENGINEERING
POST TENSION DESIGN

September 15, 2009
Our Project Number: LGE090554

CenterPoint Properties
C/O Mr. Jim Rachels
MCR Engineering
1242 Dupont Ct.
Manteca, CA 95336

Subject: **Preliminary Geotechnical Investigation
CenterPoint Intermodal Center
Airport Way, South of Roth Road
Lathrop, California**

Dear Mr. Rachels:

The following report presents the findings and conclusions of our preliminary geotechnical investigation conducted at the subject site. The purpose of the report was to provide preliminary recommendations for foundations, grading, seismic parameters in accordance with the 2007 CBC, utility construction, percolation rates, and flexible/rigid pavement sections, as indicated in our proposal dated July 14, 2009, and accepted July 28, 2009. Recommendations for this project have been provided in the body of the report. Coordination between our office and your grading contractor will help reduce the potential for soil related problems.

Key information regarding this geotechnical report is presented on the following page. This information sheet has been provided to aid you in assessing the limitations of this geotechnical investigation as well as to indicate when additional information from our office may be required.

We appreciate the opportunity of working with you on this project and look forward to providing our services in the future. Please contact us if you have any questions.

Sincerely,
NEIL O. ANDERSON & ASSOCIATES, INC.

Troy M. Schiess, Project Engineer
Professional Engineer 71404



SEP 15 2009

Garret S.H. Hubbard, Principal
Geotechnical Engineer 2588



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KEY INFORMATION REGARDING YOUR GEOTECHNICAL REPORT

➤ ***The Applicability of Geotechnical Reports is Limited***

Geotechnical reports are written to provide test results, observations, and professional opinions regarding a specific site for a specific project. Reports are tailored to the client and are influenced by each client's risk management strategies, economical constraints, and personal preferences. Since each report is a "custom fit" for a particular client, reports should not be transferred to anyone else without first consulting the geotechnical engineer.

Each geotechnical report considers only the construction information and site boundaries that existed at the time of the investigation. Modification of construction plans, such as a change in the shape, size, weight, location, or intended use of a project, nullifies the recommendations contained in the report, unless the geotechnical engineer indicates otherwise. A geotechnical report can not be used for an adjacent site. Time and money can often be saved by consulting with the geotechnical engineer when circumstances change from those which existed when the report was written.

➤ ***Site Conditions Can Change***

The conditions which existed at the time of a geotechnical investigation can change. Investigations can only report conditions at a particular time and place and no guarantee exists to ensure that recommendations will apply after natural or man made changes occur. Examples of some possible changes include: earthquakes, floods, fluctuations in groundwater, construction on or *next* to the site, and the addition or removal of soil. In addition, even the mere passing of time can affect site conditions. Consult with the geotechnical engineer to verify site conditions have not changed since the geotechnical report was completed.

➤ ***Geotechnical Findings Are Comprised Primarily of Professional Opinions***

Even if typical 6 inch borings were spaced 5 feet apart across an entire site (typical borehole spacings are on the order of at least 10's or 100's of feet apart), *less than one percent* of the soil or rock on the site would actually be explored. From this limited exploration, the geotechnical engineer is called on to provide an opinion regarding the subsurface conditions across the site, provide appropriate foundation recommendations, and predict the response of subsurface materials to numerous scenarios using information from samples that may or may not be representative of the entire site. Obviously, most of the geotechnical report is based on the professional opinion of the geotechnical engineer. The actual subsurface conditions may significantly differ from those which were encountered during the geotechnical investigation. Consequently, the most effective method of managing the risks associated with a project is to retain the geotechnical engineer who provided the report throughout construction of the project.

➤ ***Contact Your Geotechnical Engineer When in Doubt***

Time, money, and confusion can all be saved by simple explanations at critical moments. Please contact your geotechnical engineer whenever there is any doubt regarding subsurface conditions or their effect on part or all of any project.



**PRELIMINARY GEOTECHNICAL INVESTIGATION
CENTERPOINT INTERMODAL CENTER
LATHROP, CALIFORNIA**

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1.0 INTRODUCTION

This report presents the findings, conclusions, and recommendations of a preliminary geotechnical investigation conducted for the proposed CenterPoint Intermodal Center to be located on Airport Way south of Roth Road in Lathrop, California.

Our office understands that the project is in the site development planning stage and this report is providing preliminary geotechnical recommendations for the site development and initial building design. Once the site development design is completed, the building design/layout and construction loading is finalized, our office will perform a review of the site development documents and initial building designs to determine where additional borings will be required to provide our final design recommendations.

The project will consist of a new intermodal center which will include on site storm detention basins, interior asphalt/concrete roads and parking pavement, and twenty-three buildings ranging from 10,000 square feet to over 1,000,000 square feet along with associated utilities. Construction for smaller structures will likely consist of wood frame and/or concrete tilt-up with light interior steel framing. For the larger construction, framing will either be all steel frame and/or concrete tilt-up walls with steel interior framing. Buildings will utilize concrete slab on grade floors designed for warehouse or office use.

We anticipate maximum foundation loads (dead plus live) for these structures to be in the range of 2 to 4 kips per linear foot for perimeter and interior wall loads and 30 to 100 kips for isolated column loads. Since the site is relatively level, we expect that grading will consist of minor cuts and fills, less than 3 feet in vertical extent.



The preliminary geotechnical study conducted at this site was prepared for the use of the architect and engineer for application to aid in developing a preliminary design of the buildings and site development plans in accordance with generally accepted geotechnical engineering practices. No warranty is expressed or implied. This report presents the results of this study.

2.0 GENERAL (SURFICIAL) SITE CONDITIONS

At the time of our investigation the site was occupied by agricultural land with open fields, farming structures, and private residences. The private residences and farming structures are located along Airport Way. The site is relatively flat. The site is bordered to the west by the existing UPRR and SSJID French Camp Outlet Canal; to the east by Airport Way; to the north by Roth Road; and to the south by open agricultural fields.

3.0 GENERAL GEOLOGIC AND SITE CONDITIONS

A geologic map of the area was reviewed and indicated the surface soils are described as Pleistocene age Arkosic alluvium deposits of the Modesto Formation (Qm). The closest active fault greater than a magnitude of 6.5 with a slip rate greater than 2 millimeters per year is the Greenville Fault Zone located a distance of 40 kilometers from the site. A significantly more active fault with a magnitude greater than 7.0 and a slip rate equal to or greater than 5 millimeters per year is the Hayward Fault located at a distance of 66 kilometers

The 2007 California Building Code adopted January 1, 2008 references the 2006 International Building Code and the ASCE 7-05 Standard in-lieu of the Uniform Building Code previously utilized by the State of California. Following is a table of the 2007 California Building Code Soil Parameters¹ which may be used for seismic design of structures at the subject site:

2007 California Building Code Seismic Design Parameters	
Site Class	D
Mapped Spectral Acceleration Value of Rock (Short Period), S_s	0.915g
Mapped Spectral Acceleration Value of Rock (1-Second Period), S_1	0.314g
Site (Amplification) Coefficient, F_a	1.134
Site (Amplification) Coefficient, F_v	1.773
Maximum Considered Earthquake/Site Modified (MCE) Spectral Response Acceleration Value (Short Period), SM_s	1.038g
Maximum Considered Earthquake/Site Modified (MCE) Spectral	0.556g

¹ USGS Earthquake Ground Motion Parameters Version: 5.0.9 – 10/06/08



2007 California Building Code Seismic Design Parameters	
Response Acceleration Value (1-Second Period), SM_1	
Design Spectral Acceleration Value (Short Period), SD_5	0.692g
Design Spectral Acceleration Value (1-Second Period), SD_1	0.371g

A site latitude and longitude of 37.8460° and -121.2578° were utilized in conjunction with the tools provided by United States Geologic Survey web site. In accordance with 2007 California Building Code, Section 1802.2.7, this site has a peak design ground acceleration of 0.277g ($SD_5/2.5$). An analysis of earthquake induced liquefaction for the proposed site is included in Section 7.0, Liquefaction Analysis, of this report.

4.0 FIELD EXPLORATION AND LABORATORY TESTING

The field investigation conducted at this site consisted of drilling 5 exploratory test holes carried to depths of 11½ to 41½ feet below the existing ground surface (bgs). Six percolation tests were also performed at depths of about 4 and 6 feet below ground surface within the storm detention basin areas to aid in the design parameters for basins. The test holes were drilled with a CME 75 drill rig. The borings were drilled with 6-inch diameter hollow stem continuous flight auger. The locations of the test holes are shown on the Boring Location Map, Plate 1. The locations of the test holes were determined by pacing from existing site features; hence, accuracy can be implied only to the degree that this method warrants.

Sampling of the drilled test holes was performed at various depths using a California Modified 2.5 inch o.d. split spoon sampler with stainless steel tube liners or an unlined Standard Penetration Test (SPT) sampler. The sampler was driven by a 140 pound hammer with a 30-inch drop. Blow counts required to drive the sampler every 6 inches for a total of 18 inches were recorded.

Soil samples obtained from the test holes were preserved in stainless steel tubes and sealed plastic baggies until the samples could be tested in the laboratory. Samples were taken to the laboratory of Neil O. Anderson & Associates, Inc., Lodi, California and used for performing various laboratory tests. Tests performed consisted of unit weights, moisture contents, Wash 200's, Atterberg Limits, and Pocket Penetrometer readings. A summary of the test results are presented on the Log of Test Boring sheets, Plates 2 through 6.

Four bulk samples of the subgrade soil in the proposed pavement areas were obtained for performing R-value tests. The samples were obtained from the 12 to 18 inch depth. The R-value tests resulted in values of 53, 69, 69, and 66; therefore an R-value of 53



was utilized for pavement designs. For the approximate locations of these samples, see the Boring Location Map, Plate No. 1.

5.0 SOIL CONDITIONS

Visual classification of each soil stratum encountered according to ASTM D2488 (Visual – Manual Procedure) was made in the field by a representative from our office at the time the test holes were drilled. The samples obtained were checked in the laboratory by a professional engineer and classification verified according to ASTM D2487. A classification and graphical representation of each soil encountered is presented on the Log of Test Boring sheets. The test boring legend is presented on Plate No. 15.

The surface soils encountered during our field investigations were fairly uniform between test holes. Test boring B1 extended to a depth of 40½ feet and was performed for liquefaction purposes. This boring consisted of 11½ feet of medium dense silty sand which was moderately cemented at a depth of 10 feet below ground surface (bgs). This was underlain by a 4½ foot stratum of very stiff sandy silt, which was underlain by very stiff lean clay to a depth of 20 feet bgs. This was underlain by stiff sandy clay to a depth of 24 feet bgs, which was underlain by a 6 foot stratum of silty clay. This was underlain by a 5 foot stratum of very stiff to hard fat clay, which was underlain by dense sand with silt to a depth of 39 feet bgs. This was underlain by loose sandy silt to the maximum depth explored of 41½ feet bgs.

The surface soils encountered in test hole B2 consisted of 3½ feet of loose silty sand which was underlain by a 9½ foot stratum of medium dense to very dense silty sand which was moderate to strongly cemented at depths between 3½ and 6 feet bgs. This was underlain by very stiff to hard silty clay to a depth of 19 feet bgs, which was underlain by a 4 foot stratum of medium dense sand with silt. This was underlain by very stiff lean clay to the depth explored of 26½ feet bgs.

The surface soils encountered in test hole B3 consisted of medium dense sand with silt to a depth 4 feet bgs. This was underlain by a 5 foot stratum of loose sand with silt to a depth of 9 feet bgs, which was underlain by medium stiff sandy silt to the depth explored of 13 feet bgs.

The surface soils encountered in test hole B4 consisted of medium dense silty sand to a depth of 8 feet bgs which was moderately cemented between the depths of 3½ and 5 feet bgs. This was underlain by medium stiff to stiff clayey silt with sand to the depth explored of 11½ feet bgs.

The surface soils encountered in test hole B5 consisted of a 4 foot stratum of medium dense silty sand. This was underlain by a 3 foot stratum of medium dense sand with



silt which was moderately cemented between the depths of 4 and 6 feet bgs. This was underlain by medium dense sand with some silt to a depth of 13½ bgs, which was underlain by a 5 foot stratum of dense silty sand. This was underlain by medium dense clayey sand to the depth explored of 21½ feet bgs. For a more detailed description of the soils encountered in the test holes see the Log of Test Boring sheets.

Test hole logs show subsurface conditions at the date and location indicated and it is not warranted that they are representative of subsurface conditions at other locations and times.

6.0 GROUNDWATER

Groundwater was encountered in our test borings at depths ranging between 10 and 22 feet below the existing ground surface at the time the test holes were drilled. Please see the following table showing the depth to groundwater for each location.

DEPTH TO GROUNDWATER AT EACH BORING LOCATION	
BORING LOCATION	DEPTH TO GROUNDWATER (ft.) Measuring from the ground surface
B1	22
B2	20
B3	11
B4	10
B5	20

The test holes were allowed to remain open for approximately 3 to 5 hours for the groundwater to stabilize. A County groundwater map was reviewed and indicated groundwater in the area could be at a depth of 17 feet below ground surface. Groundwater conditions in the future could change due to rainfall, construction activities, irrigation, or other factors. The evaluation of these factors is beyond the scope of this study. Due to potential fluctuation of groundwater depths, and the groundwater depths encountered in our test borings, a groundwater depth of no more than 10 feet should be utilized for design purposes.

7.0 LIQUEFACTION ANALYSIS

A detailed analysis of earthquake induced liquefaction for the proposed site was completed. Liquefaction is a loss of strength in soil when a cyclic stress, such as that caused by an earthquake, is subjected to typical soils, such as loose saturated sands and silts. The cyclic stress attempts to densify the soil causing an immediate increase in pore pressure. A cyclic stress subjected to these soils may result in excessive pore pressures stimulating the soil to behave as a liquid. Factors that affect the potential for



in pore pressure. A cyclic stress subjected to these soils may result in excessive pore pressures stimulating the soil to behave as a liquid. Factors that affect the potential for liquefaction include the age and density of the deposits, depths to subsurface water, and the potential site ground acceleration from a seismic event.

Prior to providing specific recommendations for construction of the structures, the susceptibility of the site to liquefaction needs to be addressed. The following describes the results of our liquefaction analysis for the site.

The results of the dynamic "blow count" testing performed during the drilling of a 41½ foot deep boring has been used in our liquefaction analysis. Blow counts were taken approximately every 5 feet and a liquefaction analysis of each distinct stratum has been performed. Our analysis of the potential for liquefaction at the site was performed using two methods (both are based on blow count result values obtained during drilling activities). The first, and probably the most commonly used method, is that proposed by the National Center for Earthquake Engineering Research (NCEER)². This method results in a calculated factor of safety against liquefaction. The second method is a newer method proposed by R. B. Seed and results in a calculated probability of liquefaction³.

A truck mounted Mobile CME 75 drill rig was used to drill the 41½-foot deep boring. Blow counts reported in the bore logs are **corrected** (i.e. correction for a standard 60 percent efficiency).

As was indicated, two methods of analysis were used to assess the liquefaction potential of the site. Both of these methods require the same input. Besides data obtained during field and laboratory testing, a value of peak ground acceleration at the site and a magnitude of the earthquake responsible for the peak ground acceleration are required. A magnitude of 6.9 was selected as the design earthquake (Greenville Fault located 40 kilometers away from the site) and a peak horizontal ground acceleration of 0.277g was calculated and used in our liquefaction analysis. The shear wave velocity was estimated to be 670 feet per second (approximately 200 meters per second) based on correlations with blow count values. Saturated conditions were assumed to exist at a depth of 15 feet due to the depth of ground water measured in our test holes, San Joaquin County groundwater maps, and the potential for fluctuating groundwater depths in the future. The following table shows the results of our

² Youd, Leslie T., Idriss, Izzat M., *Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils*, Technical Report NCEER-97-0022, December 31, 1997.

³ Seed, R.B., *Recent Advances in Soil Liquefaction Engineering and Seismic Site Response Evaluation*, Proceedings of the Fourth International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics and Symposium in Honor of Professor W.D. Liam Finn, San Diego, California, March 26-31, 2001, Paper No. SPL-2.



liquefaction analysis based on soils encountered and test results obtained in our test boring B1 (see Plate No. 2).

LIQUEFACTION ANALYSIS			
Layer, depth below ground surface (feet)	Factor of Safety based on NCEER method¹	Probability of Liquefaction based on Seed et. Al. method²	Comments
0-11	1.97	0.00	Very low potential for liquefaction, no ground water.
11-15	4.56	0.00	Very low potential for liquefaction, no groundwater and significant amount of fines.
15-20	0.42	0.00	Low potential for liquefaction due to very stiff soil conditions, amount of fines, and high blow counts.
20-25	1.19	0.24	Very low potential for liquefaction due to stiff soil conditions, amount of fines, and high blow counts
25-30	0.25	0.00	Low potential for liquefaction due to very stiff soil conditions, amount of fines, and high blow counts
30-35	1.20	0.22	Very low potential for liquefaction due to very stiff soil conditions, amount of fines, and high blow counts
35-39	2.12	0.00	Very low potential for liquefaction due to dense soil conditions and high blow counts.
39-41½	0.72	1.00	Moderate potential for liquefaction due to low blow counts.

Note 1: A value less than 1.0 indicates liquefaction is predicted.

Note 2: The closer the value is to 1.0, the more probable liquefaction becomes.

The darker shaded cells are shown in the table to emphasis the layers most prone to liquefaction, as determined by the methods used. The results of both of these methods need to be evaluated further since they have only taken into account fine content and do not differentiate between silt and clay size particles. Even though the procedures shows the potential for liquefaction, further assessment considering the clay content of the soil layer needs to be analyzed. The Modified Chinese Criteria was used to determine if the fined grained soil layers would indeed have the potential to liquefy. This criteria states that if the soil behavior indicates a liquid limit (LL) >32 percent, a water content <0.9*LL percent, or a percent finer than 0.005mm ≥15% the potential for liquefaction is low. This criterion was used in conjunction with our analysis to evaluate the potential for liquefaction to occur in specific soil layers.

The state of the art in predicting liquefaction induced settlement is limited. Estimates of settlement are generally expected to vary on the order of a factor of 2. Our calculations of settlement due to liquefaction or strong seismic shaking are based on



the method described by Tokimatsu and Seed⁴. Due to the depth of liquefiable soils (39 feet below ground surface), our analysis indicates that the total settlement should be less than 1 inch. Differential settlement should be less than half the total settlement since the potential for liquefaction to occur is at a depth of 39 feet below ground surface, and it is overlain by a 20 foot stratum of very stiff clayey material that will help bridge any settlement in the underlying soils, and will likely occur globally across the site. Please note that these calculations are applicable only to the soils encountered in our borings. Liquefaction may potentially occur in soils below those encountered in our investigation; if deeper soils do liquefy, the potential for settlement of the site may increase.

Due to the relatively flat nature of the site and the lack of a free face condition, damage of the building from lateral spreading is not expected. However, ground oscillation and "limited" lateral displacements may still occur.

8.0 PRELIMINARY DESIGN STUDIES AND RECOMMENDATIONS

From a soil engineering standpoint, our office concludes that the site is suitable for construction of the proposed Intermodal Center; however, all of the conclusions and recommendations presented in this report should be incorporated into the design and construction to help reduce the potential for soil and foundation problems. Our main item of concern is the loose surface soils encountered in test hole B2, and providing uniform support for the proposed structures.

8.1 Grading

The site should be initially cleared of all vegetation, trees, roots, debris, pavement and deleterious material as outlined in Appendix A, Engineered Fill Specifications. Areas that are covered with light vegetation consisting of native weeds and grasses may be blended into the soil. Areas of moderate or heavy vegetation should not be blended into surficial soils, but should be cut and removed from the site. An engineer from our office is required to determine the degree of vegetation prior to grading operations. Voids resulting from the removal of any buried structures (such as irrigation structures or pipes, foundations, septic systems or water lines) should be cleaned of all loose soil and debris so that they may be backfilled during filling operations. All wells shall be abandoned in accordance with San Joaquin County requirements. After clearing operations and any cuts have been made, the subgrade thus exposed shall be scarified a minimum of 12 inches and compacted as indicated in Appendix A. Fill placed on

⁴ Tokimatsu, K. and Seed, H.B., *Evaluation of Settlements in Sands due to Earthquake Shaking*, Journal of Geotechnical Engineering, Vol. 113, No. 8, August, 1987.



building pads and in pavement areas should be non-expansive and placed as engineered fill as recommended in Appendix A. Soils encountered on the site should be suitable for use as engineered fill.

8.2 Spread Foundations

If grading is accomplished as specified, foundations for the proposed buildings may consist of shallow, spread or continuous foundations bearing on compacted native soil, engineered fill, or a combination of both. The minimum width of all foundations shall be 12 inches. The foundations shall be embedded 18 inches below lowest surrounding grade. In order to provide more uniform support for the foundations, we recommend that bottoms of all foundation excavations shall be hand compacted (by whackers, pogo sticks, vibraplates, etc.) prior to the placement of any reinforcing steel or concrete. Foundations bearing on compacted native soil may be designed using an allowable bearing capacity of 1,500 pounds per square foot (psf), for dead plus live loads.

If a higher bearing capacity is desired, the foundations shall be supported on a minimum of 2 feet (measured from the bottom of footing) of engineered fill consisting of overexcavated and compacted soil as specified in Appendix A. With the foundations supported on a minimum of 2 feet of engineered fill (measured from bottom of footing), a bearing capacity of 3,000 psf for dead plus live loads may be used in design. These bearing capacities may be increased by 1/3 for temporary wind and seismic loads.

Potential settlement, either immediate or long term, of foundations constructed as indicated above and loaded in the manner described above, should be less than 1 inch total and 1/2 inch differential across the width of the building. Care should be taken to understand settlements may vary based on actual loads and footing sizes.

To ensure footings have adequate support, special care should be taken when footings are located adjacent to trenches. The bottom of such footing should be at least 1 foot below an imaginary plane with an inclination of 1.5 horizontal to 1.0 vertical extending upward from the nearest bottom edge of the adjacent trench.

Lateral resistance for spread footing may be provided by assuming a passive pressure acting against the side of the footings equal to 350 pounds per cubic foot (pcf) equivalent fluid pressure. Lateral resistance may also be provided by computing friction between the bottom of the footing and the soil. A coefficient of friction of 0.50 should be utilized. If footings are cast against firm native soil, passive and frictional resistance may be combined but the passive resistance should be reduced by 50 percent.



8.3 Conventional Building Slabs

Moisture transmission through concrete slab-on-grade floors has been known to cause delamination, warping and other damage to floor coverings. Wood and vinyl floorings are particularly susceptible to damage. Neil O. Anderson & Associates does not profess to be experts in moisture proofing concrete slabs-on-grade, and our firm knows of no construction method that will completely eliminate the risk of damage. In order to provide some level of protection against damage, it is common practice in this area to place a capillary break and a vapor retarder beneath the slabs.

There are additional measures that may be incorporated to further reduce, but not eliminate, the risk. Some (but not all) of these measures include: using concrete with a water-cement ratio less than 0.45, employing a qualified testing laboratory to provide materials testing and quality control during concrete placement and curing, using topical concrete sealers, installing water stops at cold joints between the foundation and slab on grade, sealing the vapor retarder where plumbing penetrations occur, limiting the use of vinyl and wood flooring, and testing the concrete slab for moisture transmission rates immediately prior to placement of floor coverings. These measures may be considered if additional protection is desired.

The following recommendations are commonly used in this area and we believe these measures should be incorporated to provide a minimum level of protection against damage.

Minimum Recommendations:

Four inches of clean gravel should be placed beneath the slabs on grade. This gravel is in addition to any engineered fill that may be placed and should be well graded between a maximum size of 1 inch and a minimum size of ¼ inch with zero percent passing the No. 4 sieve. The gravel should be covered by an impervious vapor retarder such as 10 mil visqueen or equivalent. The vapor retarder should be covered by 1 to 2 inches of sand to protect it during construction and to aid in curing the concrete. This sand should meet the requirements of ACI 302.1R. However, we know from experience that most local sand will not meet these requirements. In our opinion, the sand should be a sand or silty sand containing no more than 20 percent passing the No. 200 sieve. Alternative materials must be approved by the geotechnical engineer prior to being brought to the site.

The sand should be moist but not saturated at the time of concrete placement. If the sand is saturated or free water is visible, the concrete should not be placed until the sand is dried sufficiently to only be moist or is replaced. Excessive moisture in the sand



can lead to problems with excessive moisture vapor related problems with the concrete slab on grade.

If construction will take place in winter, sand may be substituted with 3/8 inch pea-gravel. The pea gravel may not be saturated. Free water must not be visible on the gravel. If the gravel is saturated, it must be dried sufficiently to only be moist or be replaced prior to placement of concrete.

The thickness and reinforcing for the slabs on grade should be provided by the project structural engineer.

8.4 Industrial Floors

Warehouse/industrial floor slabs subjected to forklift or other heavy loads are often designed using a modulus of subgrade reaction. We recommend using a modulus of 150 pounds per cubic inch (pci) for subgrade surface soils processed according to this report. Industrial floor slabs should be underlain by at least 4 inches of Class II aggregate base over the compacted subgrade. Aggregate base should be compacted to at least 95 percent of maximum dry density obtained in the ASTM D1557 test method. Industrial floor slabs should be designed by the structural engineer. The structural engineer should select the most appropriate design method for the intended use of the slab. We recommend that all construction joints be doweled with slip dowels to provide additional strength against breakage of the slabs at the joints from heavy fork lift traffic or movement of heavy equipment over the joints.

8.5 Retaining/Screen Walls

Site retaining walls may be constructed. Retaining walls will be subject to lateral earth pressures. Site retaining walls may be supported by a spread footing type foundation.

The lateral earth pressure on a retaining wall depends on the height of the wall, type of backfill, slope of the backfill surface, and allowable horizontal movement on top of the wall. A calculated at-rest earth pressure of 55 pcf equivalent fluid density should be used for retaining walls which are restrained from rotating at the top. A calculated active earth pressure of 35 pcf equivalent fluid density should be used for site retaining walls which are allowed to rotate at the top. We have assumed the backfill will be on-site non-expansive soils and have a flat surface behind the wall. For lateral load resistance, footings may be designed with a passive earth pressure of 350 pcf. For seismic consideration, an additional wall surcharge of 10pcf should be added to the lateral equivalent fluid pressure. Equivalent fluid densities do not include allowances for surcharge loads or hydrostatic pressures. The hydrostatic pressure on the retaining walls should be relieved using drains behind the walls connected to tight lines.



8.6 Drainage

Special care should be taken to ensure adequate drainage is provided throughout the life of the structures. Properly designed and constructed foundations can be seriously damaged by neglecting to install and regularly verify performance of recommended drainage systems. Appropriate down spout extensions from roof drainage should fall on splash blocks a minimum of 2 feet from the structures or, more efficiently, be connected to tight lines that drain away from the buildings. Any flatwork adjacent to the buildings should slope a minimum of 1 percent for a distance of 5 feet. Exposed exterior subgrade (soil or non-paved areas) should slope away from the structures at a minimum slope of ½ inch per foot for a distance of 8 to 10 feet beyond the building perimeters. If this grade is unable to be obtained, proper drainage inlets will need to be placed to carry surface water away from the foundations.

Care should be taken to ensure that landscaping is not excessively irrigated and to ensure that landscaping drains away from the structures. Implementation of adequate drainage for this project can effect the surrounding developments. Consequently in addition to designing and constructing drainage for the subject site, the effects of site drainage must be taken into consideration for surrounding sites.

8.7 Excavation

As indicated previously, loose to medium dense surface soils over stiff to very stiff subsurface soils were encountered in our test borings. Cemented (hardpan) areas were encountered between 3½ feet and 10 feet in certain areas. Consequently, conventional excavating equipment may be utilized on this site. The contractor should plan his work accordingly.

8.8 Testing, Inspections and Review

Our office should be afforded the opportunity of reviewing the completed foundation and grading plans to verify that our recommendations have been properly interpreted and incorporated. Unless our office is allowed this opportunity, we disavow any responsibility from problems arising from failure to follow geotechnical recommendations or improper interpretation and implementation of our recommendations.

Our office should be retained to perform the recommended grading observations and compaction testing. Unless we have been retained to provide these services, our office cannot be held responsible for problems arising during or after construction that could



have been avoided had these services been performed. The fees for these services are in addition to that associated with this report.

8.9 Percolation Tests

As requested we performed a total of six percolation tests in basin areas for use by others in the design of the onsite retention basin ponds. The approximate locations of the percolation tests are shown on the Boring Location Map, Plate No. 1.

The test holes were drilled to depths of about 4 to 6 feet below the existing ground surface. Following drilling, the holes were cased with 2-inch diameter PVC pipe and gravel placed around the outside of the pipe. The tests were performed in accordance with generally accepted guidelines. A 12-inch head of water was maintained during the test. The drop in water level was measured every 10 minutes until a relatively consistent percolation rate was established. The percolation rate in gallons per square foot per day (gfd) was calculated based on the measurements in the last 10 minutes of the test. The test results are summarized in the following table.

Basin Location	Percolation Test	Depth of Test (ft.)	Percolation Rate, (gal/SF*day)
Profile P1 (South end of project)	PT1	4.0	100
	PT2	5.0	150
Profile P2 (Center of project)	PT3	4.0	20
	PT4	6.0	20
Profile P3 (North end of project)	PT5	4.0	30
	PT6	6.0	10

Note 1: No Factor of Safety applied to values in above table.

Note 2: Please see Plate Nos. 7 through 9 for soil profile sheets.

Since our tests were performed using clean water, the storm water runoff will likely contain materials such as silt, leaves, oil residues, and other matter that may reduce the percolation characteristics of the soils. We recommend that a factor of safety of 2 be applied to the percolation rate for use in design. The values in the table above do not have a factor of safety applied. In addition, we recommend that a vigorous program of disking the bottom and sides of the drainage basins be maintained for the life of the project. With time, the basin bottom and sides are likely to silt over and



become clogged. All intakes should be cleaned regularly following significant rains and prior to the beginning of the rainy season.

9.0 PAVEMENT RECOMMENDATIONS

We understand that the project will utilize flexible and rigid concrete pavement; recommendations for these pavements are presented next. We understand that permeable pavement may be considered for this project. Our office can provide recommendations for permeable pavement if desired.

9.1 Flexible Pavement Recommendations

Two representative soil samples were taken from the site and subjected to R-value testing in our laboratory. The locations of these soil samples are shown on the Boring Location Map, Plate No. 1. As indicated, a design R-value of 53 was utilized. Traffic indices of 5.0 and 6.0 were used to design the passenger vehicle only pavement sections for the site based on our experience with similar sites. We anticipate that truck traffic for this site could have 50 to 70 trucks per day, which results in a Traffic Index of 9.0. **The project civil engineer should be afforded the opportunity of specifying the most appropriate traffic index for the proposed traffic and usage.** If a different traffic index is desired or required, please contact our office and a suitable recommended design can be provided. Flexible (asphalt) pavement sections have been designed according to the latest addition of the Cal Trans Highway design manual and using a 20-year pavement life. The pavement sections designs are shown below.

Conventional Flexible Pavement Sections				
Subgrade R-Value	Traffic Index	Traffic	Pavement Section, Inches	
			Asphalt Concrete	Aggregate Base
53	5.0	Auto	3.0	4.0
53	6.0	Truck	3.5	5.0
53	7.0	Truck	4.0	6.0
53	9.0	Truck	5.5	6.0

The paving materials must conform to the requirements of the State of California, Department of Transportation, Standard Specifications, latest edition. Type B asphalt concrete and class 2 aggregate base should be used. The subgrade should have a minimum R-value of 53.

Studies have indicated that a major factor in extending pavement life is to provide adequate drainage for both the pavement surface and subgrade. Care should be made



during the development of the grading plan to provide for good drainage. Landscaped and irrigated planters that are constructed adjacent to pavement should have cut-off curbing constructed around them that extends a minimum of 4 inches into the subgrade soil. We recommend rigid concrete pavements in areas where heavy trucks, such as garbage trucks, will travel or make sharp turns. The above recommended pavement sections assume periodic maintenance, such as crack sealing, etc., will be performed over the life of the pavements.

9.2 Rigid Pavement Recommendations

The recommended concrete pavement sections have been designed utilizing the Portland Cement Associations manual "Thickness Design for Concrete Highway and Street Pavements". Design is based on a 20 year pavement life. The pavement sections are presented next.

RIGID PAVEMENT SECTION DESIGN				
Subgrade strength	Traffic Index	Traffic	Pavement Section, inches	Aggregate Base, inches
High	7.0	Auto/Truck	6 (4000 psi concrete)	4.0
High	9.0	Auto/Truck	8 (4000 psi concrete)	4.0

Note: 3/4 inch diameter by 16 inch smooth dowels spaced at 12 inches on center should be lightly greased and utilized at construction joints. Dowels should be cut not sheared. #4 bars at 12 inches on center may be utilized for shrinkage control, however, bars should not continue across construction or contraction joints. Reinforcement across joints restrains joints from opening as the slab shrinks and expands during temperature fluctuations.⁵ As an alternative to shrinkage reinforcement, fiber or steel mesh may be utilized. A rough finish of the concrete surface also helps to mask cracks. Contraction joints should have a maximum spacing of 12 feet on center.

The pavement area subgrade should be stripped of all organic matter, loose soil, etc., and any required cuts or fills made. A minimum of 8 inches of compacted subgrade should be provided beneath the pavement sections. The subgrade should be compacted to dry densities in excess of 95 percent of the maximum dry density obtainable in the ASTM D1557 test method.

Studies have indicated that a major factor in extending pavement life is to provide adequate drainage for both the pavement surface and subgrade. Care should be made during the development of the grading plan to provide for good drainage. Landscaped and irrigated planters that are constructed adjacent to pavement should have cut-off curbing constructed around them that extends a minimum of 4 inches into the subgrade soil. We recommend rigid concrete pavements in areas where heavy trucks, such as

⁵ ACI, Guide for concrete floor and Slab Construction, ACI 302.1R-96.



garbage trucks, will travel or make sharp turns. The above recommended pavement sections assume periodic maintenance, such as crack sealing, etc., will be performed over the life of the pavements.

10.0 UTILITY CONSTRUCTION

Based on Occupational Safety and Health Standards, the soils encountered in our test holes classify as Types A (Clay), B (silty sand), and C (sand) soils. Types A, B, and C soils require a maximum slope of $\frac{3}{4}$ H:1V, 1H:1V, and 1½H:1V (horizontal to vertical), respectively, for excavations less than 20 feet deep. The contractor should have a competent person identify all soils encountered in excavation and refer to OSHA and Cal-OSHA standards to determine appropriate methods to protect individuals working in excavations.

Backfill placed in trenches should be placed in approximately 8 inch lifts in uncompacted thickness. However, thicker lifts may be used provided the method of compaction is approved by the soil engineer and the required minimum degree of compaction is achieved. Material should be compacted to at least 90 percent of the maximum dry density obtained by the ASTM D1557 test method. The upper 8 inches of trench backfill within pavement areas should be compacted to at least 95 percent relative compaction.

11.0 LIMITATIONS

The recommendations of this report are based on the information provided regarding the proposed construction as well as the subsoil conditions encountered at the test hole locations. If the proposed construction is modified or re-sited, or if it is found during construction that subsurface conditions differ from those described on the test hole logs, the conclusions and recommendations of the report should be considered invalid unless the changes are reviewed and the conclusions and recommendations modified or approved in writing.

The analysis, conclusions and recommendations contained in this report are based on the site conditions as they existed at the time we drilled our test holes. It was assumed that the test holes are representative of the subsurface conditions throughout the site. If there is a substantial lapse of time between the submission of our report and the start of the work at the site, or if conditions have changed due to natural causes or construction operations at or adjacent to the site, we urge that our report be reviewed to determine the applicability of the conclusions and recommendations considering the changed conditions and time lapse. This report is applicable only for the project and site studied. **This report should not be used after 3 years.**



Our professional services were performed, our findings obtained, and our recommendations proposed in accordance with generally accepted engineering principles and practices. This warranty is in lieu of all other warranties either expressed or implied. Test findings and statements of professional opinion do not constitute a guarantee or warranty, expressed or implied.

The scope of our services did not include any environmental assessment or investigation for the presence or absence of wetlands, hazardous or toxic materials in the soil, surface water, groundwater or air, on or below or around this site. Any statements in this report or on the soil logs regarding odors noted or unusual or suspicious items or conditions observed are strictly for the information of our client.



APPENDIX A

Engineered Fill Specifications

SCOPE

Principal items of work included in this section are as follows:

- A. Cleaning and Striping
- B. Construction of Fill

A. CLEANING AND STRIPPING

Work includes cleaning and stripping of the building pad and surrounding area as indicated on the drawings. From this area remove all debris, irrigation lines, old pavement, trees, brush, roots, and vegetable ruin and grub out all large roots (1/2 inch or greater diameter) to a depth of at least two feet below the footing elevation. The vegetable materials and all materials from the cleaning operation shall be removed from the site.

B. CONSTRUCTION OF FILL

1. Preliminary Operations

After the cleaning and stripping operation and the cuts have been completed and before any fill is placed in any particular area, the existing surface shall be scarified to a depth of 8 inches and compacted to dry densities in excess of 90 percent of the maximum dry density as obtained by the Standard Test Methods for Laboratory Compaction Characteristics of Soil using Modified Effort, ASTM D1557 designation. The soil should be compacted at moisture contents above 1 percentage point above the optimum moisture content. It may be necessary to adjust the moisture content of the subgrade soil by watering or aeration, to bring the moisture content of the soil near optimum in order that the specified densities can be obtained.

2. Source of Material

Engineered fill materials (on site or import) shall consist of sandy silts, sands, or sands and gravels unless stated otherwise in the report. Engineered fill material shall not contain rocks greater than 3 inches in greatest dimension and should be non-expansive in nature with a plasticity index less than 12.



At least seven days prior to the placement of any fill, the engineer shall be notified of the source of materials. Samples of the proposed fill shall be obtained to determine the suitability of the materials for use as engineered fill.

3. Placing and Compacting

Fill materials shall be spread in layers and shall have a uniform moisture content that will provide the specified dry density after compaction. If necessary to obtain uniform distribution of moisture, water shall be added to each layer by sprinkling and the soil disked, harrowed, or otherwise manipulated after the water is added. The layers of the fill material shall not exceed 8 inches and each layer shall be compacted with suitable compaction equipment to provide the specified dry densities.

4. Required Densities

The dry density of the compacted earth shall be at least 90 percent of the maximum dry density obtainable by the ASTM D1557 test method. The optimum moisture content and maximum dry density will be determined by the engineer and this information supplied to the contractor.

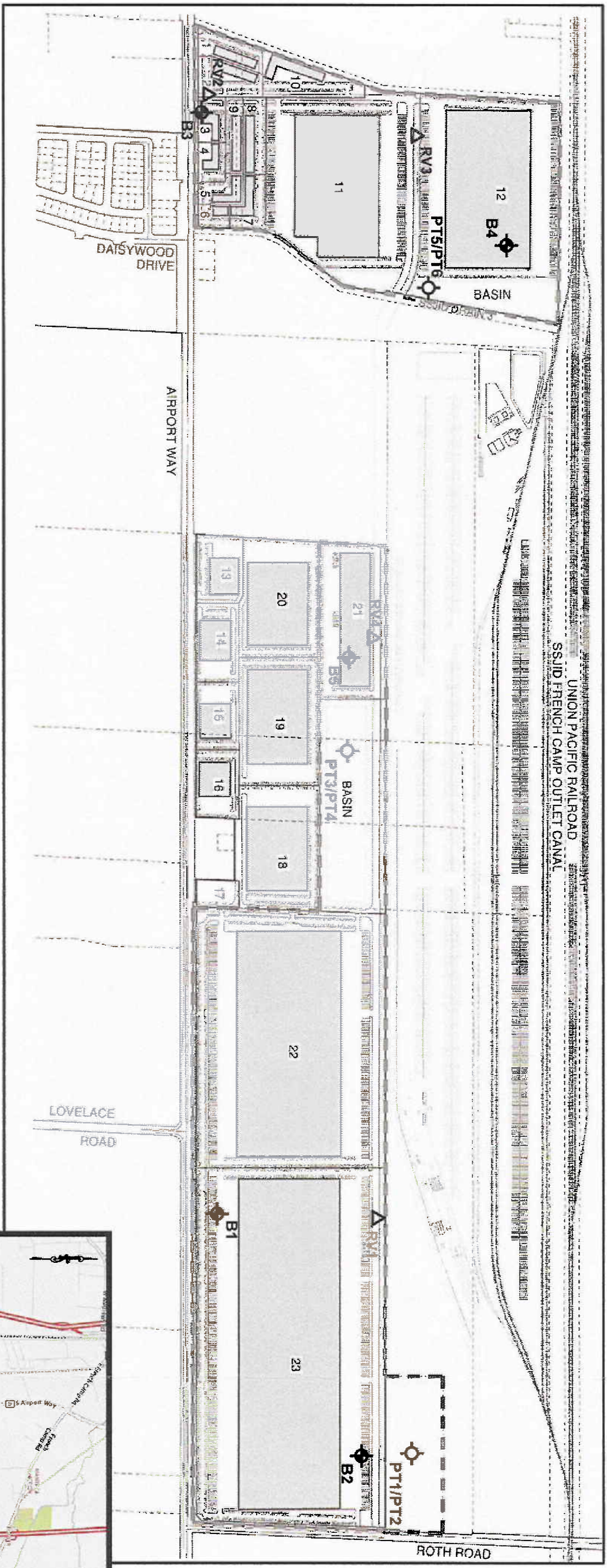
5. Seasonal Limits

No fill shall be placed during weather conditions which will alter the moisture content of the fill materials sufficiently to make adequate compaction impossible. After placing operations have been stopped because of adverse weather conditions, no additional fill material shall be placed until the last layer compacted has been checked and found to be compacted to the specified densities.

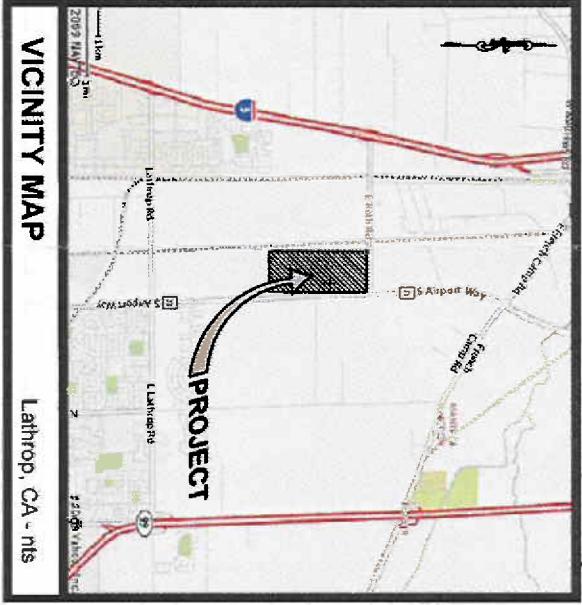
6. Control of Compaction

The density of the upper 6 inches of subgrade and of each layer of fill shall be checked by the engineer after each layer has been compacted. Field density tests shall be used to check the compaction of the fill materials. Sufficient tests shall be made on each layer by the engineer to assure adequate compaction throughout the entire area. If the dry densities are not satisfactory, the contractor will be required to increase the weight of the roller, the number of passes of the roller, or manipulate the moisture content as required to produce the specified densities.





EXPLANATION	
	Approximate location of percolation test
	Approximate location of boring
	Approximate location of R-Value



Base plan provided by MCR Engineering, Inc., Manteca, CA 95336

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BORING LOCATION MAP
CENTERPOINT INTERMODAL CENTER
 AIRPORT WAY AND SOUTH OF ROTH ROAD
 LATHROP, CA

Job No:	LG090554
Date:	9/15/09
Scale:	NONE
Drawn by:	EC
Checked by:	TS
Plate No:	1



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LOG OF TEST BORING

BOREHOLE NUMBER

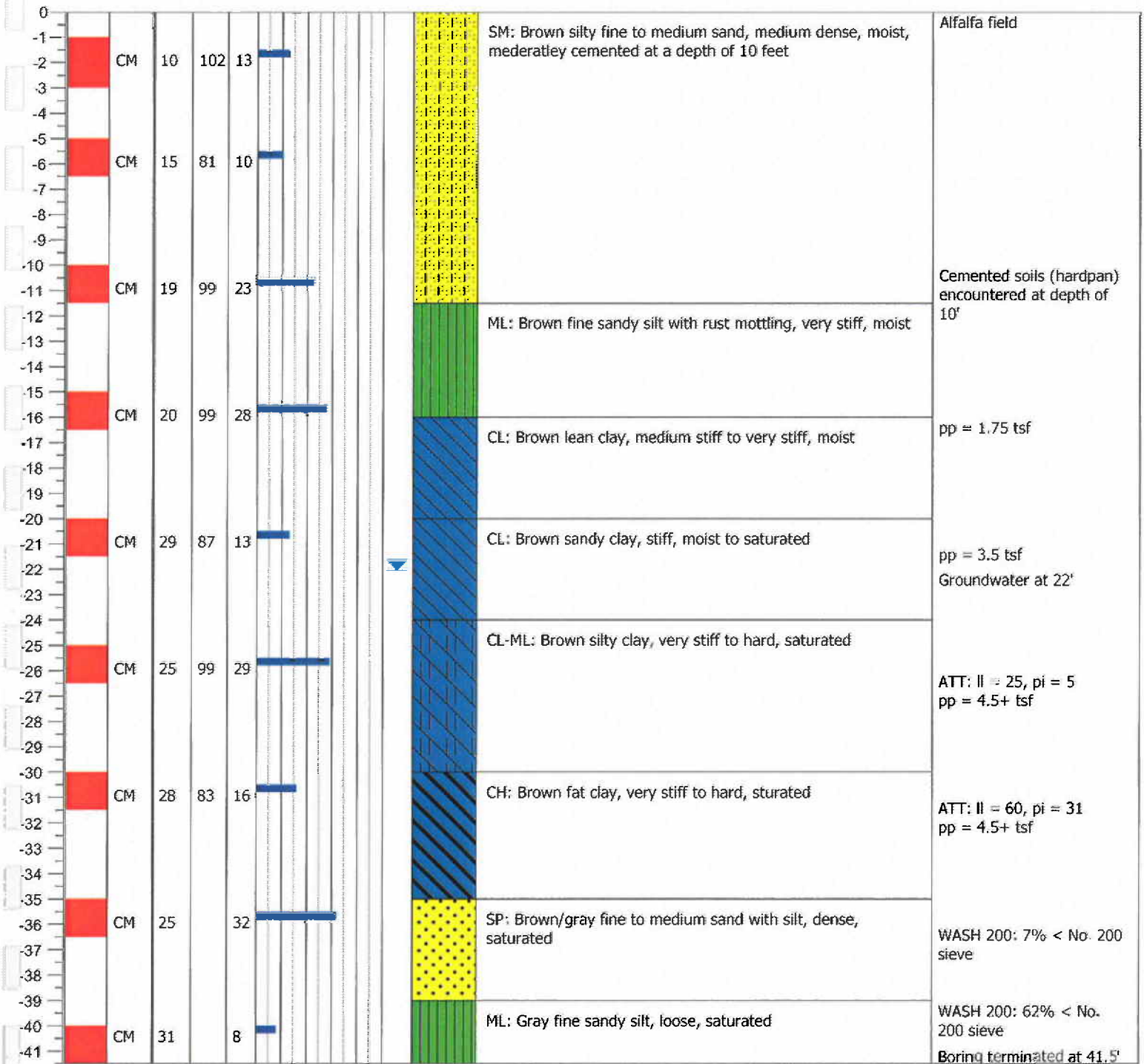
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PROJECT NUMBER: **LGE090554**
PROJECT NAME: **CenterPoint Intermodal Center**
LOCATION: **Lathrop, California**
DRILLING EQUIP.: **CME 75 Drill Rig**

DATE DRILLED: **08-03-2009**
GROUND SURFACE ELEVATION: **0.0** Feet

PLATE 2

Depth, ft.	Sample	Sampling Method	Moisture, %	Dry Density, pcf	Blow Counts	Blow Count Histogram	Ground Water	Soil Lithology	Soil Lithology Description	Notes
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LOG OF TEST BORING

BOREHOLE NUMBER

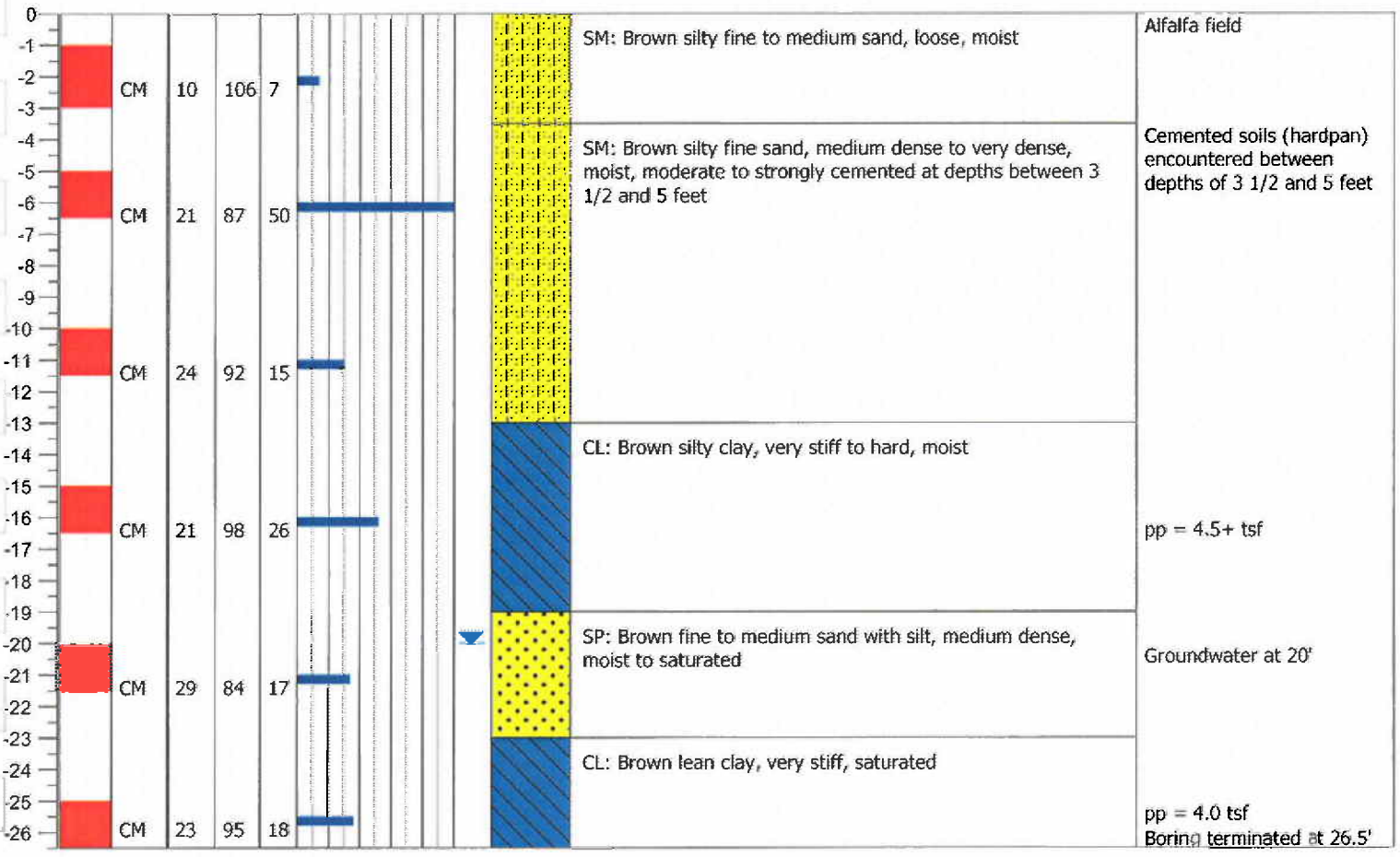
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PROJECT NUMBER: **LGE090554**
PROJECT NAME: **CenterPoint Intermodal Center**
LOCATION: **Lathrop, California**
DRILLING EQUIP.: **CME 75 Drill Rig**

DATE DRILLED: **08-03-2009**
GROUND SURFACE ELEVATION: **0.0** Feet

PLATE 3

Depth, ft.	Sample	Sampling Method	Moisture, %	Dry Density, pcf	Blow Counts	Blow Count Histogram	Ground Water	Soil Lithology	Soil Lithology Description	Notes
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LOG OF TEST BORING

BOREHOLE NUMBER

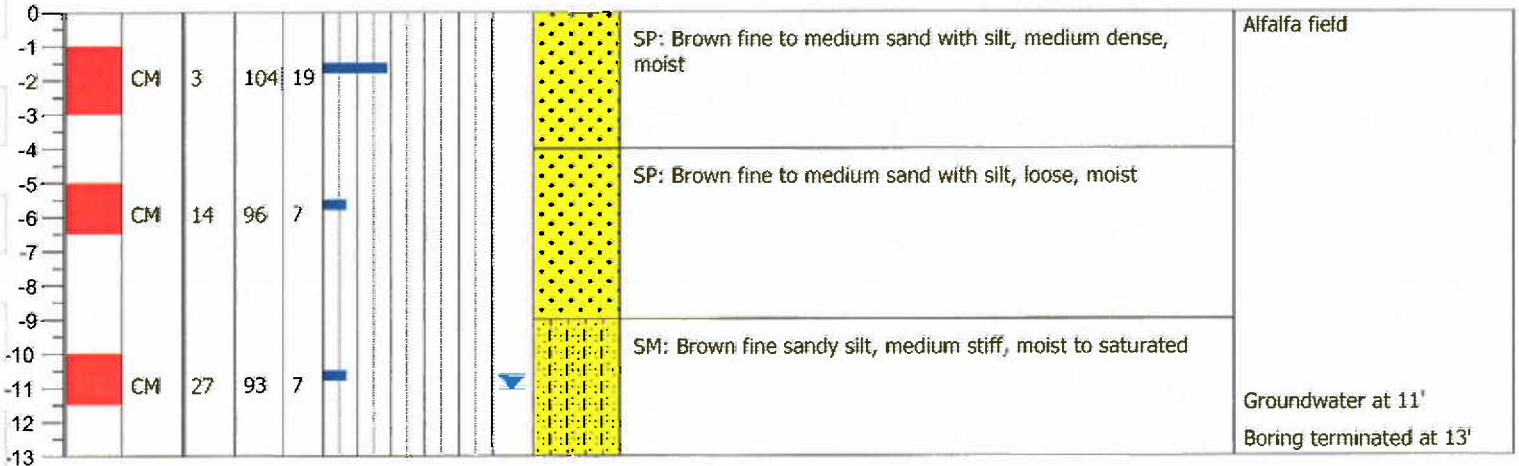
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PROJECT NUMBER: **LGE090554**
 PROJECT NAME: **CenterPoint Intermodal Center**
 LOCATION: **Lathrop, California**
 DRILLING EQUIP.: **CME 75 Drill Rig**

DATE DRILLED: **08-03-2009**
 GROUND SURFACE ELEVATION: **0.0** Feet

PLATE 4

Depth, ft.	Sample	Sampling Method	Moisture, %	Dry Density, pcf	Blow Counts	Blow Count Histogram	Ground Water	Soil Lithology	Soil Lithology Description	Notes
------------	--------	-----------------	-------------	------------------	-------------	----------------------	--------------	----------------	----------------------------	-------



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LOG OF TEST BORING

BOREHOLE NUMBER

B4

PROJECT NUMBER: **LGE090554**

DATE DRILLED: **08-03-2009**

PROJECT NAME: **CenterPoint Intermodal Center**

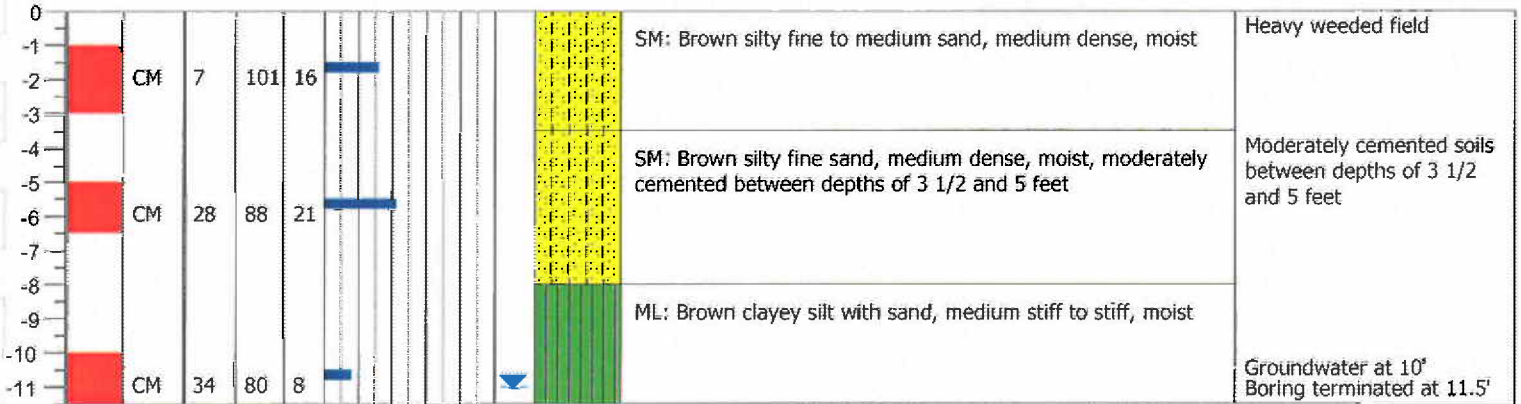
GROUND SURFACE ELEVATION: **0.0** Feet

LOCATION: **Lathrop, California**

DRILLING EQUIP.: **CME 75 Drill Rig**

PLATE 5

Depth, ft.	Sample	Sampling Method	Moisture, %	Dry Density, pcf	Blow Counts	Blow Count Histogram	Ground Water	Soil Lithology	Soil Lithology Description	Notes
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LOG OF TEST BORING

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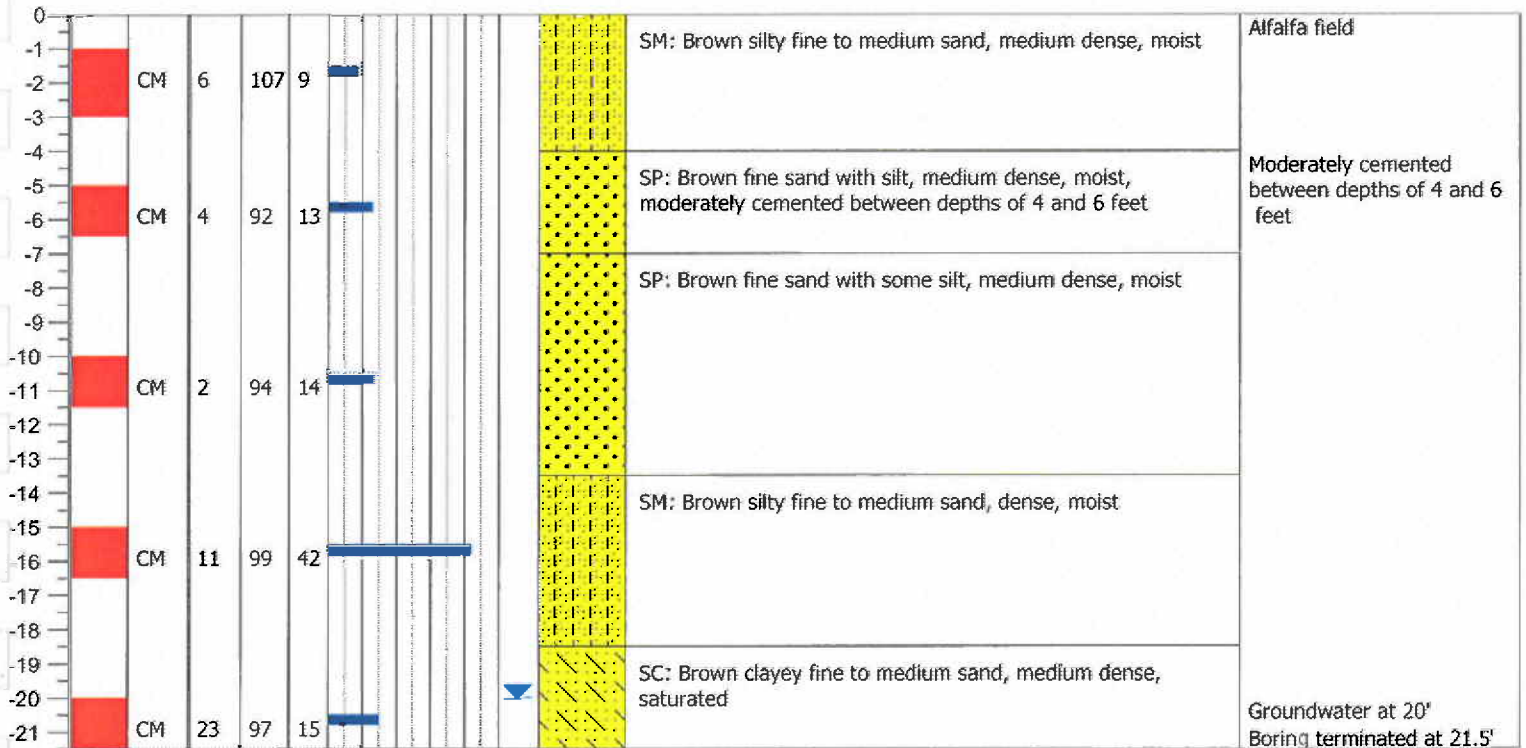
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PROJECT NUMBER: **LGE090554**
 PROJECT NAME: **CenterPoint Intermodal Center**
 LOCATION: **Lathrop, California**
 DRILLING EQUIP.: **CME 75 Drill Rig**

DATE DRILLED: **08-03-2009**
 GROUND SURFACE ELEVATION: **0.0** Feet

PLATE 6

Depth, ft.	Sample	Sampling Method	Moisture, %	Dry Density, pcf	Blow Counts	Blow Count Histogram	Ground Water	Soil Lithology	Soil Lithology Description	Notes
------------	--------	-----------------	-------------	------------------	-------------	----------------------	--------------	----------------	----------------------------	-------



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LOG OF TEST BORING

BOREHOLE NUMBER

Profile P2

PROJECT NUMBER: **LGE090554**

DATE DRILLED: **08-03-2009**

PROJECT NAME: **CenterPoint Internodal Center**

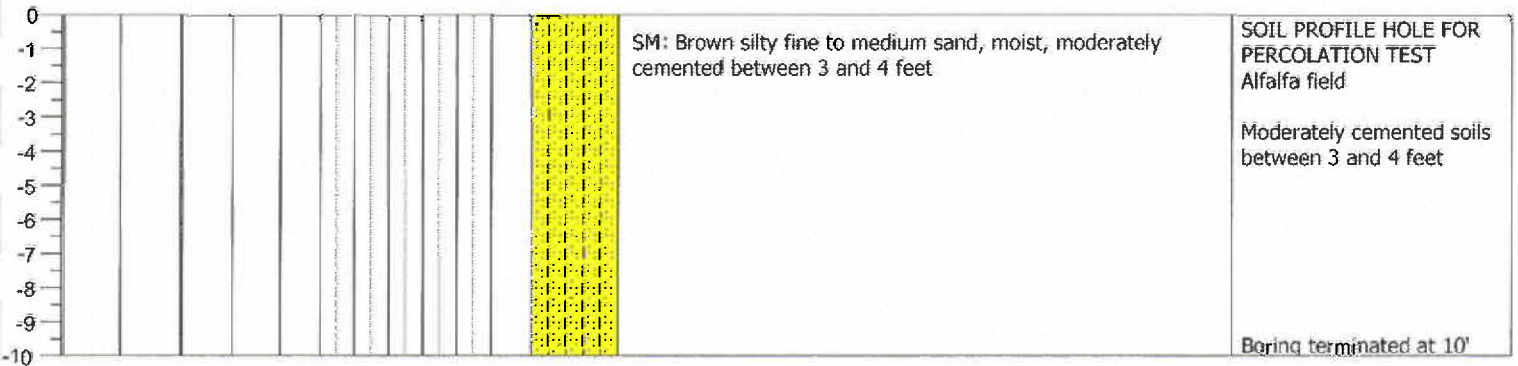
GROUND SURFACE ELEVATION: **0.0** Feet

LOCATION: **Lathrop, California**

DRILLING EQUIP.: **CME 75 Drill Rig**

PLATE 8

Depth, ft.	Sample	Sampling Method	Moisture, %	Dry Density, pcf	Blow Counts	Blow Count Histogram	Ground Water	Soil Lithology	Soil Lithology Description	Notes
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LOG OF TEST BORING

BOREHOLE NUMBER

Profile P3

PROJECT NUMBER: **LGE090554**

DATE DRILLED: **08-03-2009**

PROJECT NAME: **CenterPoint Intermodal Center**

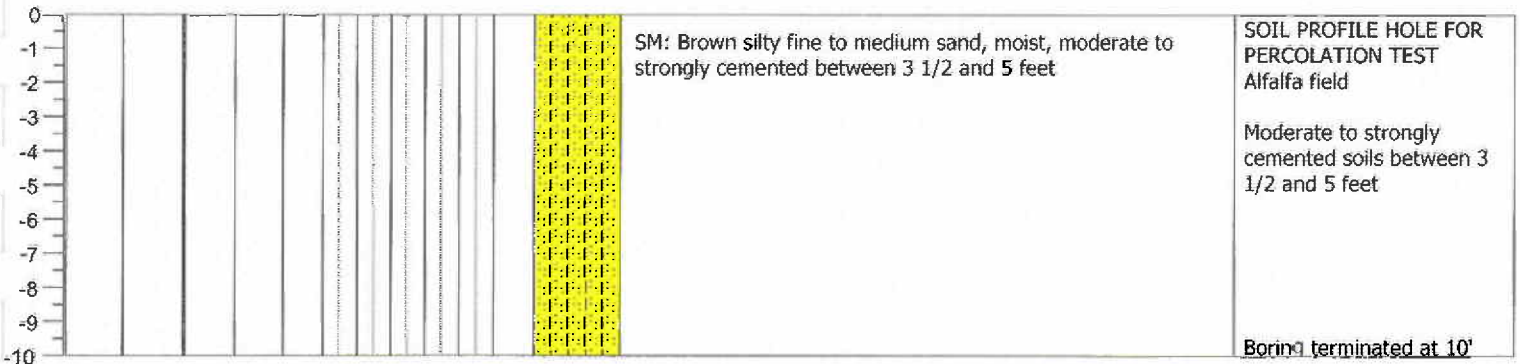
GROUND SURFACE ELEVATION: **0.0** Feet

LOCATION: **Lathrop, California**

DRILLING EQUIP.: **CME 75 Drill Rig**

PLATE 9

Depth, ft.	Sample	Sampling Method	Moisture, %	Dry Density, pcf	Blow Counts	Blow Count Histogram	Ground Water	Soil Lithology	Soil Lithology Description	Notes
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UNIFIED SOIL CLASSIFICATION SYSTEM AND BORING LOG SYMBOLS

DESCRIPTION		MAJOR DIVISIONS		
GW	Well-graded gravels, gravel sand mixtures, little or no fines.	Clean gravels (little or no fines)	Gravel and gravelly soils	Coarse grained soils more than 50% larger than No. 200 sieve
GP	Poorly-graded gravels, gravel sand mixtures, little or no fines		More than 50% of coarse fraction retained on No. 4 sieve	
GM	Silty gravels, gravel-sand-clay mixtures	Sands with appreciable amount of fines	Sands and sandy soils	
GC	Clayey gravels, gravel-sand-clay mixtures			
SW	Well-graded sands, gravelly sands, little or no fines	Clean sand (little or no fines)	More than 50% of coarse fraction passing No. 4 sieve	
SP	Poorly-graded sands, gravelly sands, little or no fines			
SM	Silty sands, sand-silt mixtures	Sands with appreciable amount of fines	More than 50% of coarse fraction passing No. 4 sieve	
SC	Clayey sands, sand-silt mixtures			
ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity	Liquid limit less than 50	Silts and clays	Fine grained soils more than 50% smaller than No. 200 sieve
CL	Inorganic clays of low to medium plasticity, gravelly clays, lean clays			
OL	Organic silts and organic silty clays of low plasticity			
MH	Inorganic silts, micaceous or diatomaceous fine sand or silty soils	Liquid limit greater than 50	Silts and clays	
CH	Inorganic clays of high plasticity, fat clays			
OH	Organic clays of medium to high plasticity, organic silts			
PT	Peat, humas swamp soils with high organic content	Highly organic soils		

DEPTH (FEET)	SAMPLE	SAMPLE TYPE	TEST TYPE	NOTES
	PS	Push Sample	Plasticity Grain Size Analysis Uniformity Coefficient Coefficient of Gradation Coefficient of Consolidation Specific Gravity Shrink/Swell Direct Shear Unconfined Compression Triaxial Compression Pocket Penetrometer Torvane Shear Consolidations	pi
	SPT	Drive Sample , 2.0" o.d., 1.38" i.d., sampler driven with 140 lb. hammer, 30" drop (Standard Penetration Test, SPT).		gr
	CM	Drive Sample , 2.5" o.d., 1.92" i.d., sampler driven with 140 lb. hammer, 30" drop, with 6" tube liners (California Modified, CM).		Cu
	ES	Ely Sample , Used to determine unit weight.		Cc
	HS	Hand Sampler , 2.0" o.d. sampler driven with 10 lb. hammer, 18" drop, with 4" tube liners.		Cv
	GS	Grab Sample , disturbed sample taken from auger tailings and sealed in plastic bag.		sg
				s/s
			ds	
			uc	
			tx	
			p	
			ts	
			c	

Plate Number 10

