APPENDIX D: GEOTECHNICAL INVESTIGATION

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
APN'S 1013-211-21 AND 1013-211-22
NORTHWEST OF FRANCIS AVENUE
AND YORBA AVENUE
CITY OF CHINO, CALIFORNIA

Prepared For:

CHINO FRANCIS ESTATES, LLC
C/O BORSTEIN ENTERPRISES
11766 Wilshire Boulevard, Suite 820
Los Angeles, California 90025

Project No. 10557.004

July 16, 2019

Leighton and Associates, Inc.

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Chino Francis Estates, LLC
c/o Borstein Enterprises
11766 Wilshire Boulevard, Suite 820
Los Angeles, California 90025

Attention: Mr. Erik Pfahler

Subject: Geotechnical Investigation
Proposed Residential Development
APNs 1013-211-21 and 1013-211-22
Northwest of Francis Avenue and Yorba Avenue
City of Chino, California

In response to your request, Leighton and Associates, Inc. has conducted a geotechnical investigation for the proposed residential development to be located on APN 1013-211-21 and 1013-211-22, northwest of Francis Avenue and Yorba Avenue, in the City of Chino, California. This report utilizes data from our referenced report (Leighton, 2016), which you provided a copy of.

Based on the results of our study, it is our professional opinion that the proposed development of the site is feasible from a geotechnical perspective, based on the current preliminary project plans. The accompanying geotechnical report presents a summary of the encountered subsurface conditions and provides geotechnical conclusions and recommendations.

We appreciate the opportunity to work with you on the development of this project. If you have any questions regarding this report, please call us at your convenience.

Respectfully submitted,

LEIGHTON AND ASSOCIATES, INC.

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1.0 INTRODUCTION

1.1 Site Location and Description

The subject property consists of approximately 12 acres and was recently utilized as grazing land for a neighboring goat farm. The property is roughly divided into thirds, with the western third occupied by numerous small rectangular concrete pads (presumably residential structures all of which had been demolished by the mid-1990s) and one maintenance shed used for the storage of materials associated with the goats currently grazing the site. The middle third is occupied by numerous elongated concrete slabs and a few animals pens associated with a former rabbit farm (present between the 1960s and mid 1990s), bee hives, and an empty maintenance shed. The eastern third of the site is primarily vacant, with a residence and several associated structures and a pool. The property drains gently to the south.

1.2 Proposed Development

The preliminary plans depict a residential development with 46 lots planed for single family residential homes, as well as drainage, utility, street, sidewalk, park, landscape and associated improvements. We expect relatively shallow cuts and fills to achieve design grade (generally on the order of 5 feet or less).

1.3 Purpose of Investigation

This report presents the results of the subsurface geotechnical exploration for the proposed development located northwest of Francis Avenue and Yorba Avenue in Chino, California (Figure 1). The purpose of this study has been to evaluate the general geotechnical conditions at the site with respect to the proposed development and provide preliminary geotechnical recommendations for design and construction.

The geotechnical exploration included hollow-stem auger soil borings, laboratory testing and geotechnical analysis to evaluate the existing conditions and develop the recommendations contained in this report. We also conducted infiltration testing to evaluate general infiltration characteristics at the depths tested for water quality basin design.

1.4 Scope of Investigation

The scope of our study has included the following tasks:

- Background Review: We reviewed available, relevant geotechnical geologic maps and reports and aerial photographs available from our in-house library. This included a review of geotechnical reports previously prepared for the site.

- Field Exploration: Previous subsurface explorations were performed on the site by Leighton in December of 2013. A total of 5 exploratory soil borings (LB-1 through LB-5) were logged and sampled to evaluate subsurface conditions. The borings were drilled to depths ranging from 21.5 to 51.5 feet below the existing ground surface (bgs) by a subcontracted drill rig operator. The borings were logged by our field representative during drilling. Relatively undisturbed soil samples were obtained at selected intervals within the borings using a California Ring Sampler. Standard Penetration Tests (SPT) were conducted at selected depths and samples were obtained. Representative bulk soil samples were also collected at shallow depths from the borings.

  Well permeameter tests were previously conducted at the 5 boring locations on the site (LB-1 through LB-5) to evaluate general infiltration rates of the subsurface soils at the depths and locations tested. The well permeameter tests were conducted based on the USBR 7300-89 method. The tests were conducted at depths of about 5 to 6 feet bgs to estimate the infiltration rate for use of shallow infiltration trenches.

  All excavations were backfilled with the soil cuttings. Logs of the geotechnical borings and the well permeameter test results are presented in Appendix B. Approximate boring and well permeameter test locations are shown on the accompanying Test Location Map, Figure 2.

- Geotechnical Laboratory Testing: Geotechnical laboratory tests were previously conducted (Leighton, 2019) on selected relatively undisturbed and bulk soil samples obtained during our field investigation. This laboratory testing program was designed to evaluate engineering characteristics of site soils. Laboratory tests conducted during this investigation include:

  - In situ moisture content and dry density
  - Maximum dry density and optimum moisture content
  - Sieve analysis for grain-size distribution

- Swell and collapse potential
- Water-soluble sulfate concentration
- Resistivity, chloride content and pH

The in situ moisture content and dry density test results are shown on the boring logs, Appendix B. The other laboratory test results are presented in Appendix C.

- Engineering Analysis: Data obtained from our background review, previous field exploration and geotechnical laboratory testing was evaluated and analyzed to develop geotechnical conclusions and provide preliminary recommendations presented in this report.

- Report Preparation: Results of the geotechnical exploration have been summarized in this report, presenting our findings, conclusions and preliminary geotechnical recommendations for design and construction of the proposed residential development.

2.0 FINDINGS

2.1 Regional Geologic Conditions

The site is located within the Chino Basin in the northern portion of the Peninsular Range geomorphic province of California. Major structural features surround this region, including the Cucamonga fault and the San Gabriel Mountains to the north, the Chino fault and Puente/Chino Hills to the west, and the San Jacinto fault to the east. This is an area of large-scale crustal disturbance as the relatively northwestward-moving Peninsular Range Province collides with the Transverse Range Province (San Gabriel and San Bernardino Mountains) to the north. Several active or potentially active faults have been mapped in the region and are believed to accommodate compression associated with this collision. The site is underlain by younger alluvial soil deposits eroded from the mountains surrounding the basin and deposited in the site vicinity.

2.2 Subsurface Soil Conditions

Based upon our review of pertinent geotechnical literature and our previous subsurface exploration, the site is underlain by alluvial soil deposits mantled in areas of the site by minor amounts of goat manure. The manure was generally less than approximately one inch thick. The alluvial soil encountered within our excavations generally consisted of combinations of sand and silt, with some gravel interspersed. The soil was generally moist and medium dense. The in-situ moisture content within the upper approximately 15 feet generally ranged from 1 to 10 percent. More detailed descriptions of the subsurface soil are presented on the boring logs.

2.2.1 Compressible and Collapsible Soil

Soil compressibility refers to a soil's potential for settlement when subjected to increased loads as from a fill surcharge. Based on our investigation, the native soil encountered is generally considered slightly to moderately compressible. Partial removal and recompaction of this material under shallow foundations is recommended to reduce the potential for adverse total and differential settlement of the proposed improvements.

Collapse potential refers to the potential settlement of a soil under existing stresses upon being wetted. Test results indicate that the alluvial soil within the upper 10 feet onsite has a minor collapse potential.

2.2.2 Expansive Soils

Expansive soils contain significant amounts of clay particles that swell considerably when wetted and shrink when dried. Foundations constructed on these soils are subjected to large uplifting forces caused by the swelling. Without proper measures taken, heaving and cracking of both building foundations and slabs-on-grade could result.

The near surface soils consist of sands and silty sands. Based on our observations conditions and experience in the area, the near-surface soil is generally expected to have a very low expansion potential.

2.2.3 Sulfate Content

Water-soluble sulfates in soil can react adversely with concrete. However, concrete in contact with soil containing sulfate concentrations of less than 0.1 percent by weight is considered to have negligible sulfate exposure based on the American Concrete Institute (ACI) provisions, adopted by the 2016 CBC (Chapter 19), and ACI 318-14.

A near-surface soil sample was tested during the previous investigation for soluble sulfate content. The results of this test indicate a sulfate content of less than 0.01 percent by weight, indicating negligible sulfate exposure. Recommendations for concrete in contact with the soil are provided in Section 3.11.

2.2.4 Resistivity, Chloride and pH

Soil corrosivity to ferrous metals can be estimated by the soil's electrical resistivity, chloride content and pH. In general, soil having a minimum resistivity less than 1,000 ohm-cm is considered severely corrosive. Soil with a chloride content of 500 parts-per-million (ppm) or more is considered corrosive to ferrous metals.

As a screening for potentially corrosive soil, representative soil samples were tested during the previous investigation to determine minimum resistivity, chloride content, and pH. The tests indicated a minimum resistivity of 8,100 ohm-cm, chloride content of 200 ppm, and pH of 6.9. Based on the chloride content, the onsite soil is considered moderately corrosive to ferrous metals.

2.3 Groundwater

Groundwater was not encountered in our borings excavated to a maximum depth of 51.5 feet below the existing ground surface (bgs). Historical groundwater mapping indicates that groundwater was approximately 150 feet bgs in 1933 (CDWR, 1970). Recent data from the California Department of Water Resources indicates groundwater levels no higher than 200 feet bgs in the area (CDWR, 2013). Based on this, groundwater has historically been deep, and shallow groundwater is not expected at the site.

2.4 Faulting and Seismicity

Our review of available in-house literature indicates that there are no known active faults traversing the site. The closest known active or potentially active fault is the Chino-Elsinore fault, located approximately 3 miles southwest of the site.

The principal seismic hazard that could affect the site is ground shaking resulting from an earthquake occurring along several major active or potentially active faults in southern California. The known regional active and potentially active faults that could produce the most significant ground shaking at the site include the Chino-Elsinore, San Jose, Cucamonga, Sierra Madre, Whittier, Elsinore-Glen Ivy, and Elysian Park Thrust faults.

The Peak Horizontal Ground Acceleration (PHGA) and hazard deaggregation were estimated using the United States Geological Survey's (USGS) 2008 Interactive Deaggregations utility. The results of this analysis indicate that the predominant modal earthquake has a PHGA of 0.76g with magnitude of approximately 6.6 (M_W) at a distance on the order of 7 kilometers for the Maximum Considered Earthquake (2% probability of exceedance in 50 years). Based on this, the corresponding PHGA for the design earthquake (2/3 of the MCE) is 0.51g.

We also estimated the design PHGA based on the 2016 California Building Code Section 1613 and 1803.5.12. The calculated peak ground acceleration is 0.67g. Based on ASCE 7-10 Equation 11.8-1, the site amplification factor (F_{PGA}) is 1, and the site modified peak ground acceleration (PGA_M) is 0.67g.

Based on these results, we have selected a design PHGA of 0.67g for seismic analysis of the onsite soils (seismic settlement).

2.5 Secondary Seismic Hazards

In general, secondary seismic hazards for sites in the region could include soil liquefaction, earthquake-induced settlement, lateral displacement, landsliding, and earthquake-induced flooding. The potential for secondary seismic hazards at the site is discussed below.

2.5.1 Liquefaction Potential

Liquefaction is the loss of soil strength or stiffness due to a buildup of pore-water pressure during severe ground shaking. Liquefaction is associated primarily with loose (low density), saturated, fine-to-medium grained, cohesionless soils. As the shaking action of an earthquake progresses, the soil grains are rearranged and the soil densifies within a short period of time. Rapid densification of the soil results in a buildup of pore-water pressure. When the pore-water pressure approaches the total overburden pressure, the soil reduces greatly in strength and temporarily behaves similarly to a fluid. Effects of liquefaction can include sand boils, settlement, and bearing capacity failures below structural foundations.

The State of California has not prepared liquefaction hazard maps for this area. San Bernardino County (2010) does not show the site in a zone of susceptibility for liquefaction.

Based on our study, current groundwater levels are deeper than 51.5 feet bgs and historic high groundwater levels are deeper than 150 feet bgs. As such, the potential for liquefaction at the site is very low.

2.5.2 Seismically Induced Settlement

During a strong seismic event, seismically induced settlement can occur within loose to moderately dense, dry or saturated granular soil. Settlement caused by ground shaking is often nonuniformly distributed, which can result in differential settlement.

Considering our recommended overexcavation recommendations, the potential total settlement resulting from seismic loading is considered low (less than 1 inch) for this site. Differential settlement resulting from seismic loading is generally assumed to be one-half of the total seismically induced settlement over a distance of 40 feet. Seismic settlement analysis is provided in Appendix D.

2.5.3 Seismically Induced Landslides

The site is generally level without significant slopes. This site is not considered susceptible to static slope instability or seismically induced landslides.

3.0 CONCLUSIONS AND RECOMMENDATIONS

Based on this study, construction of the proposed residential development is feasible from a geotechnical standpoint. No severe geologic or soils related issues were identified that would preclude development of the site for the proposed improvements. The most significant geotechnical issues at the site are those related to the potential for strong seismic shaking and potentially compressible soils. Good planning and design of the project can limit the impact of these constraints. Remedial recommendations for these and other geotechnical issues are provided in the following sections.

The site is not expected to be prone to adverse effects of slope instability or adverse differential settlement from cut/fill transitions (significant cuts and fills are not proposed).

Although not identified, abandoned septic tanks, seepage pits, or other buried structures, trash pits, or items related to past site uses may be present. If such items were encountered during grading, they would require further evaluation and special consideration.

3.1 General Earthwork and Grading

All grading should be performed in accordance with the General Earthwork and Grading Specifications presented in Appendix E, unless specifically revised or amended below or by future recommendations based on final development plans.

3.1.1 Site Preparation

Prior to construction, the site should be cleared of vegetation, trash and debris and existing concrete slabs and foundations, which should be disposed of offsite. Any underground obstructions should be removed as should large tress and their root systems. Resulting cavities should be properly backfilled and compacted. Efforts should be made to locate existing utility lines. Those lines should be removed or rerouted if they interfere with the proposed construction, and the resulting cavities should be properly backfilled and compacted. Trees should be removed.

3.1.2 Removal of Manure, Organic-Rich Soil and Uncontrolled Artificial Fill

Prior to overexcavation and recompaction of the onsite alluvial soil, all manure should be cleared and removed from the site. Heavy concentrations of organic-rich soil (containing visible organic matter or containing an organic content of 2 percent by weight or more) should be removed.

Removal and disposal of manure and organic-rich soil should be observed by Leighton and Associates. Organic content testing should be performed during removal to guide disposal operations.

In addition to the above, prior to overexcavation and recompaction of the onsite alluvial soil, any clean uncontrolled artificial fill should be removed and may be used as compacted fill for the project.

If suitable open space areas are available without proposed structures, such as a park site, it may be possible to place organic-rich soil and minor amounts of manure as non-structural fill in those areas, provided this is acceptable to the local reviewing agency. If this is done, we suggest the manure and organic-rich soils be mixed with clean soil to reduce the overall organic content and a clean soil cap be provided above the organic-rich soil.

3.1.3 Overexcavation and Recompaction

To reduce the potential for adverse differential settlement of the proposed improvements, the underlying subgrade soil should be prepared in such a manner that a uniform response to the applied loads is achieved. For structures with shallow foundations, we recommend that onsite alluvial soils be overexcavated and recompacted to a minimum depth of 3 feet below the bottom of the proposed footings or 5 feet below existing grade, whichever is deeper. Overexcavation and recompaction should extend a minimum horizontal distance of 5 feet from perimeter edges of the proposed footings.

Local conditions may require that deeper overexcavation be performed; such areas should be evaluated by Leighton during grading.

Areas outside these overexcavation limits planned for asphalt or concrete pavement, flatwork, and site walls, and areas to receive fill should be overexcavated to a minimum depth of 24 inches below the existing ground surface or 12 inches below the proposed subgrade, whichever is deeper.

After completion of the overexcavation, and prior to fill placement, the exposed surfaces should be scarified to a minimum depth of 6 inches, moisture conditioned to or slightly above optimum moisture content, and recompacted to a minimum 90 percent relative compaction, relative to the ASTM D 1557 laboratory maximum density.

These recommendations should be reviewed once a grading plan is available.

3.1.4 Fill Placement and Compaction

Manure and organic-rich soil is considered unsuitable for support of the proposed improvements, and will require offsite disposal or placement in non-structural areas. All structural fill should be visibly free of organic matter or should have a total organic matter content of less than 2.0 percent.

Onsite soil to be used for compacted structural fill should also be free of debris and oversized material (greater than 8 inches in largest dimension). Any soil to be placed as fill, whether onsite or imported material, should be reviewed and possibly tested by Leighton.

All fill soil should be placed in thin, loose lifts, moisture conditioned, as necessary, and compacted to a minimum 90 percent relative compaction. Relative compaction should be determined in accordance with ASTM Test Method D1557. Aggregate base for pavement should be compacted to a minimum of 95 percent relative compaction.

3.1.5 Import Fill Soil

Import soil to be placed as fill should be geotechnically accepted by Leighton. Preferably at least 3 working days prior to proposed import to the site, the contractor should provide Leighton pertinent information of the proposed import soil, such as location of the soil, whether stockpiled or native in place, and pertinent geotechnical reports if available. We recommend that a Leighton representative visit the proposed import site to observe the soil conditions and obtain representative soil samples. Potential issues may include soil that is more expansive than onsite soil, soil that is too wet, soil that is too rocky or too dissimilar to onsite soils, oversize material, organics, debris, etc.

3.1.6 Shrinkage and Subsidence

The change in volume of excavated and recompacted soil varies according to soil type and location. This volume change is represented as a percentage increase (bulking) or decrease (shrinkage) in volume of fill after removal and recompaction. Subsidence occurs as in-place soil (e.g., natural ground) is moisture-conditioned and densified to receive fill, such as in processing an overexcavation bottom. Subsidence is in addition to shrinkage due to recompaction of fill soil. Field and laboratory data used in our calculations included laboratory-measured maximum dry densities for soil types encountered at the subject site, the measured in-place densities of soils encountered and our experience. We preliminarily estimate the following earth volume changes will occur during grading:

Shrinkage	Approximately 15 +/- 5 percent
Subsidence (overexcavation bottom processing)	Approximately 0.15 feet

It should be noted that these values do not account for removal of manure and organic-rich soil.

The level of fill compaction, variations in the dry density of the existing soils and other factors influence the amount of volume change. Some adjustments to earthwork volume should be anticipated during grading of the site.

3.1.7 Rippability and Oversized Material

Oversized material (rock or rock fragments greater than 8 inches in dimension) was not observed during our investigation. Oversized material should not be used within structural fill areas.

3.2 Shallow Foundation Recommendations

Overexcavation and recompaction of the footing subgrade soil should be performed as detailed in Section 3.1. The following recommendations are based on the onsite soil conditions and soils with a very low expansion potential.

3.2.1 Minimum Embedment and Width

Based on our preliminary investigation, footings should have a minimum embedment of 18 inches, with a minimum width of 24 and 12 inches for isolated and continuous footings, respectively.

3.2.2 Allowable Bearing

An allowable bearing pressure of 1,800 pounds-per-square-foot (psf) may be used, based on the minimum embedment depth and width above. This allowable bearing value may be increased by 250 psf per foot increase in depth or width to a maximum allowable bearing pressure of 2,500 psf. If higher bearing pressures are required, this should be reviewed on a case-by-case basis. These allowable bearing pressures are for total dead load and sustained live loads. Footing reinforcement should be designed by the structural engineer.

3.2.3 Lateral Load Resistance

Soil resistance available to withstand lateral loads on a shallow foundation is a function of the frictional resistance along the base of the footing and the passive resistance that may develop as the face of the structure tends to move into the soil. The frictional resistance between the base of the foundation and the subgrade soil may be computed using a coefficient of friction of 0.30. The passive resistance may be computed using an allowable equivalent fluid pressure of 240 pounds per cubic foot (pcf), assuming there is constant contact between the footing and undisturbed soil. The coefficient of friction and passive resistance may be combined without further reduction.

3.2.4 Increase in Bearing and Friction - Short Duration Loads

The allowable bearing pressure and coefficient of friction values may be increased by one-third when considering loads of short duration, such as those imposed by wind and seismic forces.

3.3 Recommendations for Slabs-On-Grade

Concrete slabs-on-grade should be designed by the structural engineer in accordance with the current CBC for a soil with a very low expansion potential. Where conventional light floor loading conditions exist, the following minimum recommendations should be used. More stringent requirements may be required by local agencies, the structural engineer, the architect, or the CBC. Laboratory testing should be conducted at finish grade to evaluate the Expansion Index (EI) of near-surface subgrade soils. Slabs-on-grade should have the following minimum recommended components:

- Subgrade Moisture Conditioning: The subgrade soil should be moisture conditioned to at least 2 percent above optimum moisture content to a minimum depth of 18 inches prior to placing steel or concrete.

- Moisture Vapor Retarder: A minimum of a 10-mil vapor retarder should be placed below slabs where moisture-sensitive floor coverings or equipment is planned. Since moisture will otherwise be transmitted up from the soil through the concrete, it is important that an intact vapor retarder be installed. We recommend that the vapor retarder meet the requirements of ASTM E1745 and be installed per ASTM E1643. The structural engineer should specify pertinent concrete design parameters and moisture migration prevention measures, such as whether a sand blotter layer should be placed over the vapor retarder. Gravel or other protruding objects that could puncture the moisture retarder should be removed from the subgrade prior to placing the vapor retarder, or a stronger vapor retarder intended for the specific conditions present can be used.

- Concrete Thickness: Slabs-on-grade should be at least 4 inches thick. Reinforcing steel should be designed by the structural engineer, but as a minimum should be No. 4 rebar placed at 18 inches on center, each direction, mid-depth in the slab.

Minor cracking of the concrete as it cures, due to drying and shrinkage is normal and should be expected. However, cracking is often aggravated by a high water/cement ratio, high concrete temperature at the time of placement, small nominal aggregate size, and rapid moisture loss due to hot, dry, and/or windy weather conditions during placement and curing. Cracking due to temperature and moisture fluctuations can also be expected. Low slump concrete can reduce the potential for shrinkage cracking. Additionally, our experience indicates that reinforcement in slabs and foundations can generally reduce the potential for concrete cracking. The structural engineer should consider these components in slab design and specifications.

Moisture retarders can reduce, but not eliminate moisture vapor rise from the underlying soils up through the slab. Floor covering manufacturers should be consulted for specific recommendations.

Leighton does not practice in the field of moisture vapor transmission evaluation, since this is not specifically a geotechnical issue. Therefore, we recommend that a qualified person, such as the flooring subcontractor and/or structural engineer, be consulted with to evaluate the general and specific moisture vapor transmission paths and any impact on the proposed construction. That person should provide recommendations for mitigation of potential adverse impact of moisture vapor transmission on various components of the structures as deemed appropriate.

3.4 Seismic Design Parameters

Seismic parameters presented in this report should be considered during project design. In order to reduce the effects of ground shaking produced by regional seismic events, seismic design should be performed in accordance with the most recent edition of the California Building Code (CBC). The following data should be considered for the seismic analysis of the subject site:

2016 CBC Categorization/Coefficient	Design Value
Site Longitude (decimal degrees)	-117.704
Site Latitude (decimal degrees)	34.042
Site Class Definition (ASCE 7 Table 20.3-1)	D
Mapped Spectral Response Acceleration at 0.2s Period, Ss (Figure 1613.3.1(1))	1.771 g
Mapped Spectral Response Acceleration at 1s Period, S1 (Figure 1613.3.1(2))	0.628 g
Short Period Site Coefficient at 0.2s Period, Fa (Table 1613.3.3(1))	1.0
Long Period Site Coefficient at 1s Period, Fv (Table 1613.3.3(2)	1.5
Adjusted Spectral Response Acceleration at 0.2s Period, SMS (Eq. 16-37)	1.771 g
Adjusted Spectral Response Acceleration at 1s Period, SM1 (Eq. 16-38)	0.941 g
Design Spectral Response Acceleration at 0.2s Period, SDS (Eq. 16-39)	1.181 g
Design Spectral Response Acceleration at 1s Period, SD1 (Eq. 16-40)	0.628 g

3.5 Retaining Walls

We recommend that retaining walls be backfilled with very low expansive soil and constructed with a backdrain in accordance with the recommendations provided on Figure 3 (rear of text). Using expansive soil as retaining wall backfill will result in higher lateral earth pressures exerted on the wall. Based on these recommendations, the following parameters may be used for the design of conventional retaining walls:

Static Equivalent Fluid Weight (pcf)	
Condition	Level Backfill
Active	35 pcf
At-Rest	55 pcf
Passive	240 pcf (allowable) (Maximum of 3,500 psf)

The above values do not contain an appreciable factor of safety unless noted, so the structural engineer should apply the applicable factors of safety and/or load factors during design, as specified by the California Building Code.

Cantilever walls that are designed to yield at least 0.001H, where H is equal to the wall height, may be designed using the active condition. Rigid walls and walls braced at the top should be designed using the at-rest condition (restrained from lateral movement).

Cantilever walls for temporary excavations should be designed using an active pressure of 35 pcf (equivalent fluid pressure). If excavations are braced at the top and at specific design intervals, the earth pressure for temporary shoring may be approximated by a rectangular soil pressure distribution with the pressure per foot of width equal to 25H, where H is equal to the depth of the excavation being shored.

Passive pressure is used to compute soil resistance to lateral structural movement. In addition, for sliding resistance, a frictional resistance coefficient of 0.3 may be used at the concrete and soil interface. The lateral passive resistance should be taken into account only if it is ensured that the soil providing passive resistance, embedded against the foundation elements, will remain intact with time.

Seismic incremental loads need not to be added to walls retaining less than 6 feet, with level backfill. For walls retaining more than 6 feet, an incremental seismic earth pressure of 25H psf, where H is the retaining wall height in feet, should be applied for design in addition to static earth and surcharge pressures.

In addition to the above lateral forces due to retained earth, surcharge due to improvements, such as an adjacent structure or traffic loading, should be considered in the design of the retaining wall. Loads applied within a 1:1 projection from the surcharging structure on the stem of the wall should be considered in the design.

A soil unit weight of 120 pcf may be assumed for calculating the actual weight of the soil over the wall footing.

3.6 Infiltration Design

Five well permeameter tests (LB-1 through LB-5) were conducted to estimate the infiltration rate in various parts of the site. The well permeameter tests were conducted at depths between 4 and 6 feet below ground surface.

Well permeameter tests are useful for field measurements of soil infiltration rates, and is suited for testing when the design depth of the basin is deeper than current existing grades. The test consists of excavating a boring to the depth of the test (or deeper if it is partially backfilled with soil and a bentonite plug with a thin soil covering is placed just below the design test elevation). A layer of clean sand is placed in the boring bottom to support a float mechanism and temporary perforated well casing pipe. In addition, sand is poured around the outside of the well casing within the test zone to prevent the boring from caving/collapsing or eroding when water is added. The float mechanism, placed inside the casing, adds water stored in barrels at the top of the hole to the boring as water infiltrates into the soil, while maintaining a constant water head in the boring. The test was conducted based on the USBR 7300-89 test method.

The incremental infiltration rate as measured during intervals of the test is defined herein as the incremental flow rate of water infiltrated, divided by the surface area of the infiltration interface.

Small-scale infiltration rates were measured at the 5 well permeameter locations and ranged from approximately 0.3 to 13 inches per hour (no factor of safety applied). Infiltration at three of the five locations was too rapid to measure for normal test procedures. One of these three locations was selected based on the boring geology as the probable fastest infiltration location, and a modified test procedure was used to test the infiltration rate using a lower water surface head. The result of this test indicated an infiltration rate of 13 inches per hour. Infiltration test results are provided in Appendix B. These are raw values, before applying an appropriate factor of safety or correction factor. Based on these results, the onsite silty soils or soils with a higher fines content are not considered feasible for infiltration. Sandy soils with a low fines content are anticipated to have higher infiltration rates; however, sandy soils underlain by finer-grained soils are not considered suitable. Specific infiltration design information should be made available so testing representative to the final design conditions can be conducted. The small-scale infiltration rate should be divided by a correction factor of at least 2 for buried chambers and at least 3 for open basins, but the correction/safety factor may be higher based on project-specific aspects, based on *San Bernardino County Stormwater Program Technical Guidance Document for Water Quality Management Plans (WQMP)*.

We recommend that further evaluation be conducted after a design has been selected for an infiltration facility, since infiltration rates varied significantly across the site.

The infiltration rates described herein are for a clean, unsilted infiltration surface in native, sandy alluvial soil. These values may be reduced over time as silting of the basin or chamber occurs. Furthermore, if the basin or chamber bottom is allowed to be compacted by heavy equipment, this value is expected to be significantly reduced. Infiltration of water through soil is highly dependent on such factors as grain size distribution of the soil particles, particle shape, fines content, clay content, and density. Small changes in soil conditions, including density, can cause large differences in observed infiltration rates. Infiltration is not suitable in compacted fill.

It should be noted that during periods of prolonged precipitation, the underlying soils tend to become saturated to greater and greater depths/extents. Therefore, infiltration rates tend to decrease with prolonged rainfall. It is difficult to extrapolate longer-term, full-scale infiltration rates from small-scale tests, and as such, this is a significant source of uncertainty in infiltration rates.

Additional Review and Evaluation: Infiltration rates are anticipated to vary significantly based on the location and depth. Infiltration concepts should be discussed with Leighton as infiltration plans are being developed. Leighton should review all infiltration plans, including locations and depths of proposed facilities. Further testing should be conducted based on the design of infiltration facilities, particularly considering their type, depth and location.

General Design Considerations: The periodic flow of water carrying sediments in the basin or chamber, plus the introduction of wind-blown sediments and sediments from erosion of the basin side walls, can eventually cause the bottom of the basin or chamber to accumulate a layer of silt, which has the potential of significantly reducing the overall infiltration rate of the basin or chamber. Therefore, we recommend that significant amounts of silt/sediment not be allowed to flow into the facility within storm water, especially during construction of the project and prior to achieving a mature landscape on site. We recommend that an easily maintained, robust silt/sediment removal system be installed to pretreat storm water before it enters the infiltration facility.

As infiltrating water can seep within the soil strata nearly horizontally for long distances, it is important to consider the impact that infiltration facilities can have on nearby subterranean structures, such as basement walls or open excavations, whether onsite or offsite, and whether existing or planned. Any such nearby features should be identified and evaluated as to whether infiltrating water can impact these. Such features should be brought to Leighton's attention as they are identified.

Infiltration facilities should not be constructed adjacent to or under buildings. Setbacks should be discussed with Leighton during the planning process.

Infiltration facilities should be constructed with spillways or other appropriate means that would cause overfilling to not be a concern to the facility or nearby improvements.

For buried chambers that allow interior standing water, control/access manhole covers should not contain holes or should be screened to prevent mosquitos from entering the cambers.

Additional Design Considerations (Particularly for Open Basins): If open basins are planned, additional infiltration exploration and testing should be conducted, as the soils that will be exposed at the bottom of the basin are critical to the basin's success. Soils at the bottom of buried chambers are also important, but not as critical to their success, provided the infiltration chamber cuts through sufficiently granular soils.

In general, the rate of infiltration reduces as the head of water in the infiltration facility reduces, and it also reduces with prolonged periods of infiltration. As such, water typically infiltrates much faster near the beginning of and/or immediately after storm events than at times well after a storm when the water level in the facility has receded, since the infiltration rate is then slower due to both lower head and longer overall duration of infiltration. In open basins with compacted or silty bottoms, this could be problematic, in that, even if the basin had already infiltrated significant amounts of storm water, the lower several inches or feet of water could remain in the basin for an extended period of time, creating a prolonged open-water safety concern and potential for mosquitos. In a buried/covered infiltration chamber, these conditions would be of less concern.

Parks or play/recreation areas should not be constructed within basin bottoms or below the spillway level.

For open basins and swales, vegetation within the basin bottoms and sides is expected to help reduce erosion and help maintain infiltration rates.

Estimating infiltration rates, especially based on small-scale testing, is inexact and indefinite, and often involves known and unknown soil complexities, potentially resulting in a condition where actual infiltration rates of the completed facility are significantly less than design rates. In open infiltration basins, this could create nuisance water in the basin. As such, enhancements may be needed after completion of the basin if prolonged or frequent standing water is experienced. A potential basin enhancement, if needed, might be to install infiltration trenches or dry wells in the basin bottom to capture and infiltrate low flows and to help speed infiltration during/after storms; specific recommendations, such as minimum trench/dry well depth, would be developed based on conditions observed. Such a contingency should be anticipated for open basins.

Construction Considerations: We recommend that Leighton evaluate the infiltration facility excavations, to confirm that granular, undisturbed alluvium is exposed in the bottoms and sides. Additional excavation or evaluation may be required if silty or clayey soils are exposed.

It is critical to infiltration that the basin or chamber bottom not be allowed to be compacted during construction or maintenance; rubber-tired equipment and vehicles should not be allowed to operate on the bottom. We recommend that at least the bottom 3 feet of the basins or chambers be excavated with an excavator or similar.

If fill material is needed to be placed in the basin, such as due to removal of uncontrolled artificial fill, the fill material should be select and free-draining sand, and should be observed and evaluated by Leighton.

Maintenance Considerations: The infiltration facilities should be routinely monitored, especially before and during the rainy season, and corrective measures should be implemented as/when needed. Things to check for include proper upkeep, proper infiltration, absence of accumulated silt, and that de-silting filters/features are clean and functioning. Pretreatment desilting features should be cleaned and maintained per manufacturers' recommendations. Even with measures to prevent silt from flowing into the infiltration facility, accumulated silt may need to be removed occasionally as part of maintenance.

3.7 Pavement Design

Based on the design procedures outlined in the current Caltrans Highway Design Manual and City of Chino Standard Drawing No. 100, *Traffic Index and Notes* and using an assumed design R-value of 50, flexible pavement sections may consist of the following for the Traffic Indices indicated. Final pavement design should be based on the Traffic Index determined by the project civil engineer and R-value testing provided near the end of grading.

Asphalt Pavement Section Thickness, Type I Subgrade Soil			
Traffic Index	Asphaltic Concrete (AC) Thickness (inches)	Class 2 Aggregate Base Thickness (inches)	Total Pavement Section Thickness (inches)
Alleys	3	4	7
Residential Major and Secondary	6	8	14
Collector and Local	5	6	11

All pavement construction should be performed in accordance with the Standard Specifications for Public Works Construction or Caltrans Specifications. Field observations and periodic testing, as needed during placement of the base course materials, should be undertaken to ensure that the requirements of the standard specifications are fulfilled.

Prior to placement of aggregate base, the subgrade soil should be processed to a minimum depth of 6 inches, moisture-conditioned, as necessary, and recompacted to a minimum of 90 percent relative compaction. Aggregate base should be moisture conditioned, as necessary, and compacted to a minimum of 95 percent relative compaction.

If the pavement is to be constructed prior to construction of the structures, we recommend that the full depth of the pavement section be placed in order to support heavy construction traffic.

3.8 Temporary Excavations

All temporary excavations, including utility trenches, retaining wall excavations and other excavations should be performed in accordance with project plans, specifications and all OSHA requirements.

No surcharge loads should be permitted within a horizontal distance equal to the height of cut or 5 feet, whichever is greater from the top of the slope, unless the cut is shored appropriately. Excavations that extend below an imaginary plane inclined at 45 degrees below the edge of any adjacent existing site foundation should be properly shored to maintain support of the adjacent structures.

Cantilever shoring should be designed based on an active equivalent fluid pressure of 35 pcf. If excavations are braced at the top and at specific design intervals, the earth pressure may be approximated by a rectangular soil pressure distribution with the pressure per foot of width equal to 25H, where H is equal to the depth of the excavation being shored.

During construction, the soil conditions should be regularly evaluated to verify that conditions are as anticipated. The contractor should be responsible for providing the "competent person" required by OSHA, standards to evaluate soil conditions. Close coordination between the competent person and the geotechnical engineer should be maintained to facilitate construction while providing safe excavations.

3.9 <u>Trench Backfill</u>

Utility-type trenches onsite can be backfilled with the onsite material, provided it is free of debris, significant organic material and oversized material. Prior to backfilling the trench, pipes should be bedded and shaded in a granular material that has a sand equivalent of 30 or greater. The sand should extend 12 inches above the top of the pipe. The bedding/shading sand should be densified in-place by mechanical means, or in accordance with Greenbook specifications. The native backfill should be placed in loose layers, moisture conditioned, as necessary, and mechanically compacted using a minimum standard of 90 percent relative compaction. The thickness of layers should be based on the compaction equipment used in accordance with the Standard Specifications for Public Works Construction (Greenbook, 2015).

3.10 <u>Surface Drainage</u>

Inadequate control of runoff water and/or poorly controlled irrigation can cause the onsite soils to expand and/or shrink, producing heaving and/or settlement of foundations, flatwork, walls, and other improvements. Maintaining adequate surface drainage, proper disposal of runoff water, and control of irrigation should help reduce the potential for future soil moisture problems.

Positive surface drainage should be designed to be directed away from foundations and toward approved drainage devices, such as gutters, paved drainage swales, or watertight area drains and collector pipes.

Surface drainage should be provided to prevent ponding of water adjacent to the structures. In general, the area around the buildings should slope away from the building. We recommend that unpaved landscaped areas adjacent to the buildings be avoided. Roof runoff should be carried to suitable drainage outlets by watertight drain pipes or over paved areas.

3.11 Sulfate Attack and Corrosion Protection

Based on the results of laboratory testing, concrete structures in contact with the onsite soil will have negligible exposure to water-soluble sulfates in the soil. Type II cement may be used for concrete construction. The concrete should be designed in accordance with Table 4.3.1 of the American Concrete Institute ACI 318-14 provisions (ACI, 2014).

Based on our laboratory testing, the onsite soil is considered severely corrosive to ferrous metals. Use of non-ferrous buried pipe may be prudent, or ferrous pipe can be protected by dielectric tape, polyethylene sleeves and/or other methods, with recommendations from a corrosion engineer. Corrosion information presented in this report should be provided to your underground utility subcontractors. Additional testing and evaluation by a corrosion engineer may be warranted if corrosion protection is considered critical to the project.

3.12 Additional Geotechnical Services

The preliminary geotechnical recommendations presented in this report are based on subsurface conditions as interpreted from limited subsurface explorations and limited laboratory testing. Our preliminary geotechnical recommendations provided in this report are based on information available at the time the report was prepared and may change as plans are developed. Additional geotechnical investigation and analysis may be required based on final improvement plans. Leighton should review the site and grading plans when available and comment further on the geotechnical aspects of the project. Geotechnical observation and testing should be conducted during excavation and all phases of grading operations. Our conclusions and preliminary recommendations should be reviewed and verified by Leighton during construction and revised accordingly if geotechnical conditions encountered vary from our preliminary findings and interpretations.

Geotechnical observation and testing should be provided:

- After completion of site clearing.
- During overexcavation of compressible soil.
- During compaction of all fill materials.
- After excavation of all footings and prior to placement of concrete.
- During utility trench backfilling and compaction.
- During pavement subgrade and base preparation.
- When any unusual conditions are encountered.

4.0 LIMITATIONS

This report was based in part on data obtained from a limited number of observations, site visits, soil excavations, samples, and tests. Such information is, by necessity, incomplete. The nature of many sites is such that differing soil or geologic conditions can be present within small distances and under varying climatic conditions. Changes in subsurface conditions can and do occur over time. Therefore, our findings, conclusions, and recommendations presented in this report are based on the assumption that Leighton and Associates, Inc. will provide geotechnical observation and testing during construction.

This report was prepared for the sole use of Chino Francis Estates, LLC and Borstein Enterprises for application to the design of the proposed residential development in accordance with generally accepted geotechnical engineering practices at this time in California.

See the GBA insert on the following page for important information about this geotechnical engineering report.

Important Information about This

Geotechnical-Engineering Report

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

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, clients can benefit from a lowered exposure to the subsurface problems that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed below, contact your GBA-member geotechnical engineer. Active involvement in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Geotechnical-Engineering 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 given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled.* No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety*. Do not rely on an executive summary. Do not read selected elements only. *Read this report in full.*

You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the 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. *The geotechnical engineer who prepared this report cannot accept responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

This Report May Not Be Reliable

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be*, and, in general, *if you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying it. A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team from project start to project finish, so the individual can provide informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, *they are not final*, because the geotechnical engineer who developed them relied heavily on judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a full-time member of the design team, to:

- confer with other design-team members,
- help develop specifications,
- review pertinent elements of other design professionals' plans and specifications, and
- be on hand quickly whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction observation.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for informational purposes only*. To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include 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

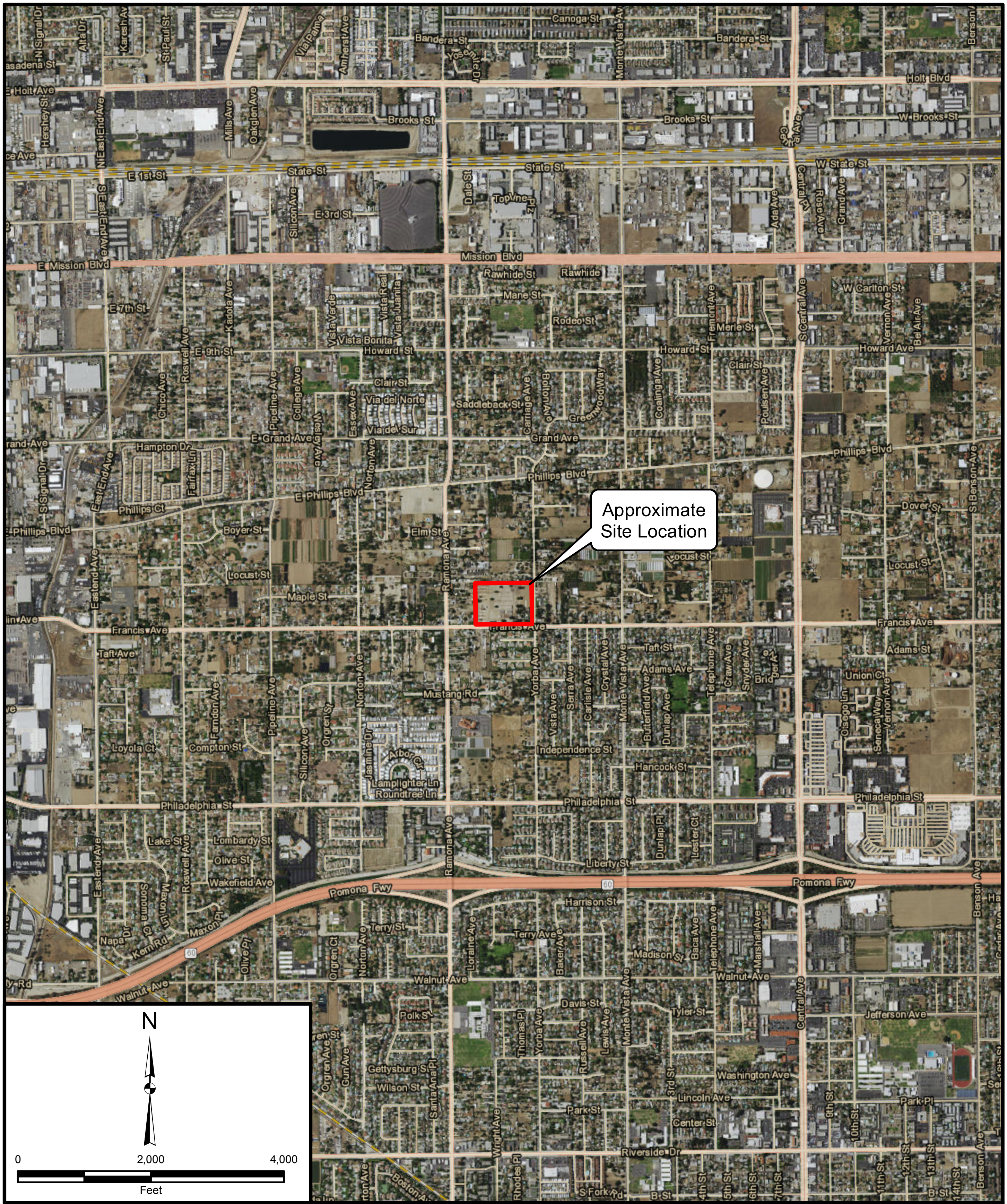
The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures*. If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old.*

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration.*** Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

GEOPROFESSIONAL BUSINESS ASSOCIATION

Telephone: 301/565-2733
e-mail: info@geoprofessional.org www.geoprofessional.org



Project: 10557.004	Eng/Geol: JDH/PB
Scale: 1 " = 2,000 '	Date: August 2016

Base Map: ESRI ArcGIS Online 2016
Thematic Information: Leighton
Author: Leighton Geomatics (mmurphy)

SITE LOCATION MAP

Proposed Residential Development, Assessor Parcel Numbers 1013-211-21 and 1013-211-22, Northwest of Francis Avenue and Yorba Linda Avenue, City of Chino, California

Figure 1

Leighton

Map Saved as V:\Drafting\10557\004\Maps\10557-004_F01_SLM_2016-08-24.mxd on 8/24/2016 1:20:00 PM

Project: 10557.004	Eng/Geol: JDH/PB
Scale: 1 " = 150 '	Date: August 2016

Base Map: ESRI ArcGIS Online 2016
Thematic Information: Leighton
Author: Leighton Geomatics (mmurphy)

TEST LOCATION MAP

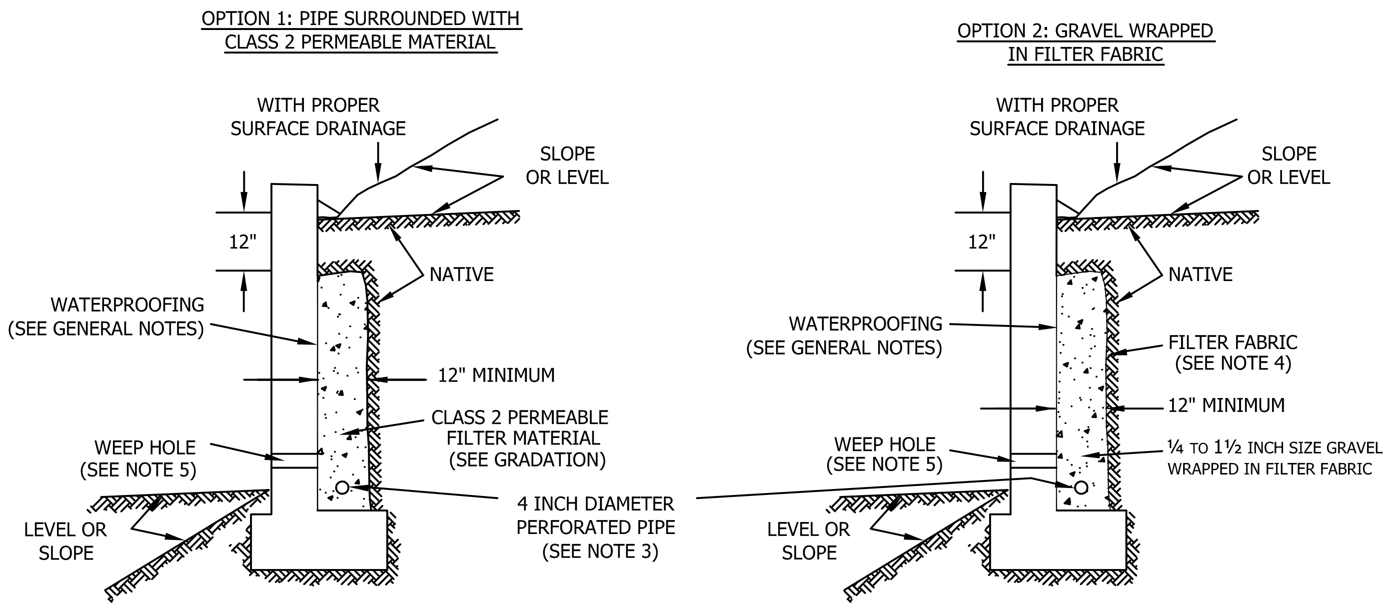
Proposed Residential Development, Assessor Parcel Numbers 1013-211-21 and 1013-211-22, Northwest of Francis Avenue and Yorba Linda Avenue, City of Chino, California

Figure 2

Leighton

Map Saved as V:\Drafting\10557\004\Maps\10557-004_F02_TLM_2016-08-24.mxd on 8/24/2016 1:17:20 PM

SUBDRAIN OPTIONS AND BACKFILL WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF ≤50



Class 2 Filter Permeable Material Gradation
Per Caltrans Specifications

Sieve Size	Percent Passing
1"	100
3/4"	90-100
3/8"	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

GENERAL NOTES:

* Waterproofing should be provided where moisture nuisance problem through the wall is undesirable.
* Water proofing of the walls is not under purview of the geotechnical engineer
* All drains should have a gradient of 1 percent minimum

*Outlet portion of the subdrain should have a 4-inch diameter solid pipe discharged into a suitable disposal area designed by the project engineer. The subdrain pipe should be accessible for maintenance (rodding)

*Other subdrain backfill options are subject to the review by the geotechnical engineer and modification of design parameters.

Notes:

1) Sand should have a sand equivalent of 30 or greater and may be densified by water jetting.
2) 1 Cu. ft. per ft. of 1/4- to 1 1/2-inch size gravel wrapped in filter fabric
3) Pipe type should be ASTM D1527 Acrylonitrile Butadiene Styrene (ABS) SDR35 or ASTM D1785 Polyvinyl Chloride plastic (PVC), Schedule 40, Armco A2000 PVC, or approved equivalent. Pipe should be installed with perforations down. Perforations should be 3/8 inch in diameter placed at the ends of a 120-degree arc in two rows at 3-inch on center (staggered)
4) Filter fabric should be Mirafi 140NC or approved equivalent.
5) Weephole should be 3-inch minimum diameter and provided at 10-foot maximum intervals. If exposure is permitted, weepholes should be located 12 inches above finished grade. If exposure is not permitted such as for a wall adjacent to a sidewalk/curb, a pipe under the sidewalk to be discharged through the curb face or equivalent should be provided. For a basement-type wall, a proper subdrain outlet system should be provided.
6) Retaining wall plans should be reviewed and approved by the geotechnical engineer.
7) Walls over six feet in height are subject to a special review by the geotechnical engineer and modifications to the above requirements.

RETAINING WALL BACKFILL AND SUBDRAIN DETAIL FOR WALLS 6 FEET OR LESS IN HEIGHT

WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF ≤50

Leighton

Figure 3

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APPENDIX A

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APPENDIX A

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APPENDIX B

GEOTECHNICAL BORING LOGS
AND INFILTRATION TEST RESULTS
(Leighton, 2016)

Leighton

GEOTECHNICAL BORING LOG LB-1

Project No.	10557.004	**Date Drilled**	12-13-13
Project	Coastal Commerce Chino	**Logged By**	JMD
Drilling Co.	2R Drilling	**Hole Diameter**	9.5"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	849'
Location	See Figure 2	**Sampled By**	JMD

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0			BULK					@Surface: dirt with some straw	
845				R-1	4 6 11	111	2	SM	@2.5' SILTY SAND, loose, light olive brown, dry to moist, fine sand, 30% fines (field estimate), trace rootlets, trace fine gravel	
	5			R-2	7 10 14	119	1	SP	@5' SAND, medium dense, light brown, dry, medium to coarse sand, trace fines, trace fine gravel, larger piece of gravel in ring sample	
840	10			R-3	10 15 21	121	2	SP	@10' SAND, medium dense, gray to brown, dry, medium sand, some gravel, 1.25" maximum gravel size	
835	15			R-4	7 12 17	108	10	ML	@15' SANDY SILT, very stiff, yellowish brown, dry to moist, homogenous	-200
830	20			S-5	6 8 10			ML SP	@20' SANDY SILT, very stiff, dark gray, dry to moist, fine sand @20.7' SAND, gray, dry to moist, fine to medium sand	
825	25								Total depth of 21.5' No groundwater encountered Backfilled with soil cuttings	
820	30									

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

*** * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * ***

GEOTECHNICAL BORING LOG LB-2

Project No.	10557.004	**Date Drilled**	12-13-13
Project	Coastal Commerce Chino	**Logged By**	JMD
Drilling Co.	2R Drilling	**Hole Diameter**	9.5"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	844'
Location	See Figure 2	**Sampled By**	JMD

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	0			BULK					@Surface: dirt with some grass	
				R-1	3 5 7	106	2	SM	@2.5' SILTY SAND, loose, light gray brown, dry, fine sand, 30% fines (field estimate), trace fine gravel	
840	5			R-2	7 9 10			SP	@5' SAND, medium dense, reddish brown, dry, medium to coarse sand, trace fines, some gravel, 1.25" maximum gravel size	
835	10			R-3	20 24 25	126	2	SP	@10' SAND, medium dense, light gray brown, dry, medium to coarse sand, angular, broken rocks up to 2.25" in sample	
830	15			S-4	7 8 9			SP	@15' SAND, medium dense, gray, dry to moist, medium sand	
825	20			R-5	17 23 45	111	15	ML	@20' SANDY SILT, very dense, olive, moist, some FeO2 staining	
820	25			S-6	7 12 11			ML-CL	@25' SILT to CLAY, very stiff, gray, dry to moist, with FeO2 staining @25.4' SAND, dry, fine to medium sand @25.6' SILT, gray, moist @25.9' CLAY, gray, moist	
815	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

*** This log is a part of a report by Leighton and should not be used as a stand-alone document. ***

GEOTECHNICAL BORING LOG LB-2

Project No.	10557.004	**Date Drilled**	12-13-13
Project	Coastal Commerce Chino	**Logged By**	JMD
Drilling Co.	2R Drilling	**Hole Diameter**	9.5"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	844'
Location	See Figure 2	**Sampled By**	JMD

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	30			S-7	8 16 21			ML	@30' SILT, hard, olive brown, dry to moist, FeO2 staining, with some clay @30.5' SILT, olive brown, dry to moist, FeO2 staining @31' SAND, dark reddish brown to light gray, dry, fine to medium sand	
810										
	35			S-8	18 24 21			SP	@35' SAND, light brown, dry to moist, with large amounts of FeO2 staining, trace fine gravel, a 1.25" piece of gravel in the sampler tip	
805										
	40			S-9	12 10 20			CL	@40' CLAY with gravel, hard, reddish brown to olive brown, gravel up to 2" large, with some silt, some FeO2 staining @41.3' SAND with gravel, dry to moist, medium to coarse sand, gravel up to 2" large	
800										
	45			S-10	15 35 24			SM	@45' SILTY SAND, very dense, reddish brown, moist, angular, 20% fines (field estimate), with some gravel, 1" maximum gravel size	
795										
	50			S-11	9 11 16			ML	@50' SILT, very stiff, olive brown, moist, with FeO2 staining, homogenous	
									Total depth of 51.5' No groundwater encountered Bakfilled with soil cuttings	
790										
	55									
785										
	60									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

GEOTECHNICAL BORING LOG LB-3

Project No.	10557.004	**Date Drilled**	12-13-13
Project	Coastal Commerce Chino	**Logged By**	JMD
Drilling Co.	2R Drilling	**Hole Diameter**	9.5"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	852'
Location	See Figure 2	**Sampled By**	JMD

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
	0		BULK					@Surface: dry grass	
850			R-1	2 4 8	104	4	SM	@2.5' SILTY SAND, loose, light brown, dry, fine sand, 40% fines (field estimate), trace rootlets	
	5		R-2	8 11 14	111	5	SM	@5' SILTY SAND, medium dense, brown, moist, fine sand, 30% fines (field estimate)	
845									
	10		R-3	11 7 13	111	4	SM	@10' SILTY SAND, medium dense, light gray brown, moist, fine sand, 30% fines (field estimate), trace fine gravel	CO
840									
	15		R-4	11 17 19	93	9	ML	@15' SILT, very stiff, gray, moist, FeO2 staining, homongenous	AL
835									
	20		S-5	5 7 9			CL ML	@20' CLAY, very stiff, gray, moist, FeO2 staining @20.5' SILT, gray, moist, FeO2 staining	
830									
	25		S-6	5 5 11			ML		
825									
	30								

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

*** * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * ***

GEOTECHNICAL BORING LOG LB-3

Project No.	10557.004	**Date Drilled**	12-13-13
Project	Coastal Commerce Chino	**Logged By**	JMD
Drilling Co.	2R Drilling	**Hole Diameter**	9.5"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	852'
Location	See Figure 2	**Sampled By**	JMD

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
	30			S-7	12 14 16			ML	@30' SANDY SILT, very stiff, gray, moist	
820								SM	@31.1' SILTY SAND, gray, dry, fine sand, 20% fines (field estimate),	
	35			S-8	17 13 8			SP	@35' SAND, medium dense, reddish brown, medium to coarse sand	
815								CL	@36.3' CLAY, olive brown, moist, large amount of FeO2 staining	
	40			S-9	9 14 26			ML	@40' SANDY SILT, hard, olive brown, moist, large amount of FeO2 staining	
810										
	45			S-10	6 8 9			ML	@45' SILT, very stiff, light brown, large amount of FeO2 staining, homogenous	
805										
	50			S-11	14 14 20			SP	@50' SAND, dense, light gray brown, dry to moist, fine sand, trace fines	
								ML	@51.2' SILT, light brown, large amount of FeO2 staining	
800									Total depth of 51.5' No groundwater encountered Backfilled with soil cuttings	
	55									
795										
	60									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

GEOTECHNICAL BORING LOG LB-4

Project No.	10557.004	**Date Drilled**	12-13-13
Project	Coastal Commerce Chino	**Logged By**	JMD
Drilling Co.	2R Drilling	**Hole Diameter**	9.5"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	850'
Location	See Figure 2	**Sampled By**	JMD

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N / S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
850	0			BULK					@Surface: dirt	MD, CR
				R-1	5 8 8			SM	@2.5' SILTY SAND, loose, light brown to brown, dry, 20% fines (field estimate), with some gravel, 2" maximum gravel size	
845	5			R-2	6 11 10	121	2	SP-SM	@5' SAND to SILTY SAND, medium dense, light brown, dry to moist, 10% fines (field estimate), with some gravel, 1.25" maximum gravel size	-200
840	10			R-3	20 30 40			SP	@10' SAND, dense, gray, dry to moist, trace fines, some fine and medium sand, some gravel, 1.5" maximum gravel size	
835	15			S-4	6 7 7			SM	@15' SILTY SAND, medium dense, olive, moist, fine sand, 40% fines (field estimate), trace coarse sand	
830	20			R-5	10 19 26	100	19	CL	@20' CLAY, very stiff, olive, moist, with FeO2 staining. homogenous	
									Total depth of 21.5' No groundwater encountered Backfilled with soil cuttings	
825	25									
820	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

GEOTECHNICAL BORING LOG LB-5

Project No.	10557.004	**Date Drilled**	12-13-13
Project	Coastal Commerce Chino	**Logged By**	JMD
Drilling Co.	2R Drilling	**Hole Diameter**	10"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	848'
Location	See Figure 2	**Sampled By**	JMD

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	0			BULK					@Surface: dirt	
845										
	5			R-2	12 28 34	117	3	SP	@5' SAND, dense, gray brown, moist, medium sand, with some gravel, 1" maximum gravel size	
840										
	10			R-3	12 16 18	106	3	SP	@10' SAND, medium dense, gray to reddish brown, moist, medium sand, trace gravel, 2" maximum gravel size	
835										
	15			R-4	17 14 17	105	2	SP	@15' SAND, medium dense, olive, moist, trace fines, trace fine gravel, trace FeO2 staining	
830										
	20			S-5	7 6 6			SM	@20' SILTY SAND, medium dense, olive, dry to moist, fine sand, 40% fines (field estimate), some FeO2 staining	
									Total depth of 21.5' No groundwater encountered Backfilled with soil cuttings	
825										
	25									
820										
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

General Test Setup Data of Well Permeameter, from USBR 7300-89 Method.

Project: Coastal Commercial Chino, Project No. 10557.004

	Item	Units	LB-1		LB-2		LB-3		LB-4		LB-5			
	Exploration #/Location:		*LB-1*		*LB-2*		*LB-3*		*LB-4*		*LB-5*			
	Approx. Test Depth (ft):		6		4		6		6		5			
	Date Tested, start/finish:		12/16/2013		12/16/2013		12/16/2013		12/16/2013		12/16/2013			
	Tested by:		JMD		JMD		JMD		JMD		JMD			
	USCS Soil Type:													
	Weather (start to finish)		Warm, clear											
	Liquid Used/pH:		water from garden hose											
	Well Prep:		straight drill, tamp											
a.	**Diameter of barrel (in.):**		22.5		22.5		22.5		22.5		22.5		22.5	
b.	**No. of Supply barrels:**		1		1		1		1		1		1	
c.	**Measured boring diameter**	inches	9.5		9.5		9.5		9.5		10		13	
d.	Approx Depth to groundwater below GS	feet	200		200		200		200		200		200	
	Depths from string line (or top of ex. pavement):													
f.	to ground surface (=0 if no string line used)	ft/in.	0. ft		0. ft		0. ft		0. ft		0. ft			
g.	**to Bot of Boring** (or top of soil over Bentonite)	ft/in.	6. ft	1. in.	4. ft	3. in.	5. ft	7. in.	5. ft	8. in.	4. ft	10. in.		
i	to Top of Sand (bot of float assbly) (dry)	ft/in.	5. ft	10. in.	4. ft	2. in.	5. ft	4.5 in.	5. ft	5. in.	4. ft	6. in.		
k	to Top of casing after adding water (negative is above string line)	ft/in.	0. ft	-3. in.			0. ft	-0.75 in.			0. ft	-1. in.		
m	Top of Float assembly Rod, when pushed to bottom	ft/in.		34.75 in.				33.5 in.				14.88 in.		
n	**top of float assembly rod, floating, water level stable**	ft/in.		30.5 in.				25.13 in.				26.5 in.		
p	**Float Assembly (choose one)**		Long body				Long body				Long body			
q	Float Assembly extension (0=none)	inches	12				12				0			
s	free play in float assembly (water level stablized)	inches	2.5				1.25				2.5			
t	Length of float assembly (=lookup p)	inches	23		#N/A		23		#N/A		23		#N/A	
u	Length of float assembly plus extension (=q+t)	inches	35		#N/A		35		#N/A		23		#N/A	
v	**Ht from water surface to top of float rod (=lookup p)**	inches	16.75		#N/A		16.75		#N/A		16.75		#N/A	
w	range of float movement (=lookup p)	inches	6.75		#N/A		6.75		#N/A		6.75		#N/A	
x	Depth to Water Surface (=n+v)		47.3 in.		#N/A in.		41.9 in.		#N/A in.		43.3 in.		#N/A in.	
h	**Depth of water in Well, "h" (=g-x)**		**25.8 in=** 2.15 ft		**#N/A** #N/A		**25.1 in=** 2.09 ft		**#N/A** #N/A		**14.8 in=** 1.23 ft		**#N/A** #N/A	
y	Total Area of barrels (in.^2):		397		397		397		397		397		397	
r	**Well Radius, "r" (=c/2)**		**4.8** in.		**4.8** in.		**4.8** in.		**4.8** in.		**5.0** in.		**6.5** in.	

Results of Well Permeameter Test, from USBR 7300-89 Method.

Project: **Coastal Commercial Chino, Project No. 10557.004**

Exploration #/Location: ***LB-1*** Initial Depth to top of float rod (in.) 30.5

Leighton

Field Data **Calculations**

Date (and comments)	Time	**Water Level in Supply Barrel (in.)**	Depth to top of float rod (when changed) ft	in.	Water Temp in Barrel (deg F)	DL Interpre-tation? ("Y")	DL -- Head of Water in Barrel (in.)	h, Height of Water in Well (in.)	h/r	Total Elapsed Time (minutes)	Δt (min)	Vol Change (in.^3)	Flow (in^3/min)	q, Flow (in^3/hr)	V (Fig 9)	K20, Coef. Of Permeability at 20 deg C (in./hr)	**Infiltration Rate [flow/surf area] (in./hr) (FS=1)**
Start Date 12/16/2013	Start time: 12:52:00 PM																
12/16/13	12:52	29.25			**74**			25.75	5.4	0					0.9		
12/16/13	12:53	28						25.75	5.4	0	1	497	497	29805	0.9	10.01	**14.65**
12/16/13	12:54	27						25.75	5.4	0	1	397	397	23844	0.9	8.00	**11.72**
12/16/13	12:55	26.625						25.75	5.4	0	1	149	149	8942	0.9	3.00	**4.39**
12/16/13	12:57	25.875						25.75	5.4	0	2	298	149	8942	0.9	3.00	**4.39**
12/16/13	13:05	20.25						25.75	5.4	0	8	2235	279	16766	0.9	5.63	**8.24**
12/16/13	13:23	10.75			76			25.75	5.4	0	18	3775	210	12585	0.9	4.11	**6.02**
										0							
12/16/13	13:27	31.125			76			25.75	5.4	0					0.9		
12/16/13	13:49	20.25						25.75	5.4	0	22	4322	196	11787	0.9	3.85	**5.64**
12/16/13	14:01	14.25			77			25.75	5.4	0	12	2384	199	11922	0.9	3.85	**5.63**
										0							
12/16/13	14:06	31.375			77			25.75	5.4	0					0.9		
12/16/13	14:37	18.5			77			25.75	5.4	0	31	5117	165	9903	0.9	3.20	**4.68**
12/16/13	15:07	7.25			77			25.75	5.4	0	30	4471	149	8942	0.9	2.89	**4.22**
12/16/13	15:20	3						25.75	5.4	0	13	1689	130	7795	0.9	2.52	**3.68**
										0							

Results of Well Permeameter Test, from USBR 7300-89 Method.

Project: **Coastal Commercial Chino, Project No. 10557.004**

Exploration #/Location: ***LB-3***

Initial Depth to top of float rod (in.) 25.125

Leighton

Field Data | **Calculations**

Date (and comments)	Time	**Water Level in Supply Barrel (in.)**	Depth to top of float rod (when changed) ft	Depth to top of float rod (when changed) in.	Water Temp in Barrel (deg F)	DL Interpre-tation? ("Y")	DL -- Head of Water in Barrel (in.)	h, Height of Water in Well (in.)	h/r	Total Elapsed Time (minutes)	Δt (min)	Vol Change (in.^3)	Flow (in^3/min)	q, Flow (in^3/hr)	Cumulative Vol (gal)	V (Fig 9)	K20, Coef. Of Permeability at 20 deg C (in./hr)	**Infiltration Rate [flow/surf area] (in./hr) (FS=1)**
Start Date: 12/16/2013	Start time: 10:25:00 AM																	
12/16/13	10:25	30.25			**69**			25.125	5.3	0					0	1.0		
12/16/13	11:04	28.375			74			25.125	5.3		39	745	19	1146		0.9	0.40	**0.58**
12/16/13	11:35	27.375			77			25.125	5.3		31	397	13	769		0.9	0.26	**0.37**
12/16/13	12:27	25.75			79			25.125	5.3		52	646	12	745		0.8	0.24	**0.35**
12/16/13	13:09	24.5			81			25.125	5.3		42	497	12	710		0.8	0.23	**0.33**
12/16/13	13:53	23.25			81			25.125	5.3		44	497	11	677		0.8	0.22	**0.31**
12/16/13	14:49	20.75			82			25.125	5.3		56	994	18	1064		0.8	0.34	**0.49**
12/16/13	15:45	19.125			83			25.125	5.3		56	646	12	692		0.8	0.22	**0.31**

Results of Well Permeameter Test, from USBR 7300-89 Method.

Project: **Coastal Commercial Chino, Project No. 10557.004**

Exploration #/Location: ***LB-5***

Initial Depth to top of float rod (in.) 26.5

Leighton

Field Data | **Calculations**

Date (and comments)	Time	**Water Level in Supply Barrel (in.)**	Depth to top of float rod (when changed) ft	in.	Water Temp in Barrel (deg F)	DL Interpre-tation? ("Y")	DL -- Head of Water in Barrel (in.)	h, Height of Water in Well (in.)	h/r	Total Elapsed Time (minutes)	Δt (min)	Vol Change (in.^3)	Flow (in^3/min)	q, Flow (in^3/hr)	Cumulative Vol (gal)	V (Fig 9)	K20, Coef. Of Permeability at 20 deg C (in./hr)	**Infiltration Rate [flow/surf area] (in./hr) (FS=1)**
Start Date	Start time:																	
12/16/2013	2:25:00 PM																	
12/16/13	14:25	31			**77**			14.75	3.0	0					0	0.9		
12/16/13	14:26	30						14.75	3.0		1	397	397	23844		0.9	16.33	**17.75**
12/16/13	14:27	29.125						14.75	3.0		1	348	348	20864		0.9	14.29	**15.53**
12/16/13	14:28	28.125						14.75	3.0		1	397	397	23844		0.9	16.33	**17.75**
12/16/13	14:29	27.25						14.75	3.0		1	348	348	20864		0.9	14.29	**15.53**
12/16/13	14:30	26.25						14.75	3.0		1	397	397	23844		0.9	16.33	**17.75**
12/16/13	14:32	24.375						14.75	3.0		2	745	373	22354		0.9	15.31	**16.64**
12/16/13	14:42	15.375			77			14.75	3.0		10	3577	358	21460		0.9	14.70	**15.97**
12/16/13	14:53	6						14.75	3.0		11	3726	339	20322		0.9	13.92	**15.13**
12/16/13	15:01	25.125			79			14.75	3.0							0.8		
12/16/13	15:02	24.5						14.75	3.0		1	248	248	14903		0.8	9.96	**10.82**
12/16/13	15:24	7.75			79			14.75	3.0	0	22	6657	303	18154	0	0.8	12.13	**13.18**
12/16/13	15:31	2.375						14.75	3.0	0	7	2136	305	18309	0	0.8	12.23	**13.30**
										0					0			

APPENDIX C

LABORATORY TEST RESULTS
(Leighton, 2016)

Leighton

TESTS for SULFATE CONTENT
CHLORIDE CONTENT and pH of SOILS

Leighton

Project Name:	Coastal Commercial Chino	Tested By :	G. Berdy	Date:	12/26/13
Project No. :	10557.004	Data Input By:	J. Ward	Date:	01/03/14

Boring No.	LB-4			
Sample No.	B-4			
Sample Depth (ft)	0-5			
Soil Identification:	Olive brown (SP-SM)g			
Wet Weight of Soil + Container (g)	301.40			
Dry Weight of Soil + Container (g)	299.00			
Weight of Container (g)	64.80			
Moisture Content (%)	1.02			
Weight of Soaked Soil (g)	100.50			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	31			
Crucible No.	28			
Furnace Temperature (°C)	820			
Time In / Time Out	8:50/9:35			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	21.1490			
Wt. of Crucible (g)	21.1467			
Wt. of Residue (g) (A)	0.0023			
PPM of Sulfate (A) x 41150	94.65			
PPM of Sulfate, Dry Weight Basis	**96**			

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	15			
ml of AgNO3 Soln. Used in Titration (C)	1.2			
PPM of Chloride (C -0.2) * 100 * 30 / B	200			
PPM of Chloride, Dry Wt. Basis	**202**			

pH TEST, DOT California Test 532/643

pH Value	**6.94**			
Temperature °C	21.0			

SOIL RESISTIVITY TEST

DOT CA TEST 532 / 643

Leighton

Project Name:	Coastal Commercial Chino	Tested By :	G. Berdy	Date:	12/31/13
Project No. :	10557.004	Data Input By:	J. Ward	Date:	01/03/14
Boring No.:	LB-4	Depth (ft.) :	0-5		
Sample No. :	B-4				

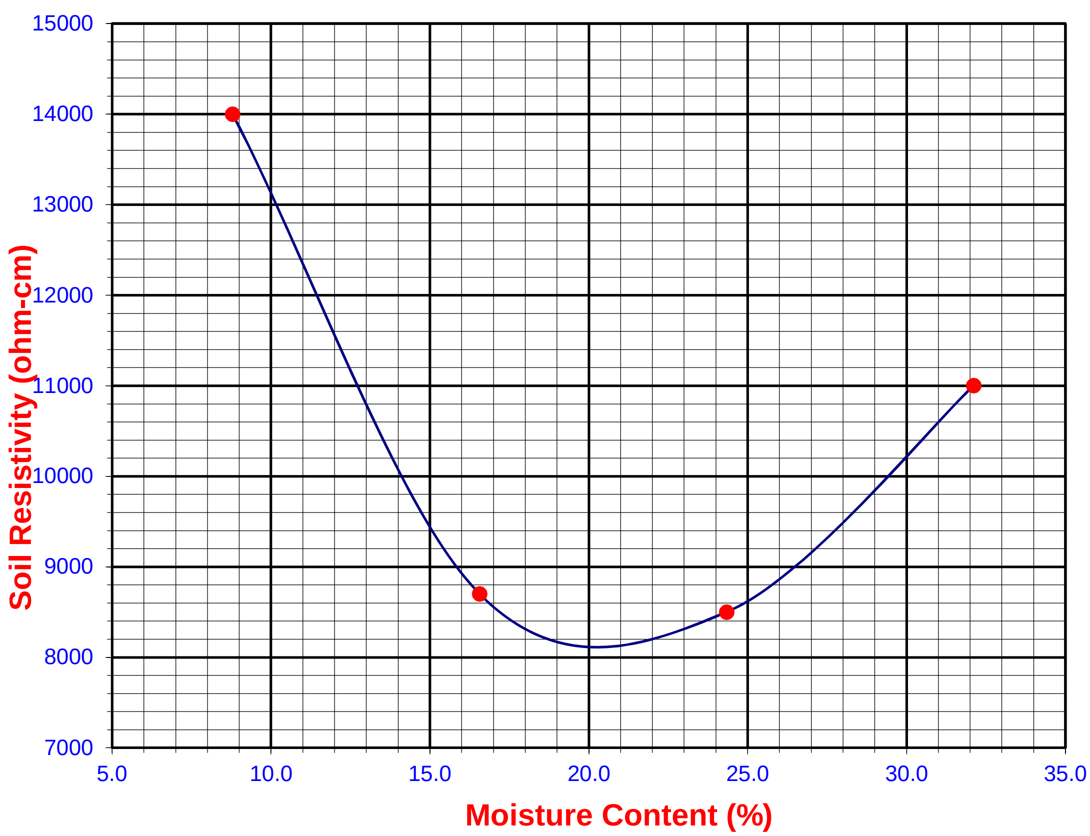
Soil Identification:* Olive brown (SP-SM)g

*California Test 643 requires soil specimens to consist only of portions of samples passing through the No. 8 US Standard Sieve before resistivity testing. Therefore, this test method may not be representative for coarser materials.

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	10	8.80	14000	14000
2	20	16.57	8700	8700
3	30	24.34	8500	8500
4	40	32.11	11000	11000
5				

Moisture Content (%) (MCi)	1.02
Wet Wt. of Soil + Cont. (g)	301.40
Dry Wt. of Soil + Cont. (g)	299.00
Wt. of Container (g)	64.80
Container No.	
Initial Soil Wt. (g) (Wt)	130.00
Box Constant	1.000
MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 532 / 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 532 / 643	
8100	**20.3**	**96**	**202**	**6.94**	**21.0**



Leighton

TESTS for SULFATE CONTENT
CHLORIDE CONTENT and pH of SOILS

Project Name:	Coastal Commercial Chino	Tested By :	G. Berdy	Date:	12/26/13
Project No. :	10557.004	Data Input By:	J. Ward	Date:	01/03/14

Boring No.	LB-4			
Sample No.	B-4			
Sample Depth (ft)	0-5			
Soil Identification:	Olive brown (SP-SM)g			
Wet Weight of Soil + Container (g)	301.40			
Dry Weight of Soil + Container (g)	299.00			
Weight of Container (g)	64.80			
Moisture Content (%)	1.02			
Weight of Soaked Soil (g)	100.50			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	31			
Crucible No.	28			
Furnace Temperature (°C)	820			
Time In / Time Out	8:50/9:35			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	21.1490			
Wt. of Crucible (g)	21.1467			
Wt. of Residue (g) (A)	0.0023			
PPM of Sulfate (A) x 41150	94.65			
PPM of Sulfate, Dry Weight Basis	**96**			

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	15			
ml of AgNO3 Soln. Used in Titration (C)	1.2			
PPM of Chloride (C -0.2) * 100 * 30 / B	200			
PPM of Chloride, Dry Wt. Basis	**202**			

pH TEST, DOT California Test 532/643

pH Value	**6.94**			
Temperature °C	21.0			

SOIL RESISTIVITY TEST

DOT CA TEST 532 / 643

Project Name:	Coastal Commercial Chino	Tested By :	G. Berdy	Date:	12/31/13
Project No. :	10557.004	Data Input By:	J. Ward	Date:	01/03/14
Boring No.:	LB-4	Depth (ft.) :	0-5		
Sample No. :	B-4				

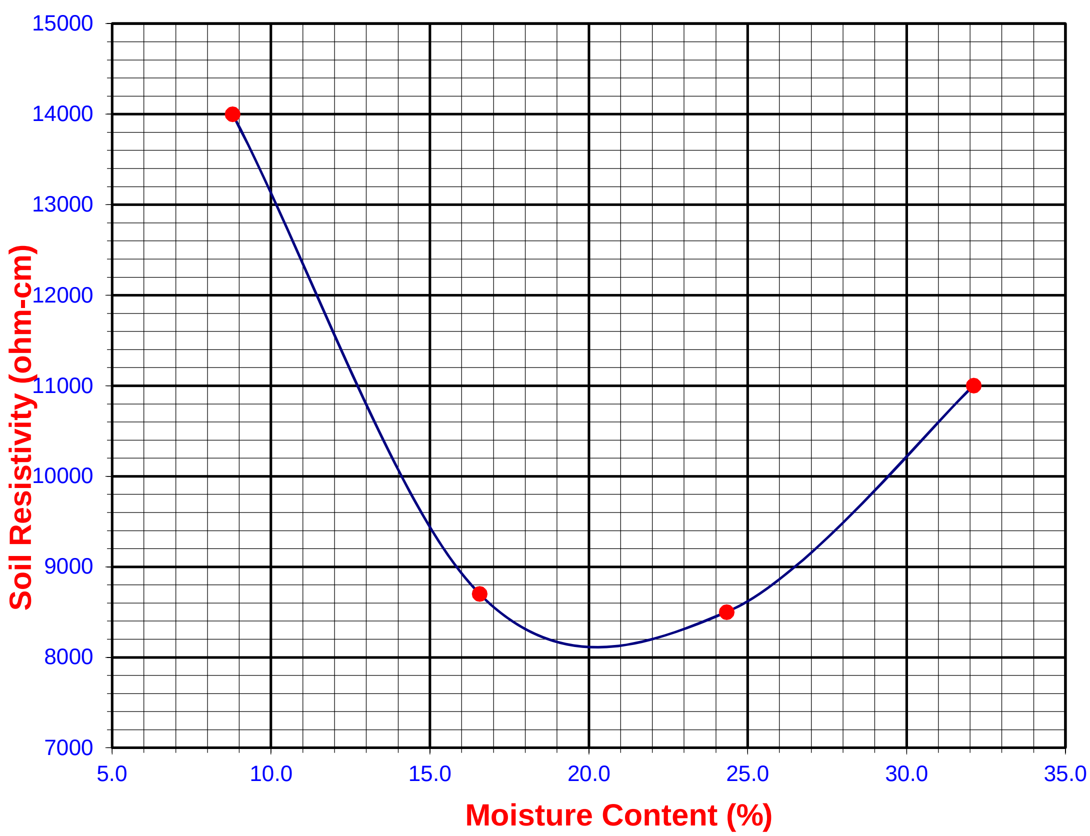
Soil Identification:* Olive brown (SP-SM)g

*California Test 643 requires soil specimens to consist only of portions of samples passing through the No. 8 US Standard Sieve before resistivity testing. Therefore, this test method may not be representative for coarser materials.

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	10	8.80	14000	14000
2	20	16.57	8700	8700
3	30	24.34	8500	8500
4	40	32.11	11000	11000
5				

Moisture Content (%) (MCi)	1.02
Wet Wt. of Soil + Cont. (g)	301.40
Dry Wt. of Soil + Cont. (g)	299.00
Wt. of Container (g)	64.80
Container No.	
Initial Soil Wt. (g) (Wt)	130.00
Box Constant	1.000
MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 532 / 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 532 / 643	
8100	**20.3**	**96**	**202**	**6.94**	**21.0**



MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Leighton

Project Name:	Coastal Commercial Chino	Tested By:	O. Figueroa	Date:	12/27/13
Project No.:	10557.004	Input By:	J. Ward	Date:	01/03/14
Boring No.:	LB-4	Depth (ft.):	0-5		
Sample No.:	B-4				
Soil Identification:	Olive brown poorly-graded sand with silt and gravel (SP-SM)g				

Preparation Method:	X	Moist
		Dry
Compaction Method	X	Mechanical Ram
		Manual Ram

Scalp Fraction (%)	
#3/4	
#3/8	
#4	15.4

Rammer Weight (lb.) = 10.0

Height of Drop (in.) = 18.0

Mold Volume (ft³) 0.03310

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3773.0	3834.0	3898.0	3899.0		
Weight of Mold (g)	1859.0	1859.0	1859.0	1859.0		
Net Weight of Soil (g)	1914.0	1975.0	2039.0	2040.0		
Wet Weight of Soil + Cont. (g)	475.80	450.50	423.80	506.90		
Dry Weight of Soil + Cont. (g)	462.20	428.90	395.80	463.60		
Weight of Container (g)	48.50	51.30	54.80	52.70		
Moisture Content (%)	3.29	5.72	8.21	10.54		
Wet Density (pcf)	127.5	131.5	135.8	135.9		
Dry Density (pcf)	123.4	124.4	125.5	122.9		

Maximum Dry Density (pcf)	**125.5**	**Optimum Moisture Content (%)**	**8.0**
Corrected Dry Density (pcf)	**130.5**	**Corrected Moisture Content (%)**	**7.0**

X **Procedure A**

Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
May be used if +#4 is 20% or less

☐ **Procedure B**

Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
Use if +#4 is >20% and +3/8 in. is 20% or less

☐ **Procedure C**

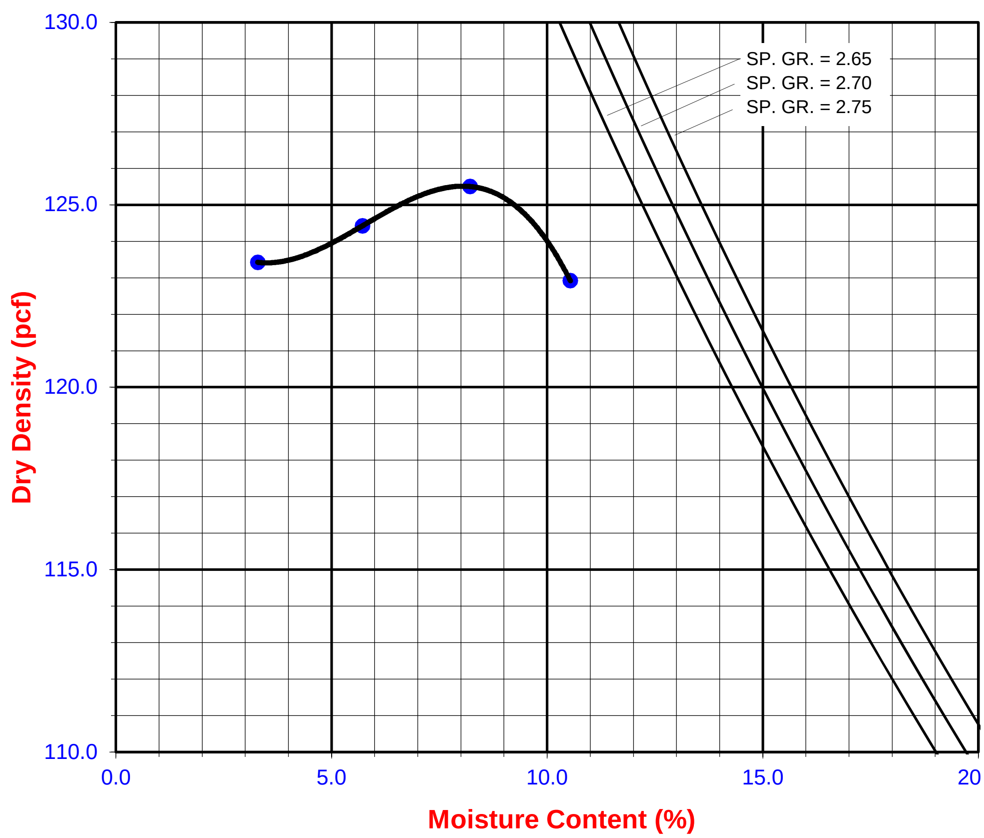
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL,PL,PI



ATTERBERG LIMITS

ASTM D 4318

Leighton

Project Name:	Coastal Commercial Chino	Tested By:	G. Bathala	Date:	12/30/13
Project No. :	10557.004	Input By:	J. Ward	Date:	01/03/13
Boring No.:	LB-3	Checked By:	J. Ward		
Sample No.:	R-4	Depth (ft.)	15.0		
Soil Identification:	Olive sandy silt s(ML)				

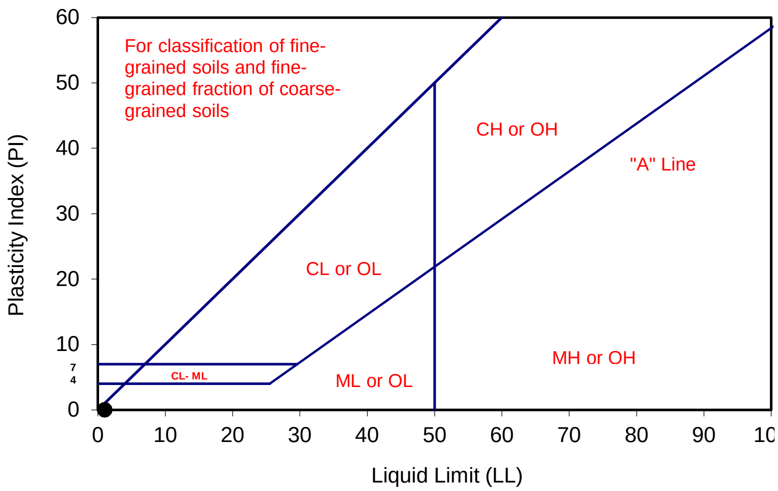
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			5			
Wet Wt. of Soil + Cont. (g)	18.01	16.94	35.69	**Cannot get more than 5 blows:**		
Dry Wt. of Soil + Cont. (g)	17.10	16.27	30.12	**NonPlastic**		
Wt. of Container (g)	13.51	13.61	13.51			
Moisture Content (%) [Wn]	25.35	25.19	33.53			

Liquid Limit	**NP**
Plastic Limit	**25**
Plasticity Index	**NP**
Classification	**NP**

PI at "A" - Line = 0.73(LL-20) =

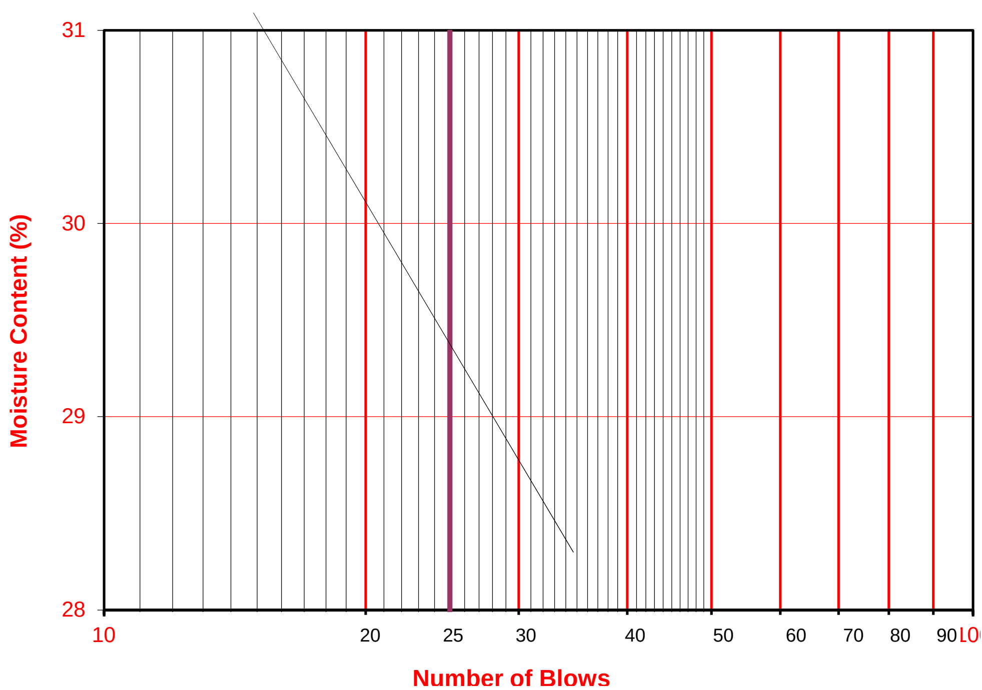
One - Point Liquid Limit Calculation

$LL = Wn(N/25)^{0.121}$



PROCEDURES USED

- [ ] Wet Preparation
  Multipoint - Wet
- [X] Dry Preparation
  Multipoint - Dry
- [X] Procedure A
  Multipoint Test
- [ ] Procedure B
  One-point Test



ONE-DIMENSIONAL SWELL OR SETTLEMENT POTENTIAL OF COHESIVE SOILS
(ASTM D 4546)

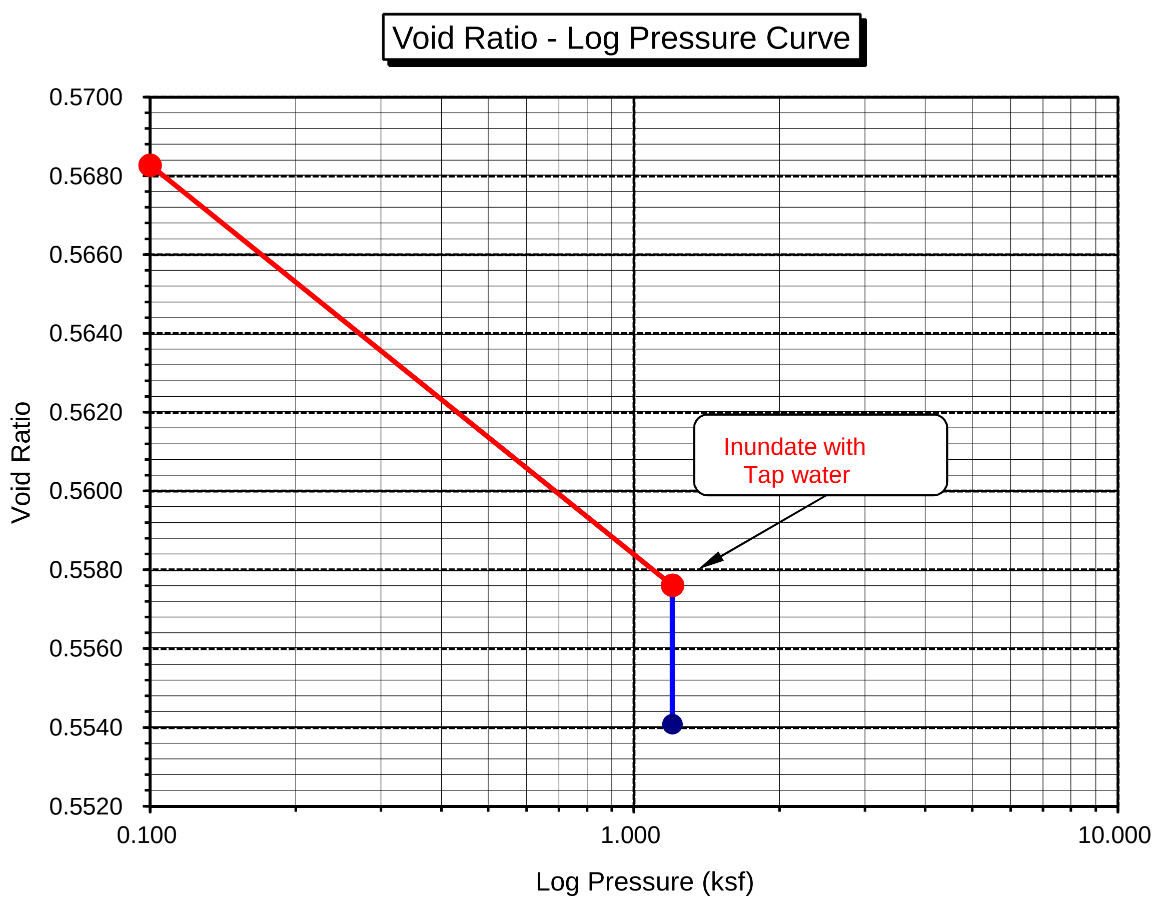
Project Name:	Coastal Commercial Chino	Tested By:	G. Bathala	Date:	12/20/13
Project No.:	10557.004	Checked By:	J. Ward	Date:	01/03/14
Boring No.:	LB-3	Sample Type:	Ring		
Sample No.:	R-3	Depth (ft.)	10.0		
Sample Description:	Olive silty sand (SM)				

Initial Dry Density (pcf):	107.5
Initial Moisture (%):	3.61
Initial Length (in.):	1.0000
Initial Dial Reading:	0.3063
Diameter(in):	2.416

Final Dry Density (pcf):	108.5
Final Moisture (%) :	17.2
Initial Void Ratio:	0.5683
Specific Gravity(assumed):	2.70
Initial Saturation (%)	17.2

Pressure (p) (ksf)	Final Reading (in)	Apparent Thickness (in)	Load Compliance (%)	Swell (+) Settlement (-) % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.100	0.3063	1.0000	0.00	0.00	0.5683	0.00
1.200	0.2983	0.9920	0.12	-0.80	0.5576	-0.68
H2O	0.2961	0.9898	0.12	-1.03	0.5541	-0.91

Percent Swell (+) / Settlement (-) After Inundation = -0.23



Boring No.	LB-1	LB-4						
Sample No.	R-4	R-2						
Depth (ft.)	15.0	5.0						
Sample Type	Ring	Ring						
Soil Identification	Brown poorly-graded sand with silt and gravel (SP-SM)g	Olive brown sandy silt s(ML)						
Moisture Correction								
Wet Weight of Soil + Container (g)	0.0	0.0						
Dry Weight of Soil + Container (g)	0.0	0.0						
Weight of Container (g)	1.0	1.0						
Moisture Content (%)	0.00	0.00						
Sample Dry Weight Determination								
Weight of Sample + Container (g)	822.7	915.4						
Weight of Container (g)	250.0	252.4						
Weight of Dry Sample (g)	572.7	663.0						
Container No.:								
After Wash								
Method (A or B)	B	B						
Dry Weight of Sample + Cont. (g)	782.9	519.1						
Weight of Container (g)	250.0	252.4						
Dry Weight of Sample (g)	532.9	266.7						
% Passing No. 200 Sieve	**6.9**	**59.8**						
% Retained No. 200 Sieve	93.1	40.2						

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PERCENT PASSING
No. 200 SIEVE
ASTM D 1140

Project Name: Coastal Commercial Chino
Project No.: 10557.004
Client Name: L&A/Rancho Cucamonga
Tested By: S. Felter Date: 12/23/13

APPENDIX D

SUMMARY OF SECONDARY SEISMIC HAZARD ANALYSIS

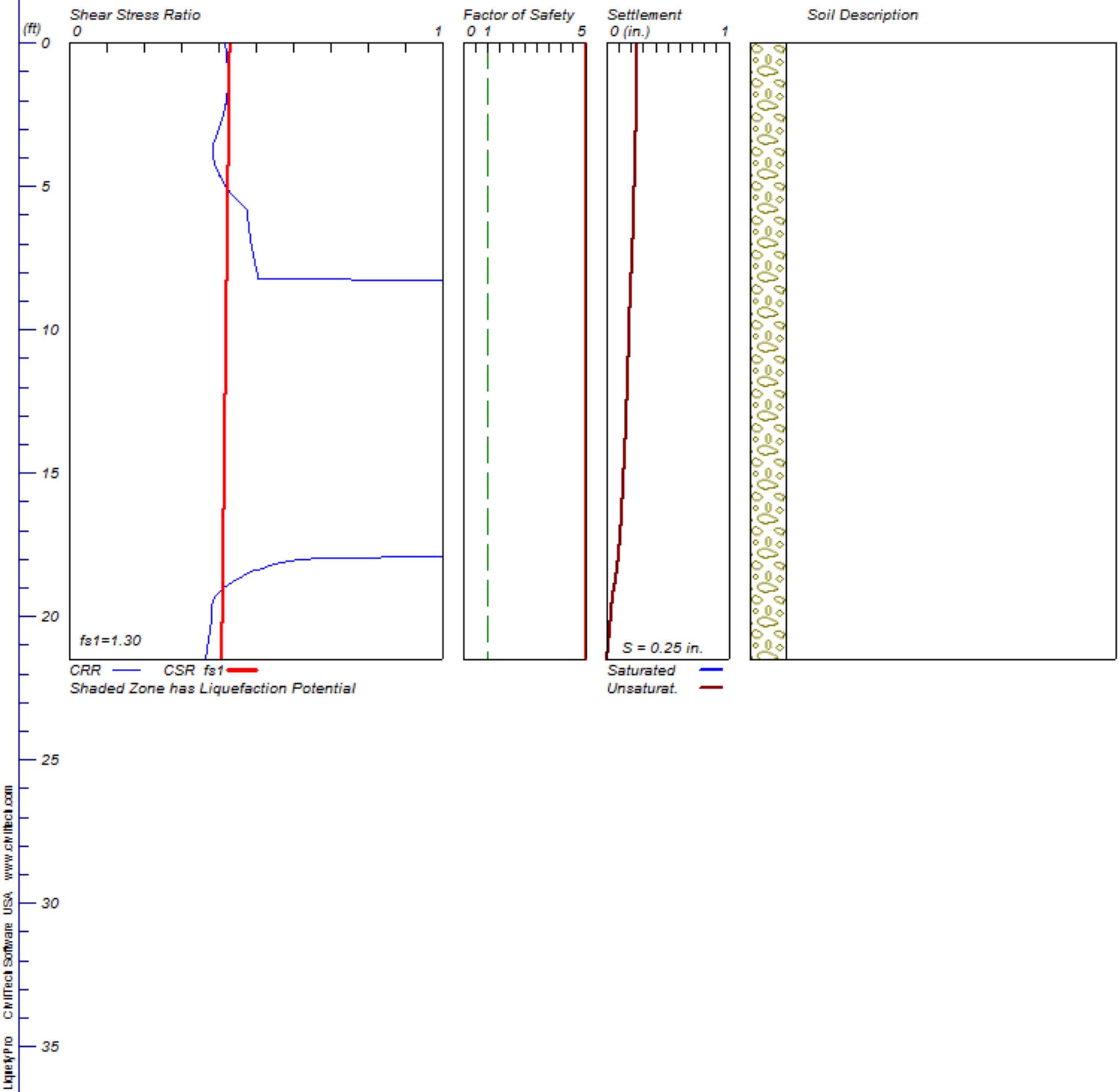
Leighton

SEISMIC SETTLEMENT ANALYSIS

Coastal Commerce Chino

Hole No.=LB-1 Water Depth=100 ft Surface Elev.=849

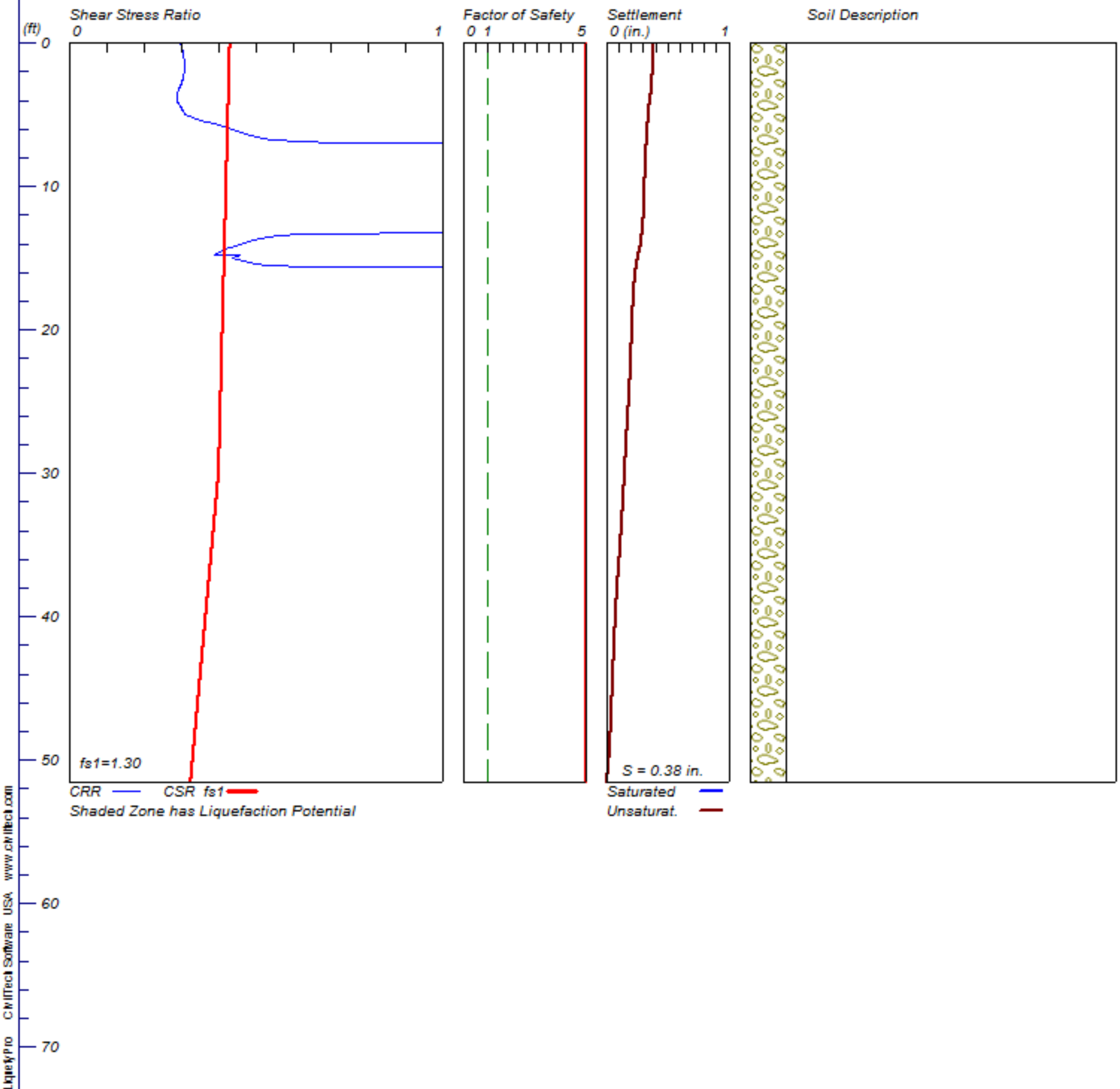
Magnitude=6.57
Acceleration=0.51g



SEISMIC SETTLEMENT ANALYSIS

Coastal Commerce Chino

Hole No.=LB-2 Water Depth=100 ft Surface Elev.=844 ***Magnitude=6.57 Acceleration=0.51g***



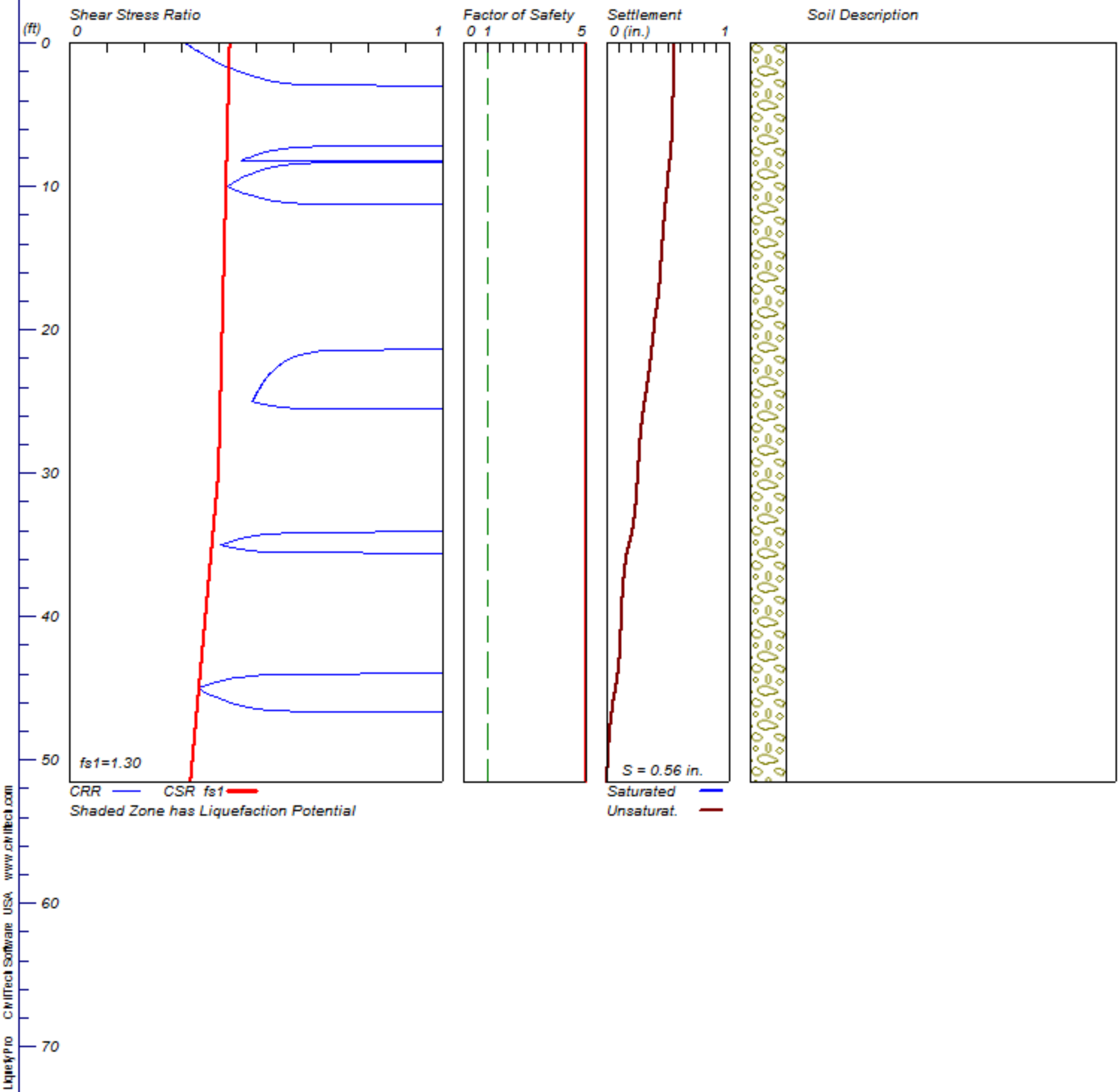
LEIGHTON **10557.004** **Plate A-2**

SEISMIC SETTLEMENT ANALYSIS

Coastal Commerce Chino

Hole No.=LB-3 Water Depth=100 ft Surface Elev.=852

Magnitude=6.57
Acceleration=0.51g

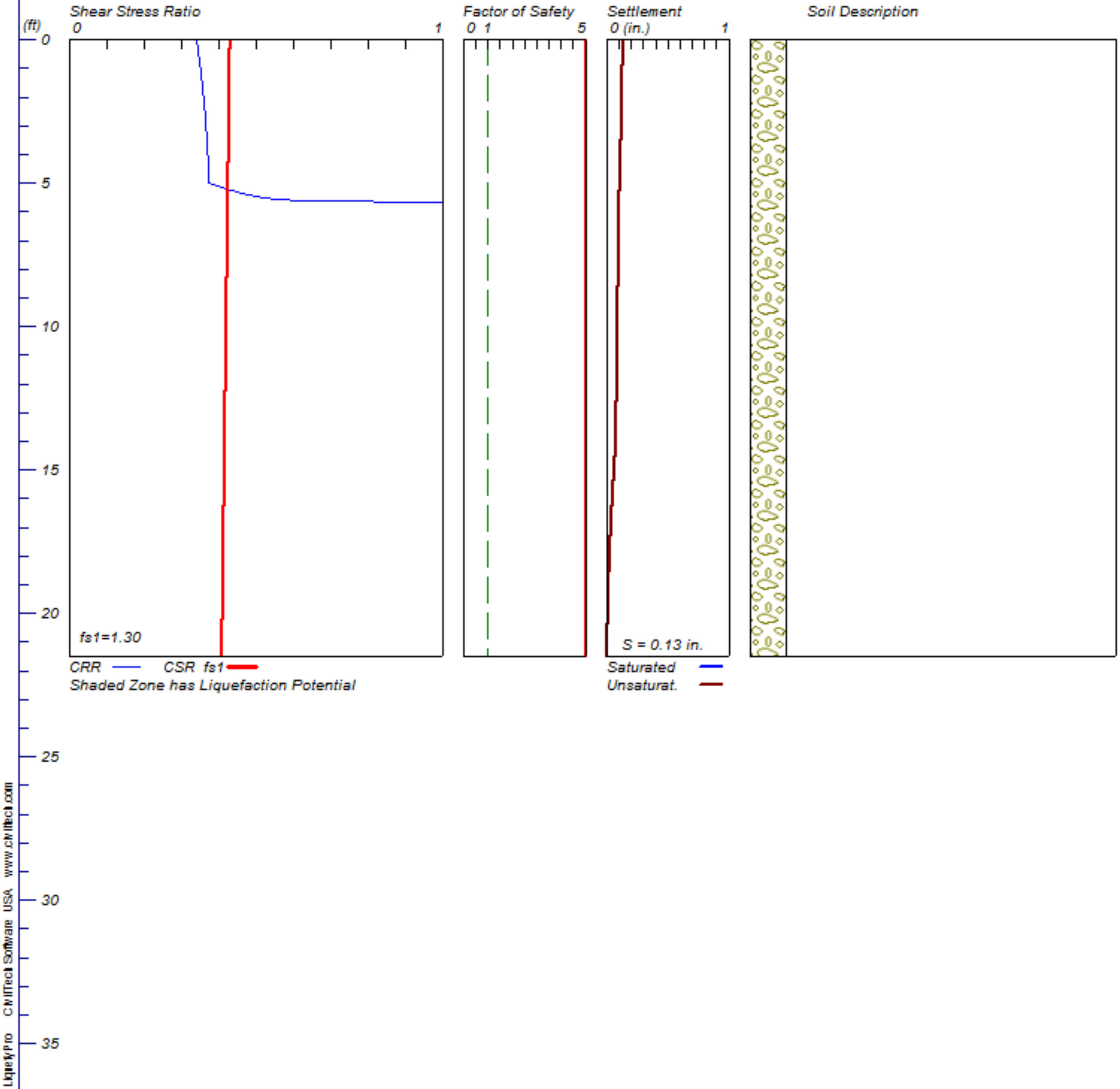


SEISMIC SETTLEMENT ANALYSIS

Coastal Commerce Chino

Hole No.=LB-4 Water Depth=100 ft Surface Elev.=850

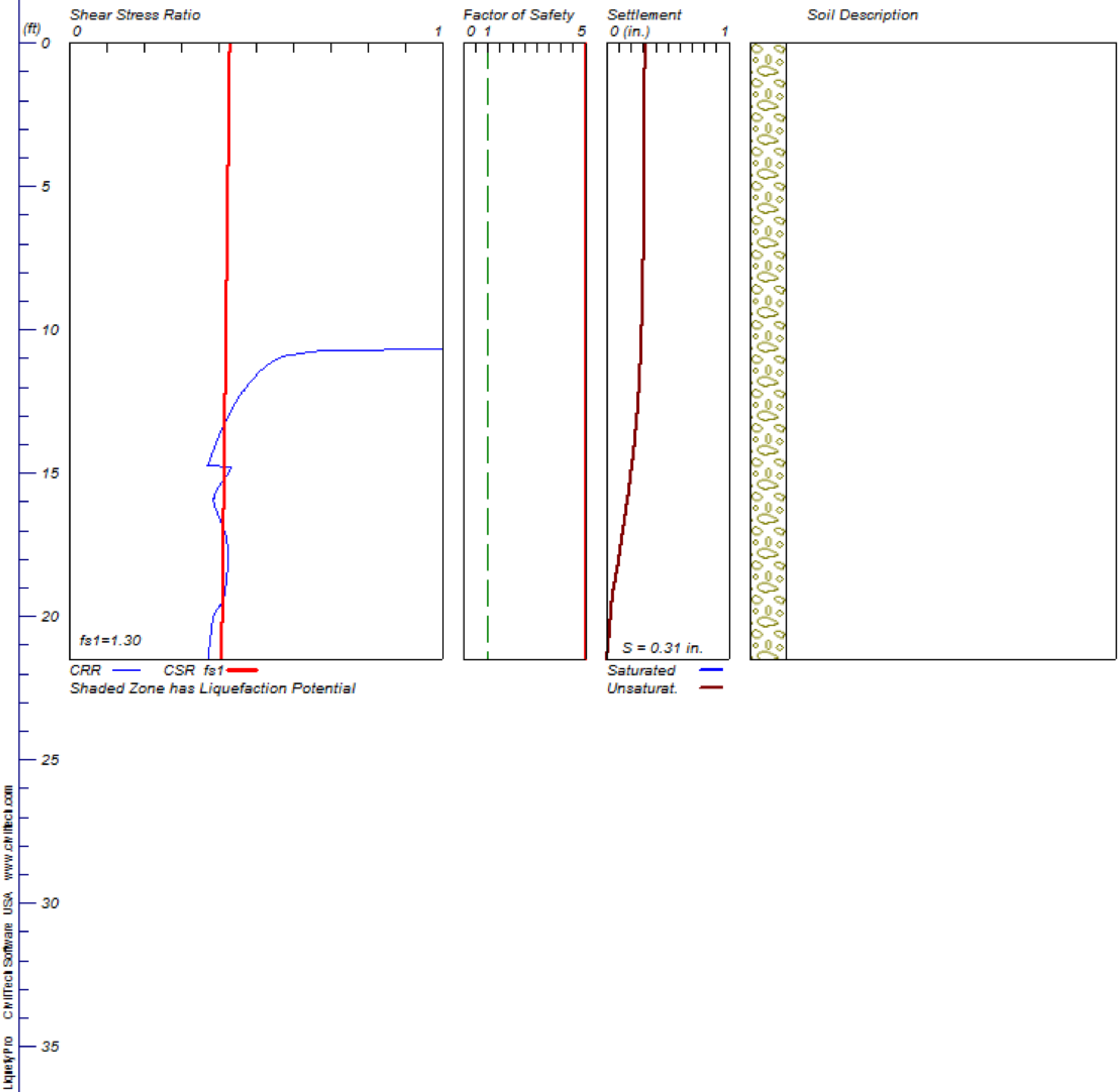
Magnitude=6.57
Acceleration=0.51g



SEISMIC SETTLEMENT ANALYSIS
Coastal Commerce Chino

Hole No.=LB-5 Water Depth=100 ft Surface Elev.=848

Magnitude=6.57
Acceleration=0.51g



APPENDIX E

GENERAL EARTHWORK AND GRADING SPECIFICATIONS

Leighton

APPENDIX E
LEIGHTON AND ASSOCIATES, INC.
GENERAL EARTHWORK AND GRADING SPECIFICATIONS FOR ROUGH GRADING

TABLE OF CONTENTS

Leighton

1.0 GENERAL

1.1 Intent

These General Earthwork and Grading Specifications are for the grading and earthwork shown on the approved grading plan(s) and/or indicated in the geotechnical report(s). These Specifications are a part of the recommendations contained in the geotechnical report(s). In case of conflict, the specific recommendations in the geotechnical report shall supersede these more general Specifications. Observations of the earthwork by the project Geotechnical Consultant during the course of grading may result in new or revised recommendations that could supersede these specifications or the recommendations in the geotechnical report(s).

1.2 The Geotechnical Consultant of Record

Prior to commencement of work, the owner shall employ the Geotechnical Consultant of Record (Geotechnical Consultant). The Geotechnical Consultants shall be responsible for reviewing the approved geotechnical report(s) and accepting the adequacy of the preliminary geotechnical findings, conclusions, and recommendations prior to the commencement of the grading.

Prior to commencement of grading, the Geotechnical Consultant shall review the "work plan" prepared by the Earthwork Contractor (Contractor) and schedule sufficient personnel to perform the appropriate level of observation, mapping, and compaction testing.

During the grading and earthwork operations, the Geotechnical Consultant shall observe, map, and document the subsurface exposures to verify the geotechnical design assumptions. If the observed conditions are found to be significantly different than the interpreted assumptions during the design phase, the Geotechnical Consultant shall inform the owner, recommend appropriate changes in design to accommodate the observed conditions, and notify the review agency where required. Subsurface areas to be geotechnically observed, mapped, elevations recorded, and/or tested include natural ground after it has been cleared for receiving fill but before fill is placed, bottoms of all "remedial removal" areas, all key bottoms, and benches made on sloping ground to receive fill.

The Geotechnical Consultant shall observe the moisture-conditioning and processing of the subgrade and fill materials and perform relative compaction testing of fill to determine the attained level of compaction.

The Geotechnical Consultant shall provide the test results to the owner and the Contractor on a routine and frequent basis.

1.3 The Earthwork Contractor

The Earthwork Contractor (Contractor) shall be qualified, experienced, and knowledgeable in earthwork logistics, preparation and processing of ground to receive fill, moisture-conditioning and processing of fill, and compacting fill. The Contractor shall review and accept the plans, geotechnical report(s), and these Specifications prior to commencement of grading. The Contractor shall be solely responsible for performing the grading in accordance with the plans and specifications.

The Contractor shall prepare and submit to the owner and the Geotechnical Consultant a work plan that indicates the sequence of earthwork grading, the number of "spreads" of work and the estimated quantities of daily earthwork contemplated for the site prior to commencement of grading. The Contractor shall inform the owner and the Geotechnical Consultant of changes in work schedules and updates to the work plan at least 24 hours in advance of such changes so that appropriate observations and tests can be planned and accomplished. The Contractor shall not assume that the Geotechnical Consultant is aware of all grading operations.

The Contractor shall have the sole responsibility to provide adequate equipment and methods to accomplish the earthwork in accordance with the applicable grading codes and agency ordinances, these Specifications, and the recommendations in the approved geotechnical report(s) and grading plan(s). If, in the opinion of the Geotechnical Consultant, unsatisfactory conditions, such as unsuitable soil, improper moisture condition, inadequate compaction, insufficient buttress key size, adverse weather, etc., are resulting in a quality of work less than required in these specifications, the Geotechnical Consultant shall reject the work and may recommend to the owner that construction be stopped until the conditions are rectified.

2.0 PREPARATION OF AREAS TO BE FILLED

2.1 Clearing and Grubbing

Vegetation, such as brush, grass, roots, and other deleterious material shall be sufficiently removed and properly disposed of in a method acceptable to the owner, governing agencies, and the Geotechnical Consultant.

The Geotechnical Consultant shall evaluate the extent of these removals depending on specific site conditions. Earth fill material shall not contain more than 1 percent of organic materials (by volume). No fill lift shall contain more than 5 percent of organic matter. Nesting of the organic materials shall not be allowed.

If potentially hazardous materials are encountered, the Contractor shall stop work in the affected area, and a hazardous material specialist shall be informed immediately for proper evaluation and handling of these materials prior to continuing to work in that area.

As presently defined by the State of California, most refined petroleum products (gasoline, diesel fuel, motor oil, grease, coolant, etc.) have chemical constituents that are considered to be hazardous waste. As such, the indiscriminate dumping or spillage of these fluids onto the ground may constitute a misdemeanor, punishable by fines and/or imprisonment, and shall not be allowed.

2.2 Processing

Existing ground that has been declared satisfactory for support of fill by the Geotechnical Consultant shall be scarified to a minimum depth of 6 inches. Existing ground that is not satisfactory shall be overexcavated as specified in the following section. Scarification shall continue until soils are broken down and free of large clay lumps or clods and the working surface is reasonably uniform, flat, and free of uneven features that would inhibit uniform compaction.

2.3 Overexcavation

In addition to removals and overexcavations recommended in the approved geotechnical report(s) and the grading plan, soft, loose, dry, saturated, spongy, organic-rich, highly fractured or otherwise unsuitable ground shall be overexcavated to competent ground as evaluated by the Geotechnical Consultant during grading.

2.4 Benching

Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical units), the ground shall be stepped or benched. Please see the Standard Details for a graphic illustration. The lowest bench or key shall be a minimum of 15 feet wide and at least 2 feet deep, into competent material as evaluated by the Geotechnical Consultant. Other benches shall be excavated a minimum height of 4 feet into competent material or as otherwise recommended by the Geotechnical

Consultant. Fill placed on ground sloping flatter than 5:1 shall also be benched or otherwise overexcavated to provide a flat subgrade for the fill.

2.5 Evaluation/Acceptance of Fill Areas

All areas to receive fill, including removal and processed areas, key bottoms, and benches, shall be observed, mapped, elevations recorded, and/or tested prior to being accepted by the Geotechnical Consultant as suitable to receive fill. The Contractor shall obtain a written acceptance from the Geotechnical Consultant prior to fill placement. A licensed surveyor shall provide the survey control for determining elevations of processed areas, keys, and benches.

3.0 FILL MATERIAL

3.1 General

Material to be used as fill shall be essentially free of organic matter and other deleterious substances evaluated and accepted by the Geotechnical Consultant prior to placement. Soils of poor quality, such as those with unacceptable gradation, high expansion potential, or low strength shall be placed in areas acceptable to the Geotechnical Consultant or mixed with other soils to achieve satisfactory fill material.

3.2 Oversize

Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 8 inches, shall not be buried or placed in fill unless location, materials, and placement methods are specifically accepted by the Geotechnical Consultant. Placement operations shall be such that nesting of oversized material does not occur and such that oversize material is completely surrounded by compacted or densified fill. Oversize material shall not be placed within 10 vertical feet of finish grade or within 2 feet of future utilities or underground construction.

3.3 Import

If importing of fill material is required for grading, proposed import material shall meet the requirements of Section 3.1. The potential import source shall be given to the Geotechnical Consultant at least 48 hours (2 working days) before importing begins so that its suitability can be determined and appropriate tests performed.

4.0 FILL PLACEMENT AND COMPACTION

4.1 Fill Layers

Approved fill material shall be placed in areas prepared to receive fill (per Section 3.0) in near-horizontal layers not exceeding 8 inches in loose thickness. The Geotechnical Consultant may accept thicker layers if testing indicates the grading procedures can adequately compact the thicker layers. Each layer shall be spread evenly and mixed thoroughly to attain relative uniformity of material and moisture throughout.

4.2 Fill Moisture Conditioning

Fill soils shall be watered, dried back, blended, and/or mixed, as necessary to attain a relatively uniform moisture content at or slightly over optimum. Maximum density and optimum soil moisture content tests shall be performed in accordance with the American Society of Testing and Materials (ASTM Test Method D1557).

4.3 Compaction of Fill

After each layer has been moisture-conditioned, mixed, and evenly spread, it shall be uniformly compacted to not less than 90 percent of maximum dry density (ASTM Test Method D1557). Compaction equipment shall be adequately sized and be either specifically designed for soil compaction or of proven reliability to efficiently achieve the specified level of compaction with uniformity.

4.4 Compaction of Fill Slopes

In addition to normal compaction procedures specified above, compaction of slopes shall be accomplished by backrolling of slopes with sheepsfoot rollers at increments of 3 to 4 feet in fill elevation, or by other methods producing satisfactory results acceptable to the Geotechnical Consultant. Upon completion of grading, relative compaction of the fill, out to the slope face, shall be at least 90 percent of maximum density per ASTM Test Method D1557.

4.5 Compaction Testing

Field-tests for moisture content and relative compaction of the fill soils shall be performed by the Geotechnical Consultant. Location and frequency of tests shall be at the Consultant's discretion based on field conditions encountered. Compaction test locations will not necessarily be selected on a random basis. Test locations shall be selected to verify adequacy of compaction levels in areas that are judged to be prone to inadequate compaction (such as close to slope faces and at the fill/bedrock benches).

4.6 <u>Frequency of Compaction Testing</u>

Tests shall be taken at intervals not exceeding 2 feet in vertical rise and/or 1,000 cubic yards of compacted fill soils embankment. In addition, as a guideline, at least one test shall be taken on slope faces for each 5,000 square feet of slope face and/or each 10 feet of vertical height of slope. The Contractor shall assure that fill construction is such that the testing schedule can be accomplished by the Geotechnical Consultant. The Contractor shall stop or slow down the earthwork construction if these minimum standards are not met.

4.7 <u>Compaction Test Locations</u>

The Geotechnical Consultant shall document the approximate elevation and horizontal coordinates of each test location. The Contractor shall coordinate with the project surveyor to assure that sufficient grade stakes are established so that the Geotechnical Consultant can determine the test locations with sufficient accuracy. At a minimum, two grade stakes within a horizontal distance of 100 feet and vertically less than 5 feet apart from potential test locations shall be provided.

5.0 SUBDRAIN INSTALLATION

Subdrain systems shall be installed in accordance with the approved geotechnical report(s), the grading plan, and the Standard Details. The Geotechnical Consultant may recommend additional subdrains and/or changes in subdrain extent, location, grade, or material depending on conditions encountered during grading. All subdrains shall be surveyed by a land surveyor/civil engineer for line and grade after installation and prior to burial. Sufficient time should be allowed by the Contractor for these surveys.

6.0 EXCAVATION

Excavations, as well as over-excavation for remedial purposes, shall be evaluated by the Geotechnical Consultant during grading. Remedial removal depths shown on geotechnical plans are estimates only. The actual extent of removal shall be determined by the Geotechnical Consultant based on the field evaluation of exposed conditions during grading. Where fill-over-cut slopes are to be graded, the cut portion of the slope shall be made, evaluated, and accepted by the Geotechnical Consultant prior to placement of materials for construction of the fill portion of the slope, unless otherwise recommended by the Geotechnical Consultant.

7.0 TRENCH BACKFILLS

7.1 Safety

The Contractor shall follow all OSHA and Cal/OSHA requirements for safety of trench excavations.

7.2 Bedding and Backfill

All bedding and backfill of utility trenches shall be performed in accordance with the applicable provisions of Standard Specifications of Public Works Construction. Bedding material shall have a Sand Equivalent greater than 30 (SE>30). The bedding shall be placed to 1 foot over the top of the conduit and densified by jetting. Backfill shall be placed and densified to a minimum of 90 percent of relative compaction from 1 foot above the top of the conduit to the surface.

The Geotechnical Consultant shall test the trench backfill for relative compaction. At least one test should be made for every 300 feet of trench and 2 feet of fill.

7.3 Lift Thickness

Lift thickness of trench backfill shall not exceed those allowed in the Standard Specifications of Public Works Construction unless the Contractor can demonstrate to the Geotechnical Consultant that the fill lift can be compacted to the minimum relative compaction by his alternative equipment and method.

7.4 Observation and Testing

The jetting of the bedding around the conduits shall be observed by the Geotechnical Consultant.

GEOTECHNICAL INVESTIGATION, PROPOSED RESIDENTIAL DEVELOPMENT, APN'S 1013-211-21 AND 1013-211-22, NORTHWEST OF FRANCIS AVENUE AND YORBA AVENUE, CITY OF CHINO, CALIFORNIA

Prepared For:

COASTAL COMMERCIAL PROPERTIES
1020 Second Street, Suite C
Encinitas, California 92024

Project No. 10557.004

August 26, 2016

Leighton and Associates, Inc.
A LEIGHTON GROUP COMPANY

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A LEIGHTON GROUP COMPANY

August 26, 2016

Project No. 10557.004

To: Coastal Commercial Properties
1020 Second Street, Suite C
Encinitas, California 92024

Attention: Mr. Brett Crowder

Subject: Geotechnical Investigation, Proposed Residential Development, APNs 1013-211-21 and 1013-211-22, Northwest of Francis Avenue and Yorba Avenue, City of Chino, California

In response to your request, Leighton and Associates, Inc. has conducted a geotechnical investigation for the proposed residential development to be located on APN 1013-211-21 and 1013-211-22, northwest of Francis Avenue and Yorba Avenue, in the City of Chino, California. This report updates our original geotechnical report for the subject property dated January 9, 2014.

Based on the results of our study, it is our professional opinion that the proposed development of the site is feasible from a geotechnical perspective, based on the current preliminary project plans. The accompanying geotechnical report presents a summary of our current investigation and provides geotechnical conclusions and recommendations.

10532 Acacia Street, Suite B-6 ▪ Rancho Cucamonga, CA 91730-5455
909.484.2205 ▪ 909.484.2170 Fax

We appreciate the opportunity to work with you on the development of this project. If you have any questions regarding this report, please call us at your convenience.

Respectfully submitted,

LEIGHTON AND ASSOCIATES, INC.

Jason D. Hertzberg, GE 2711
Principal Engineer

Philip A. Buchiarelli, CEG 1715
Principal Geologist

JDO/JDH/PB/rsm

Distribution: (2) Addressee

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1.0 INTRODUCTION

1.1 Site Location and Description

The subject property consists of approximately 12 acres and was recently utilized as grazing land for a neighboring goat farm. The property is roughly broken up into thirds, with the western third occupied by numerous small rectangular concrete pads (presumably residential structures all of which had been demolished by the mid-1990s) and one maintenance shed used for the storage of materials associated with the goats currently grazing the site. The middle third is occupied by numerous elongated concrete slabs and a few animals pens associated with a former rabbit farm (present between the 1960 and mid 1990s), bee hives, and an empty maintenance shed. The eastern third of the site is primarily vacant, with a residence containing several structures and a pool. The property slopes gently to the south.

1.2 Proposed Development

The preliminary plans that have been provided by you depict a residential development with 46 lots that we assume would be planed for single family residential homes, as well as drainage, utility, street, sidewalk, a small park, landscape and associated improvements. We would expect relatively shallow cuts and fills to achieve design grade (generally on the order of 5 feet or less).

1.3 Purpose of Investigation

This report presents the updated results of our geotechnical investigation for the subject site located northwest of Francis Avenue and Yorba Avenue in Chino, California (Figure 1). The purpose of this study has been to evaluate the general geotechnical conditions at the site with respect to the proposed development and provide preliminary geotechnical recommendations for design and construction.

Our geotechnical exploration included hollow-stem auger soil borings, laboratory testing and geotechnical analysis to evaluate the existing conditions and develop the recommendations contained in this report. We also conducted infiltration testing to evaluate general infiltration characteristics at the depths tested for water quality basin design.

1.4 Scope of Investigation

The scope of our study has included the following tasks:

- Background Review: We reviewed available, relevant geotechnical geologic maps and reports and aerial photographs available from our in-house library. This included a review of geotechnical reports previously prepared for the site.

- Utility Coordination: We contacted Underground Service Alert (USA) prior to excavating borings and test pits so that utility companies could mark utilities onsite. We also coordinated our work with you and the property representative.

- Field Exploration: Previous subsurface explorations have been performed on the site by Leighton in December of 2013. A total of 5 exploratory soil borings (LB-1 through LB-5) were logged and sampled onsite to evaluate subsurface conditions.

    - The borings were drilled to depths ranging from 21.5 to 51.5 feet below the existing ground surface (bgs) by a subcontracted drill rig operator. The borings were logged by our field representative during drilling. Relatively undisturbed soil samples were obtained at selected intervals within the borings using a California Ring Sampler. Standard Penetration Tests (SPT) were conducted at selected depths and samples were obtained. Representative bulk soil samples were also collected at shallow depths from the borings.

    - Well permeameter tests were conducted at the 5 boring locations on the site (LB-1 through LB-5) to evaluate general infiltration rates of the subsurface soils at the depths and locations tested. The well permeameter tests were conducted based on the USBR 7300-89 method. All tests were conducted at depths of about 5 to 6 feet bgs to estimate the infiltration rate for use of shallow infiltration trenches.

  All excavations were backfilled with the soil cuttings. Logs of the geotechnical borings and the well permeameter test results are presented in Appendix B. Approximate boring and well permeameter test locations are shown on the accompanying Test Location Map, Figure 2.

- Geotechnical Laboratory Testing: Geotechnical laboratory tests were conducted on selected relatively undisturbed and bulk soil samples obtained during our field investigation. This laboratory testing program was designed to evaluate engineering characteristics of site soils. Laboratory tests conducted during this investigation include:

  - In situ moisture content and dry density
  - Maximum dry density and optimum moisture content
  - Sieve analysis for grain-size distribution
  - Swell and collapse potential
  - Water-soluble sulfate concentration
  - Resistivity, chloride content and pH

  The in situ moisture content and dry density test results are shown on the boring logs, Appendix B. The other laboratory test results are presented in Appendix C.

- Engineering Analysis: Data obtained from our background review, previous field exploration and geotechnical laboratory testing was evaluated and analyzed to develop geotechnical conclusions and provide preliminary recommendations presented in this report.

- Report Preparation: Results of our preliminary geotechnical investigation have been summarized in this report, presenting our findings, conclusions and preliminary geotechnical recommendations for design and construction of the proposed residential development.

2.0 FINDINGS

2.1 Regional Geologic Conditions

The site is located within the Chino Basin in the northern portion of the Peninsular Range geomorphic province of California. Major structural features surround this region, including the Cucamonga fault and the San Gabriel Mountains to the north, the Chino fault and Puente/Chino Hills to the west, and the San Jacinto fault to the east. This is an area of large-scale crustal disturbance as the relatively northwestward-moving Peninsular Range Province collides with the Transverse Range Province (San Gabriel and San Bernardino Mountains) to the north. Several active or potentially active faults have been mapped in the region and are believed to accommodate compression associated with this collision. The site is underlain by younger alluvial soil deposits eroded from the mountains surrounding the basin and deposited in the site vicinity.

2.2 Subsurface Soil Conditions

Based upon our review of pertinent geotechnical literature and our subsurface exploration, the site is underlain by alluvial soil deposits mantled in areas of the site by minor amounts of goat manure. The manure was generally less than approximately one inch thick. The alluvial soil encountered within our excavations generally consisted of combinations of sand and silt, with some gravel interspersed. The soil was generally moist and medium dense. The in-situ moisture content within the upper approximately 15 feet generally ranged from 1 to 10 percent. More detailed descriptions of the subsurface soil are presented on the boring logs.

2.2.1 Compressible and Collapsible Soil

Soil compressibility refers to a soil's potential for settlement when subjected to increased loads as from a fill surcharge. Based on our investigation, the native soil encountered is generally considered slightly to moderately compressible. Partial removal and recompaction of this material under shallow foundations is recommended to reduce the potential for adverse total and differential settlement of the proposed improvements.

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Collapse potential refers to the potential settlement of a soil under existing stresses upon being wetted. Test results indicate that the alluvial soil within the upper 10 feet onsite has a minor collapse potential.

2.2.2 Expansive Soils

Expansive soils contain significant amounts of clay particles that swell considerably when wetted and shrink when dried. Foundations constructed on these soils are subjected to large uplifting forces caused by the swelling. Without proper measures taken, heaving and cracking of both building foundations and slabs-on-grade could result.

The near surface soils consist of sands and silty sands. Based on our observations conditions and experience in the area, the near-surface soil is generally expected to have a very low expansion potential.

2.2.3 Sulfate Content

Water-soluble sulfates in soil can react adversely with concrete. However, concrete in contact with soil containing sulfate concentrations of less than 0.1 percent by weight is considered to have negligible sulfate exposure based on the American Concrete Institute (ACI) provisions, adopted by the 2010 CBC (CBC, 2010, Chapter 19, and ACI, 2005, Chapter 4).

A near-surface soil sample was tested during this investigation for soluble sulfate content. The results of this test indicate a sulfate content of less than 0.01 percent by weight, indicating negligible sulfate exposure. Recommendations for concrete in contact with the soil are provided in Section 3.11.

2.2.4 Resistivity, Chloride and pH

Soil corrosivity to ferrous metals can be estimated by the soil's electrical resistivity, chloride content and pH. In general, soil having a minimum resistivity less than 1,000 ohm-cm is considered severely corrosive. Soil with a chloride content of 500 parts-per-million (ppm) or more is considered corrosive to ferrous metals.

As a screening for potentially corrosive soil, representative soil samples were tested during this investigation to determine minimum resistivity, chloride content, and pH. The tests indicated a minimum resistivity of 8,100 ohm-cm, chloride content of 200 ppm, and pH of 6.9. Based on the chloride content, the onsite soil is considered moderately corrosive to ferrous metals.

2.3 Groundwater

Groundwater was not encountered in our borings excavated to a maximum depth of 51.5 feet below the existing ground surface (bgs). Historical groundwater mapping indicates that groundwater was approximately 150 feet bgs in 1933 (CDWR, 1970). Recent data from the California Department of Water Resources indicates groundwater levels no higher than 200 feet bgs in the area (CDWR, 2013). Based on this, groundwater has historically been deep, and shallow groundwater is not expected at the site.

2.4 Faulting and Seismicity

Our review of available in-house literature indicates that there are no known active faults traversing the site. The closest known active or potentially active fault is the Chino-Elsinore fault, located approximately 3 miles southwest of the site.

The principal seismic hazard that could affect the site is ground shaking resulting from an earthquake occurring along several major active or potentially active faults in southern California. The known regional active and potentially active faults that could produce the most significant ground shaking at the site include the Chino-Elsinore, San Jose, Cucamonga, Sierra Madre, Whittier, Elsinore-Glen Ivy, and Elysian Park Thrust faults.

The Peak Horizontal Ground Acceleration (PHGA) and hazard deaggregation were estimated using the United States Geological Survey's (USGS) 2008 Interactive Deaggregations utility. The results of this analysis indicate that the predominant modal earthquake has a PHGA of 0.76g with magnitude of approximately 6.6 (M_W) at a distance on the order of 7 kilometers for the Maximum Considered Earthquake (2% probability of exceedance in 50 years). Based on this, the corresponding PHGA for the design earthquake (2/3 of the MCE) is 0.51g.

We also estimated the design PHGA based on the 2013 California Building Code Section 1613. The calculated S_{DS} value at the site is 1.18g (see Section 3.4). Dividing this by a factor of 2.5 results in a design peak horizontal ground acceleration (PHGA) of 0.47g, per 2013 CBC, Section 1803.5.12(2).

Based on these results, we have selected a design PHGA of 0.51g for seismic analysis of the onsite soils (seismic settlement).

2.5 Secondary Seismic Hazards

In general, secondary seismic hazards for sites in the region could include soil liquefaction, earthquake-induced settlement, lateral displacement, landsliding, and earthquake-induced flooding. The potential for secondary seismic hazards at the site is discussed below.

2.5.1 Liquefaction Potential

Liquefaction is the loss of soil strength or stiffness due to a buildup of pore-water pressure during severe ground shaking. Liquefaction is associated primarily with loose (low density), saturated, fine-to-medium grained, cohesionless soils. As the shaking action of an earthquake progresses, the soil grains are rearranged and the soil densifies within a short period of time. Rapid densification of the soil results in a buildup of pore-water pressure. When the pore-water pressure approaches the total overburden pressure, the soil reduces greatly in strength and temporarily behaves similarly to a fluid. Effects of liquefaction can include sand boils, settlement, and bearing capacity failures below structural foundations.

The State of California has not prepared liquefaction hazard maps for this area. San Bernardino County (2010) does not show the site in a zone of susceptibility for liquefaction.

Based on our study, current groundwater levels are deeper than 51.5 feet bgs and historic high groundwater levels are deeper than 150 feet bgs. As such, the potential for liquefaction at the site is very low.

2.5.2 Seismically Induced Settlement

During a strong seismic event, seismically induced settlement can occur within loose to moderately dense, dry or saturated granular soil. Settlement caused by ground shaking is often nonuniformly distributed, which can result in differential settlement.

Considering our recommended overexcavation recommendations, the potential total settlement resulting from seismic loading is considered low (less than 1 inch) for this site. Differential settlement resulting from seismic loading is generally assumed to be one-half of the total seismically induced settlement over a distance of 40 feet. Seismic settlement analysis is provided in Appendix D.

2.5.3 Seismically Induced Landslides

The site is generally level without significant slopes. This site is not considered susceptible to static slope instability or seismically induced landslides.

3.0 CONCLUSIONS AND RECOMMENDATIONS

Based on this investigation, construction of the proposed residential development is feasible from a geotechnical standpoint. No severe geologic or soils related issues were identified that would preclude development of the site for the proposed improvements. The most significant geotechnical issues at the site are those related to the potential for strong seismic shaking and potentially compressible soils. Good planning and design of the project can limit the impact of these constraints. Remedial recommendations for these and other geotechnical issues are provided in the following sections.

The site is not expected to be prone to adverse effects of slope instability or adverse differential settlement from cut/fill transitions (significant cuts and fills are not proposed).

Although not identified during this investigation, abandoned septic tanks, seepage pits, or other buried structures, trash pits, or items related to past site uses may be present. If such items were encountered during grading, they would require further evaluation and special consideration.

3.1 General Earthwork and Grading

All grading should be performed in accordance with the General Earthwork and Grading Specifications presented in Appendix E, unless specifically revised or amended below or by future recommendations based on final development plans.

3.1.1 Site Preparation

Prior to construction, the site should be cleared of vegetation, trash and debris, which should be disposed of offsite. Any underground obstructions should be removed as should large tress and their root systems. Resulting cavities should be properly backfilled and compacted. Efforts should be made to locate existing utility lines. Those lines should be removed or rerouted if they interfere with the proposed construction, and the resulting cavities should be properly backfilled and compacted. Trees should be removed.

3.1.2 Removal of Manure, Organic-Rich Soil and Uncontrolled Artificial Fill

Prior to overexcavation and recompaction of the onsite alluvial soil, all manure should be cleared and removed from the site. Heavy concentrations of organic-rich soil (containing visible organic matter or containing an organic content of 2 percent by weight or more) should be removed.

Removal and disposal of manure and organic-rich soil should be observed by Leighton and Associates. Organic content testing should be performed during removal to guide disposal operations.

In addition to the above, prior to overexcavation and recompaction of the onsite alluvial soil, any clean uncontrolled artificial fill should be removed and may be used as compacted fill for the project.

If suitable open space areas are available without proposed structures, such as a park site, it may be possible to place organic-rich soil and minor amounts of manure as non-structural fill in those areas, provided this is acceptable to the local reviewing agency. If this is done, we suggest the manure and organic-rich soils be mixed with clean soil to reduce the overall organic content and a clean soil cap be provided above the organic-rich soil.

3.1.3 Overexcavation and Recompaction

To reduce the potential for adverse differential settlement of the proposed improvements, the underlying subgrade soil should be prepared in such a manner that a uniform response to the applied loads is achieved. For structures with shallow foundations, we recommend that onsite alluvial soils be overexcavated and recompacted to a minimum depth of 3 feet below the bottom of the proposed footings or 5 feet below existing grade, whichever is deeper. Overexcavation and recompaction should extend a minimum horizontal distance of 5 feet from perimeter edges of the proposed footings.

Local conditions may require that deeper overexcavation be performed; such areas should be evaluated by Leighton during grading.

Areas outside these overexcavation limits planned for asphalt or concrete pavement, flatwork, and site walls, and areas to receive fill should be overexcavated to a minimum depth of 24 inches below the existing ground surface or 12 inches below the proposed subgrade, whichever is deeper.

After completion of the overexcavation, and prior to fill placement, the exposed surfaces should be scarified to a minimum depth of 6 inches, moisture conditioned to or slightly above optimum moisture content, and recompacted to a minimum 90 percent relative compaction, relative to the ASTM D 1557 laboratory maximum density.

These recommendations should be reviewed once a grading plan is available.

3.1.4 Fill Placement and Compaction

Manure and organic-rich soil is considered unsuitable for support of the proposed improvements, and will require offsite disposal or placement in non-structural areas. All structural fill should be visibly free of organic matter or should have a total organic matter content of less than 2.0 percent.

Onsite soil to be used for compacted structural fill should also be free of debris and oversized material (greater than 8 inches in largest dimension). Any soil to be placed as fill, whether onsite or imported material, should be reviewed and possibly tested by Leighton.

All fill soil should be placed in thin, loose lifts, moisture conditioned, as necessary, and compacted to a minimum 90 percent relative compaction. Relative compaction should be determined in accordance with ASTM Test Method D1557. Aggregate base for pavement should be compacted to a minimum of 95 percent relative compaction.

3.1.5 Import Fill Soil

Import soil to be placed as fill should be geotechnically accepted by Leighton. Preferably at least 3 working days prior to proposed import to the site, the contractor should provide Leighton pertinent information of the proposed import soil, such as location of the soil, whether stockpiled or native in place, and pertinent geotechnical reports if available. We recommend that a Leighton representative visit the proposed import site to observe the soil conditions and obtain representative soil samples. Potential issues may include soil that is more expansive than onsite soil, soil that is too wet, soil that is too rocky or too dissimilar to onsite soils, oversize material, organics, debris, etc.

3.1.6 Shrinkage and Subsidence

The change in volume of excavated and recompacted soil varies according to soil type and location. This volume change is represented as a percentage increase (bulking) or decrease (shrinkage) in volume of fill after removal and recompaction. Subsidence occurs as in-place soil (e.g., natural ground) is moisture-conditioned and densified to receive fill, such as in processing an overexcavation bottom. Subsidence is in addition to shrinkage due to recompaction of fill soil. Field and laboratory data used in our calculations included laboratory-measured maximum dry densities for soil types encountered at the subject site, the measured in-place densities of soils encountered and our experience. We preliminarily estimate the following earth volume changes will occur during grading:

Shrinkage	Approximately 15 +/- 5 percent
Subsidence (overexcavation bottom processing)	Approximately 0.15 feet

It should be noted that these values do not account for removal of manure and organic-rich soil.

The level of fill compaction, variations in the dry density of the existing soils and other factors influence the amount of volume change. Some adjustments to earthwork volume should be anticipated during grading of the site.

3.1.7 Rippability and Oversized Material

Oversized material (rock or rock fragments greater than 8 inches in dimension) was not observed during our investigation. Oversized material should not be used within structural fill areas.

3.2 Shallow Foundation Recommendations

Overexcavation and recompaction of the footing subgrade soil should be performed as detailed in Section 3.1. The following recommendations are based on the onsite soil conditions and soils with a very low expansion potential.

3.2.1 Minimum Embedment and Width

Based on our preliminary investigation, footings should have a minimum embedment of 18 inches, with a minimum width of 24 and 12 inches for isolated and continuous footings, respectively.

3.2.2 Allowable Bearing

An allowable bearing pressure of 1,800 pounds-per-square-foot (psf) may be used, based on the minimum embedment depth and width above. This allowable bearing value may be increased by 250 psf per foot increase in depth or width to a maximum allowable bearing pressure of 2,500 psf. If higher bearing pressures are required, this should be reviewed on a case-by-case basis. These allowable bearing pressures are for total dead load and sustained live loads. Footing reinforcement should be designed by the structural engineer.

3.2.3 Lateral Load Resistance

Soil resistance available to withstand lateral loads on a shallow foundation is a function of the frictional resistance along the base of the footing and the passive resistance that may develop as the face of the structure tends to move into the soil. The frictional resistance between the base of the foundation and the subgrade soil may be computed using a coefficient of friction of 0.30. The passive resistance may be computed using an allowable equivalent fluid pressure of 240 pounds per cubic foot (pcf), assuming there is constant contact between the footing and undisturbed soil. The coefficient of friction and passive resistance may be combined without further reduction.

3.2.4 Increase in Bearing and Friction - Short Duration Loads

The allowable bearing pressure and coefficient of friction values may be increased by one-third when considering loads of short duration, such as those imposed by wind and seismic forces.

3.3 Recommendations for Slabs-On-Grade

Concrete slabs-on-grade should be designed by the structural engineer in accordance with the current CBC for a soil with a very low expansion potential. Where conventional light floor loading conditions exist, the following minimum recommendations should be used. More stringent requirements may be required by local agencies, the structural engineer, the architect, or the CBC. Laboratory testing should be conducted at finish grade to evaluate the Expansion Index (EI) of near-surface subgrade soils. Slabs-on-grade should have the following minimum recommended components:

Subgrade Moisture Conditioning: The subgrade soil should be moisture conditioned to at least 2 percent above optimum moisture content to a minimum depth of 18 inches prior to placing steel or concrete.

- Moisture Vapor Retarder: A minimum of a 10-mil vapor retarder should be placed below slabs where moisture-sensitive floor coverings or equipment is planned. Since moisture will otherwise be transmitted up from the soil through the concrete, it is important that an intact vapor retarder be installed. We recommend that the vapor retarder meet the requirements of ASTM E1745 and be installed per ASTM E1643. The structural engineer should specify pertinent concrete design parameters and moisture migration prevention measures, such as whether a sand blotter layer should be placed over the vapor retarder. Gravel or other protruding objects that could puncture the moisture retarder should be removed from the subgrade prior to placing the vapor retarder, or a stronger vapor retarder intended for the specific conditions present can be used.

- Concrete Thickness: Slabs-on-grade should be at least 4 inches thick. Reinforcing steel should be designed by the structural engineer, but as a minimum should be No. 4 rebar placed at 18 inches on center, each direction, mid-depth in the slab.

Minor cracking of the concrete as it cures, due to drying and shrinkage is normal and should be expected. However, cracking is often aggravated by a high water/cement ratio, high concrete temperature at the time of placement, small nominal aggregate size, and rapid moisture loss due to hot, dry, and/or windy weather conditions during placement and curing. Cracking due to temperature and moisture fluctuations can also be expected. Low slump concrete can reduce the potential for shrinkage cracking. Additionally, our experience indicates that reinforcement in slabs and foundations can generally reduce the potential for concrete cracking. The structural engineer should consider these components in slab design and specifications.

Moisture retarders can reduce, but not eliminate moisture vapor rise from the underlying soils up through the slab. Floor covering manufacturers should be consulted for specific recommendations.

Leighton does not practice in the field of moisture vapor transmission evaluation, since this is not specifically a geotechnical issue. Therefore, we recommend that a qualified person, such as the flooring subcontractor and/or structural engineer, be consulted with to evaluate the general and specific moisture vapor transmission paths and any impact on the proposed construction. That person should provide recommendations for mitigation of potential adverse impact of moisture vapor transmission on various components of the structures as deemed appropriate.

3.4 Seismic Design Parameters

Seismic parameters presented in this report should be considered during project design. In order to reduce the effects of ground shaking produced by regional seismic events, seismic design should be performed in accordance with the most recent edition of the California Building Code (CBC). The following data should be considered for the seismic analysis of the subject site:

2013 CBC Categorization/Coefficient	Design Value
Site Longitude (decimal degrees)	-117.704
Site Latitude (decimal degrees)	34.042
Site Class Definition (ASCE 7 Table 20.3-1)	D
Mapped Spectral Response Acceleration at 0.2s Period, S_s (Figure 1613.3.1(1))	1.771 g
Mapped Spectral Response Acceleration at 1s Period, S_1 (Figure 1613.3.1(2))	0.628 g
Short Period Site Coefficient at 0.2s Period, F_a (Table 1613.3.3(1))	1.0
Long Period Site Coefficient at 1s Period, F_v (Table 1613.3.3(2)	1.5
Adjusted Spectral Response Acceleration at 0.2s Period, S_{MS} (Eq. 16-37)	1.771 g
Adjusted Spectral Response Acceleration at 1s Period, S_{M1} (Eq. 16-38)	0.941 g
Design Spectral Response Acceleration at 0.2s Period, S_{DS} (Eq. 16-39)	1.181 g
Design Spectral Response Acceleration at 1s Period, S_{D1} (Eq. 16-40)	0.628 g

3.5 Retaining Walls

We recommend that retaining walls be backfilled with very low expansive soil and constructed with a backdrain in accordance with the recommendations provided on Figure 3 (rear of text). Using expansive soil as retaining wall backfill will result in higher lateral earth pressures exerted on the wall. Based on these recommendations, the following parameters may be used for the design of conventional retaining walls:

Static Equivalent Fluid Weight (pcf)	
Condition	Level Backfill
Active	35 pcf
At-Rest	55 pcf
Passive	240 pcf (allowable) (Maximum of 3,500 psf)

The above values do not contain an appreciable factor of safety unless noted, so the structural engineer should apply the applicable factors of safety and/or load factors during design, as specified by the California Building Code.

Cantilever walls that are designed to yield at least 0.001H, where H is equal to the wall height, may be designed using the active condition. Rigid walls and walls braced at the top should be designed using the at-rest condition.

Passive pressure is used to compute soil resistance to lateral structural movement. In addition, for sliding resistance, a frictional resistance coefficient of 0.3 may be used at the concrete and soil interface. The lateral passive resistance should be taken into account only if it is ensured that the soil providing passive resistance, embedded against the foundation elements, will remain intact with time.

In addition to the above lateral forces due to retained earth, surcharge due to improvements, such as an adjacent structure or traffic loading, should be considered in the design of the retaining wall. Loads applied within a 1:1 projection from the surcharging structure on the stem of the wall should be considered in the design.

A soil unit weight of 120 pcf may be assumed for calculating the actual weight of the soil over the wall footing.

3.6 Infiltration Design

Five well permeameter tests (LB-1 through LB-5) were conducted to estimate the infiltration rate in various parts of the site. The well permeameter tests were conducted at depths between 4 and 6 feet below ground surface.

Well permeameter tests are useful for field measurements of soil infiltration rates, and is suited for testing when the design depth of the basin is deeper than current existing grades. The test consists of excavating a boring to the depth of the test (or deeper if it is partially backfilled with soil and a bentonite plug with a thin soil covering is placed just below the design test elevation). A layer of clean sand is placed in the boring bottom to support a float mechanism and temporary perforated well casing pipe. In addition, sand is poured around the outside of the well casing within the test zone to prevent the boring from caving/collapsing or eroding when water is added. The float mechanism, placed inside the casing, adds water stored in barrels at the top of the hole to the boring as water infiltrates into the soil, while maintaining a constant water head in the boring. The test was conducted based on the USBR 7300-89 test method. The incremental infiltration rate as measured during intervals of the test is defined as the incremental flow rate of water infiltrated, divided by the surface area of the infiltration interface.

Small-scale infiltration rates were measured at the 5 well permeameter locations and ranged from approximately 0.3 to 13 inches per hour (no factor of safety applied). Infiltration at three of the five locations was too rapid to measure for normal test procedures. One of these three locations was selected based on the boring geology as the probable fastest infiltration location, and a modified test procedure was used to test the infiltration rate using a lower water surface head. The result of this test indicated an infiltration rate of 13 inches per hour. Infiltration test results are provided in Appendix B. These are raw values, before applying an appropriate factor of safety or correction factor. Based on these results, the onsite silty soils or soils with a higher fines content are not considered feasible for infiltration. Sandy soils with a low fines content are anticipated to have higher infiltration rates; however, sandy soils underlain by finer-grained soils are not considered suitable. Specific infiltration design information should be made available so testing representative to the final design conditions can be conducted. The small-scale infiltration rate should be divided by a correction factor of at least 2 for buried chambers and at least 3 for open basins, but the correction/safety factor may be higher based on project-specific aspects, based on *San Bernardino County Stormwater Program Technical Guidance Document for Water Quality Management Plans (WQMP)*.

We recommend that further testing be conducted after a design has been selected for an infiltration facility, since infiltration rates varied significantly across the site.

The infiltration rates described herein are for a clean, unsilted infiltration surface in native, sandy alluvial soil. These values may be reduced over time as silting of the basin or chamber occurs. Furthermore, if the basin or chamber bottom is allowed to be compacted by heavy equipment, this value is expected to be significantly reduced. Infiltration of water through soil is highly dependent on such factors as grain size distribution of the soil particles, particle shape, fines content, clay content, and density. Small changes in soil conditions, including density, can cause large differences in observed infiltration rates. Infiltration is not suitable in compacted fill.

It should be noted that during periods of prolonged precipitation, the underlying soils tend to become saturated to greater and greater depths/extents. Therefore, infiltration rates tend to decrease with prolonged rainfall. It is difficult to extrapolate longer-term, full-scale infiltration rates from small-scale tests, and as such, this is a significant source of uncertainty in infiltration rates.

Additional Review and Evaluation:

Infiltration rates are anticipated to vary significantly based on the location and depth. Infiltration concepts should be discussed with Leighton as infiltration plans are being developed. Leighton should review all infiltration plans, including locations and depths of proposed facilities. Further testing should be conducted based on the design of infiltration facilities, particularly considering their type, depth and location.

General Design Considerations:

The periodic flow of water carrying sediments in the basin or chamber, plus the introduction of wind-blown sediments and sediments from erosion of the basin side walls, can eventually cause the bottom of the basin or chamber to accumulate a layer of silt, which has the potential of significantly reducing the overall infiltration rate of the basin or chamber. Therefore, we recommend that significant amounts of silt/sediment not be allowed to flow into the facility within storm water, especially during construction of the project and prior to achieving a mature landscape on site. We recommend that an easily maintained, robust silt/sediment removal system be installed to pretreat storm water before it enters the infiltration facility.

As infiltrating water can seep within the soil strata nearly horizontally for long distances, it is important to consider the impact that infiltration facilities can have on nearby subterranean structures, such as basement walls or open excavations, whether onsite or offsite, and whether existing or planned. Any such nearby features should be identified and evaluated as to whether infiltrating water can impact these. Such features should be brought to Leighton's attention as they are identified.

Infiltration facilities should not be constructed adjacent to or under buildings. Setbacks should be discussed with Leighton during the planning process.

Infiltration facilities should be constructed with spillways or other appropriate means that would cause overfilling to not be a concern to the facility or nearby improvements.

For buried chambers that allow interior standing water, control/access manhole covers should not contain holes or should be screened to prevent mosquitos from entering the cambers.

Additional Design Considerations (Particularly for Open Basins):
If open basins are planned, additional infiltration exploration and testing should be conducted, as the soils that will be exposed at the bottom of the basin are critical to the basin's success. Soils at the bottom of buried chambers are also important, but not as critical to their success, provided the infiltration chamber cuts through sufficiently granular soils.

In general, the rate of infiltration reduces as the head of water in the infiltration facility reduces, and it also reduces with prolonged periods of infiltration. As such, water typically infiltrates much faster near the beginning of and/or immediately after storm events than at times well after a storm when the water level in the facility has receded, since the infiltration rate is then slower due to both lower head and longer overall duration of infiltration. In open basins with compacted or silty bottoms, this could be problematic, in that, even if the basin had already infiltrated significant amounts of storm water, the lower several inches or feet of water could remain in the basin for an extended period of time, creating a prolonged open-water safety concern and potential for mosquitos. In a buried/covered infiltration chamber, these conditions would be of less concern.

Parks or play/recreation areas should not be constructed within basin bottoms or below the spillway level.

For open basins and swales, vegetation within the basin bottoms and sides is expected to help reduce erosion and help maintain infiltration rates.

Estimating infiltration rates, especially based on small-scale testing, is inexact and indefinite, and often involves known and unknown soil complexities, potentially resulting in a condition where actual infiltration rates of the completed facility are significantly less than design rates. In open infiltration basins, this could create nuisance water in the basin. As such, enhancements may be needed after completion of the basin if prolonged or frequent standing water is experienced. A potential basin enhancement, if needed, might be to install infiltration trenches or dry wells in the basin bottom to capture and infiltrate low flows and to help speed infiltration during/after storms; specific recommendations, such as minimum trench/dry well depth, would be developed based on conditions observed. Such a contingency should be anticipated for open basins.

Construction Considerations:
We recommend that Leighton evaluate the infiltration facility excavations, to confirm that granular, undisturbed alluvium is exposed in the bottoms and sides. Additional excavation or evaluation may be required if silty or clayey soils are exposed.

It is critical to infiltration that the basin or chamber bottom not be allowed to be compacted during construction or maintenance; rubber-tired equipment and vehicles should not be allowed to operate on the bottom. We recommend that at least the bottom 3 feet of the basins or chambers be excavated with an excavator or similar.

If fill material is needed to be placed in the basin, such as due to removal of uncontrolled artificial fill, the fill material should be select and free-draining sand, and should be observed and evaluated by Leighton.

Maintenance Considerations:
The infiltration facilities should be routinely monitored, especially before and during the rainy season, and corrective measures should be implemented as/when needed. Things to check for include proper upkeep, proper infiltration, absence of accumulated silt, and that de-silting filters/features are clean and functioning. Pretreatment desilting features should be cleaned and maintained per manufacturers' recommendations. Even with measures to prevent silt from flowing into the infiltration facility, accumulated silt may need to be removed occasionally as part of maintenance.

3.7 Pavement Design

Based on the design procedures outlined in the current Caltrans Highway Design Manual, and using an assumed design R-value of 50, flexible pavement sections may consist of the following for the Traffic Indices indicated. Final pavement design should be based on the Traffic Index determined by the project civil engineer and R-value testing provided near the end of grading.

Asphalt Pavement Section Thickness, Type I Subgrade Soil			
Traffic Index	Asphaltic Concrete (AC) Thickness (inches)	Class 2 Aggregate Base Thickness (inches)	Total Pavement Section Thickness (inches)
5 or less	3	4	7
6	3	4.5	7.5
7	4	4.5	8.5

All pavement construction should be performed in accordance with the Standard Specifications for Public Works Construction or Caltrans Specifications. Field observations and periodic testing, as needed during placement of the base course materials, should be undertaken to ensure that the requirements of the standard specifications are fulfilled.

Prior to placement of aggregate base, the subgrade soil should be processed to a minimum depth of 6 inches, moisture-conditioned, as necessary, and recompacted to a minimum of 90 percent relative compaction. Aggregate base should be moisture conditioned, as necessary, and compacted to a minimum of 95 percent relative compaction.

If the pavement is to be constructed prior to construction of the structures, we recommend that the full depth of the pavement section be placed in order to support heavy construction traffic.

3.8 Temporary Excavations

All temporary excavations, including utility trenches, retaining wall excavations and other excavations should be performed in accordance with project plans, specifications and all OSHA requirements.

No surcharge loads should be permitted within a horizontal distance equal to the height of cut or 5 feet, whichever is greater from the top of the slope, unless the cut is shored appropriately. Excavations that extend below an imaginary plane inclined at 45 degrees below the edge of any adjacent existing site foundation should be properly shored to maintain support of the adjacent structures.

Cantilever shoring should be designed based on an active equivalent fluid pressure of 35 pcf. If excavations are braced at the top and at specific design intervals, the active pressure may then be approximated by a rectangular soil pressure distribution with the pressure per foot of width equal to 25H, where H is equal to the depth of the excavation being shored.

During construction, the soil conditions should be regularly evaluated to verify that conditions are as anticipated. The contractor should be responsible for providing the "competent person" required by OSHA, standards to evaluate soil conditions. Close coordination between the competent person and the geotechnical engineer should be maintained to facilitate construction while providing safe excavations.

3.9 Trench Backfill

Utility-type trenches onsite can be backfilled with the onsite material, provided it is free of debris, significant organic material and oversized material. Prior to backfilling the trench, pipes should be bedded and shaded in a granular material that has a sand equivalent of 30 or greater. The sand should extend 12 inches above the top of the pipe. The bedding/shading sand should be densified in-place by mechanical means, or in accordance with Greenbook specifications. The native backfill should be placed in loose layers, moisture conditioned, as necessary, and mechanically compacted using a minimum standard of 90 percent relative compaction. The thickness of layers should be based on the compaction equipment used in accordance with the Standard Specifications for Public Works Construction (Greenbook, 2015).

3.10 Surface Drainage

Inadequate control of runoff water and/or poorly controlled irrigation can cause the onsite soils to expand and/or shrink, producing heaving and/or settlement of foundations, flatwork, walls, and other improvements. Maintaining adequate surface drainage, proper disposal of runoff water, and control of irrigation should help reduce the potential for future soil moisture problems.

Positive surface drainage should be designed to be directed away from foundations and toward approved drainage devices, such as gutters, paved drainage swales, or watertight area drains and collector pipes.

Surface drainage should be provided to prevent ponding of water adjacent to the structures. In general, the area around the buildings should slope away from the building. We recommend that unpaved landscaped areas adjacent to the buildings be avoided. Roof runoff should be carried to suitable drainage outlets by watertight drain pipes or over paved areas.

3.11 Sulfate Attack and Corrosion Protection

Based on the results of laboratory testing, concrete structures in contact with the onsite soil will have negligible exposure to water-soluble sulfates in the soil. Type II cement may be used for concrete construction. The concrete should be designed in accordance with Table 4.3.1 of the American Concrete Institute ACI 318-08 provisions (ACI, 2008).

Based on our laboratory testing, the onsite soil is considered severely corrosive to ferrous metals. Use of non-ferrous buried pipe may be prudent, or ferrous pipe can be protected by dielectric tape, polyethylene sleeves and/or other methods, with recommendations from a corrosion engineer. Corrosion information presented in this report should be provided to your underground utility subcontractors. Additional testing and evaluation by a corrosion engineer may be warranted if corrosion protection is considered critical to the project.

3.12 Additional Geotechnical Services

The preliminary geotechnical recommendations presented in this report are based on subsurface conditions as interpreted from limited subsurface explorations and limited laboratory testing. Our preliminary geotechnical recommendations provided in this report are based on information available at the time the report was prepared and may change as plans are developed. Additional geotechnical investigation and analysis may be required based on final improvement plans. Leighton should review the site and grading plans when available and comment further on the geotechnical aspects of the project. Geotechnical observation and testing should be conducted during excavation and all phases of grading operations. Our conclusions and preliminary recommendations should be reviewed and verified by Leighton during construction and revised accordingly if geotechnical conditions encountered vary from our preliminary findings and interpretations.

Geotechnical observation and testing should be provided:

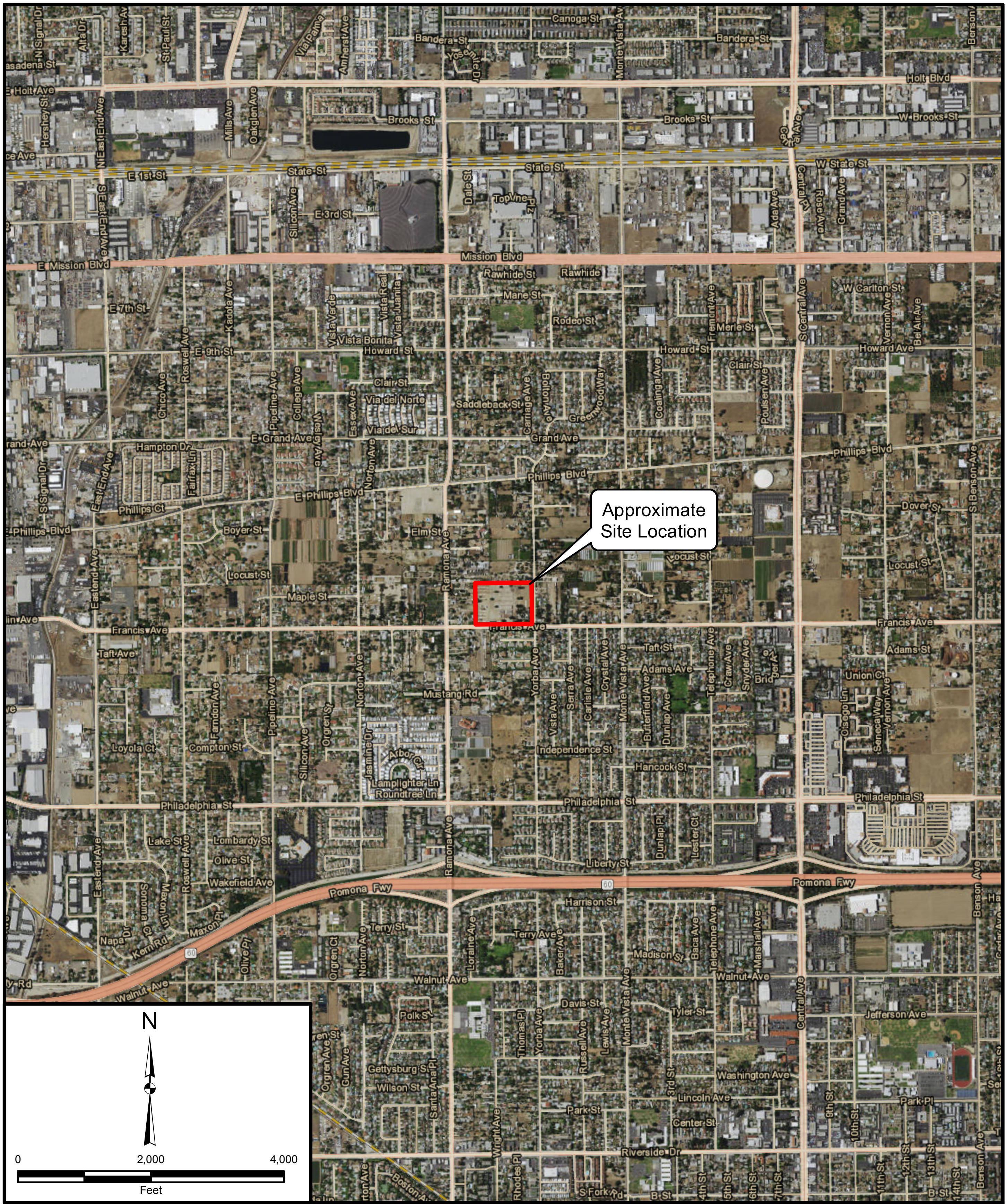
- After completion of site clearing.
- During overexcavation of compressible soil.
- During compaction of all fill materials.
- After excavation of all footings and prior to placement of concrete.
- During utility trench backfilling and compaction.
- During pavement subgrade and base preparation.
- When any unusual conditions are encountered.

4.0 LIMITATIONS

This report was based in part on data obtained from a limited number of observations, site visits, soil excavations, samples, and tests. Such information is, by necessity, incomplete. The nature of many sites is such that differing soil or geologic conditions can be present within small distances and under varying climatic conditions. Changes in subsurface conditions can and do occur over time. Therefore, our findings, conclusions, and recommendations presented in this report are based on the assumption that Leighton and Associates, Inc. will provide geotechnical observation and testing during construction.

This report was prepared for the sole use of Stratham Company for application to the design of the proposed residential development in accordance with generally accepted geotechnical engineering practices at this time in California.

See the GBA insert on the following page for important information about this geotechnical engineering report.



Project: 10557.004 | Eng/Geol: JDH/PB
Scale: 1 " = 2,000 ' | Date: August 2016

Base Map: ESRI ArcGIS Online 2016
Thematic Information: Leighton
Author: Leighton Geomatics (mmurphy)

SITE LOCATION MAP

Proposed Residential Development, Assessor Parcel Numbers 1013-211-21 and 1013-211-22, Northwest of Francis Avenue and Yorba Linda Avenue, City of Chino, California

Figure 1

Leighton

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Project: 10557.004	Eng/Geol: JDH/PB
Scale: 1 " = 150 '	Date: August 2016

Base Map: ESRI ArcGIS Online 2016
Thematic Information: Leighton
Author: Leighton Geomatics (mmurphy)

TEST LOCATION MAP

Proposed Residential Development, Assessor Parcel Numbers 1013-211-21 and 1013-211-22, Northwest of Francis Avenue and Yorba Linda Avenue, City of Chino, California

Figure 2

Leighton

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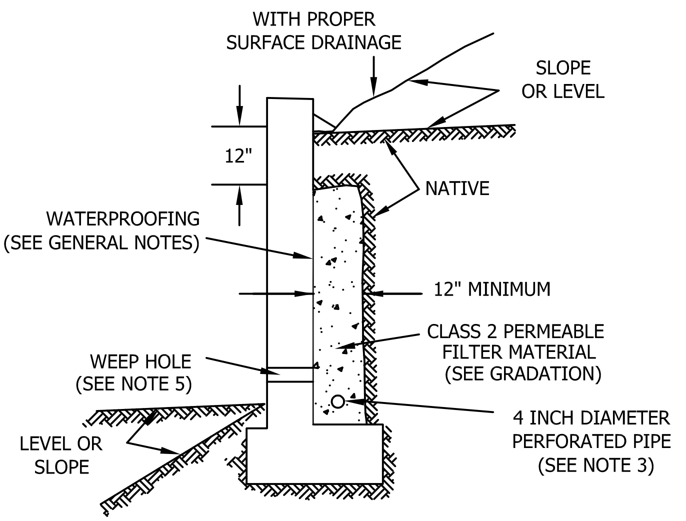
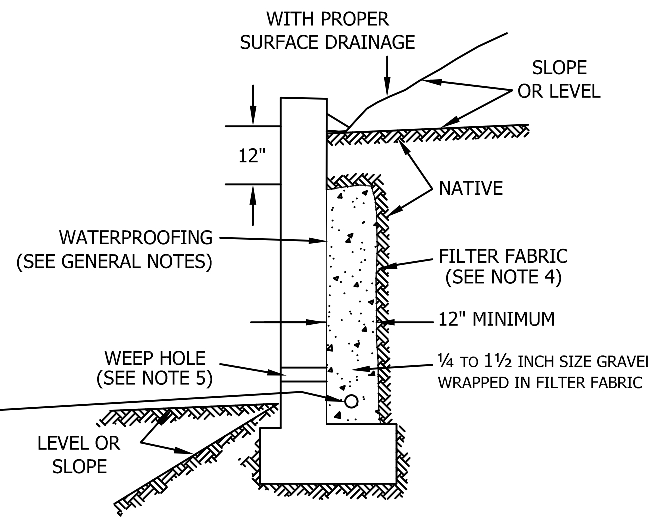
SUBDRAIN OPTIONS AND BACKFILL WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF ≤50

OPTION 1: PIPE SURROUNDED WITH CLASS 2 PERMEABLE MATERIAL



OPTION 2: GRAVEL WRAPPED IN FILTER FABRIC



Class 2 Filter Permeable Material Gradation
Per Caltrans Specifications

Sieve Size	Percent Passing
1"	100
3/4"	90-100
3/8"	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

GENERAL NOTES:

* Waterproofing should be provided where moisture nuisance problem through the wall is undesirable.
* Water proofing of the walls is not under purview of the geotechnical engineer
* All drains should have a gradient of 1 percent minimum
*Outlet portion of the subdrain should have a 4-inch diameter solid pipe discharged into a suitable disposal area designed by the project engineer. The subdrain pipe should be accessible for maintenance (rodding)
*Other subdrain backfill options are subject to the review by the geotechnical engineer and modification of design parameters.

Notes:
1) Sand should have a sand equivalent of 30 or greater and may be densified by water jetting.
2) 1 Cu. ft. per ft. of 1/4- to 1 1/2-inch size gravel wrapped in filter fabric
3) Pipe type should be ASTM D1527 Acrylonitrile Butadiene Styrene (ABS) SDR35 or ASTM D1785 Polyvinyl Chloride plastic (PVC), Schedule 40, Armco A2000 PVC, or approved equivalent. Pipe should be installed with perforations down. Perforations should be 3/8 inch in diameter placed at the ends of a 120-degree arc in two rows at 3-inch on center (staggered)
4) Filter fabric should be Mirafi 140NC or approved equivalent.
5) Weephole should be 3-inch minimum diameter and provided at 10-foot maximum intervals. If exposure is permitted, weepholes should be located 12 inches above finished grade. If exposure is not permitted such as for a wall adjacent to a sidewalk/curb, a pipe under the sidewalk to be discharged through the curb face or equivalent should be provided. For a basement-type wall, a proper subdrain outlet system should be provided.
6) Retaining wall plans should be reviewed and approved by the geotechnical engineer.
7) Walls over six feet in height are subject to a special review by the geotechnical engineer and modifications to the above requirements.

RETAINING WALL BACKFILL AND SUBDRAIN DETAIL FOR WALLS 6 FEET OR LESS IN HEIGHT

WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF ≤50

Leighton

Figure 3

P:Drafting\templates\details\retain-wall-backfill-and subdrain.dwg (7/00)

APPENDIX A

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APPENDIX A

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APPENDIX B

GEOTECHNICAL BORING LOGS

GEOTECHNICAL BORING LOG LB-1

Project No.	10557.004	**Date Drilled**	12-13-13
Project	Coastal Commerce Chino	**Logged By**	JMD
Drilling Co.	2R Drilling	**Hole Diameter**	9.5"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	849'
Location	See Figure 2	**Sampled By**	JMD

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	0			BULK					@Surface: dirt with some straw	
				R-1	4 6 11	111	2	SM	@2.5' SILTY SAND, loose, light olive brown, dry to moist, fine sand, 30% fines (field estimate), trace rootlets, trace fine gravel	
845	5			R-2	7 10 14	119	1	SP	@5' SAND, medium dense, light brown, dry, medium to coarse sand, trace fines, trace fine gravel, larger piece of gravel in ring sample	
840	10			R-3	10 15 21	121	2	SP	@10' SAND, medium dense, gray to brown, dry, medium sand, some gravel, 1.25" maximum gravel size	
835	15			R-4	7 12 17	108	10	ML	@15' SANDY SILT, very stiff, yellowish brown, dry to moist, homogenous	-200
830	20			S-5	6 8 10			ML SP	@20' SANDY SILT, very stiff, dark gray, dry to moist, fine sand @20.7' SAND, gray, dry to moist, fine to medium sand	
									Total depth of 21.5' No groundwater encountered Backfilled with soil cuttings	
825	25									
820	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

*** This log is a part of a report by Leighton and should not be used as a stand-alone document. ***

GEOTECHNICAL BORING LOG LB-2

Project No.	10557.004	**Date Drilled**	12-13-13
Project	Coastal Commerce Chino	**Logged By**	JMD
Drilling Co.	2R Drilling	**Hole Diameter**	9.5"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	844'
Location	See Figure 2	**Sampled By**	JMD

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	0			BULK					@Surface: dirt with some grass	
				R-1	3 5 7	106	2	SM	@2.5' SILTY SAND, loose, light gray brown, dry, fine sand, 30% fines (field estimate), trace fine gravel	
840	5			R-2	7 9 10			SP	@5' SAND, medium dense, reddish brown, dry, medium to coarse sand, trace fines, some gravel, 1.25" maximum gravel size	
835	10			R-3	20 24 25	126	2	SP	@10' SAND, medium dense, light gray brown, dry, medium to coarse sand, angular, broken rocks up to 2.25" in sample	
830	15			S-4	7 8 9			SP	@15' SAND, medium dense, gray, dry to moist, medium sand	
825	20			R-5	17 23 45	111	15	ML	@20' SANDY SILT, very dense, olive, moist, some FeO2 staining	
820	25			S-6	7 12 11			ML-CL	@25' SILT to CLAY, very stiff, gray, dry to moist, with FeO2 staining @25.4' SAND, dry, fine to medium sand @25.6' SILT, gray, moist @25.9' CLAY, gray, moist	
815	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

*** This log is a part of a report by Leighton and should not be used as a stand-alone document. ***

GEOTECHNICAL BORING LOG LB-2

Project No.	10557.004	**Date Drilled**	12-13-13
Project	Coastal Commerce Chino	**Logged By**	JMD
Drilling Co.	2R Drilling	**Hole Diameter**	9.5"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	844'
Location	See Figure 2	**Sampled By**	JMD

SOIL DESCRIPTION

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	30			S-7	8 16 21			ML	@30' SILT, hard, olive brown, dry to moist, FeO2 staining, with some clay @30.5' SILT, olive brown, dry to moist, FeO2 staining @31' SAND, dark reddish brown to light gray, dry, fine to medium sand	
810										
	35			S-8	18 24 21			SP	@35' SAND, light brown, dry to moist, with large amounts of FeO2 staining, trace fine gravel, a 1.25" piece of gravel in the sampler tip	
805										
	40			S-9	12 10 20			CL	@40' CLAY with gravel, hard, reddish brown to olive brown, gravel up to 2" large, with some silt, some FeO2 staining @41.3' SAND with gravel, dry to moist, medium to coarse sand, gravel up to 2" large	
800										
	45			S-10	15 35 24			SM	@45' SILTY SAND, very dense, reddish brown, moist, angular, 20% fines (field estimate), with some gravel, 1" maximum gravel size	
795										
	50			S-11	9 11 16			ML	@50' SILT, very stiff, olive brown, moist, with FeO2 staining, homogenous	
									Total depth of 51.5' No groundwater encountered Bakfilled with soil cuttings	
790										
	55									
785										
	60									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:

-200	% FINES PASSING	DS	DIRECT SHEAR	SA	SIEVE ANALYSIS
AL	ATTERBERG LIMITS	EI	EXPANSION INDEX	SE	SAND EQUIVALENT
CN	CONSOLIDATION	H	HYDROMETER	SG	SPECIFIC GRAVITY
CO	COLLAPSE	MD	MAXIMUM DENSITY	UC	UNCONFINED COMPRESSIVE STRENGTH
CR	CORROSION	PP	POCKET PENETROMETER		
CU	UNDRAINED TRIAXIAL	RV	R VALUE		

*** This log is a part of a report by Leighton and should not be used as a stand-alone document. ***

GEOTECHNICAL BORING LOG LB-3

Project No.	10557.004	**Date Drilled**	12-13-13
Project	Coastal Commerce Chino	**Logged By**	JMD
Drilling Co.	2R Drilling	**Hole Diameter**	9.5"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	852'
Location	See Figure 2	**Sampled By**	JMD

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0			BULK					@Surface: dry grass	
850				R-1	2 4 8	104	4	SM	@2.5' SILTY SAND, loose, light brown, dry, fine sand, 40% fines (field estimate), trace rootlets	
	5			R-2	8 11 14	111	5	SM	@5' SILTY SAND, medium dense, brown, moist, fine sand, 30% fines (field estimate)	
845										
	10			R-3	11 7 13	111	4	SM	@10' SILTY SAND, medium dense, light gray brown, moist, fine sand, 30% fines (field estimate), trace fine gravel	CO
840										
	15			R-4	11 17 19	93	9	ML	@15' SILT, very stiff, gray, moist, FeO2 staining, homongenous	AL
835										
	20			S-5	5 7 9			CL ML	@20' CLAY, very stiff, gray, moist, FeO2 staining @20.5' SILT, gray, moist, FeO2 staining	
830										
	25			S-6	5 5 11			ML		
825										
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

* * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * *

GEOTECHNICAL BORING LOG LB-3

Project No.	10557.004	**Date Drilled**	12-13-13
Project	Coastal Commerce Chino	**Logged By**	JMD
Drilling Co.	2R Drilling	**Hole Diameter**	9.5"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	852'
Location	See Figure 2	**Sampled By**	JMD

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	30			S-7	12 14 16			ML	@30' SANDY SILT, very stiff, gray, moist	
820								SM	@31.1' SILTY SAND, gray, dry, fine sand, 20% fines (field estimate),	
	35			S-8	17 13 8			SP	@35' SAND, medium dense, reddish brown, medium to coarse sand	
815								CL	@36.3' CLAY, olive brown, moist, large amount of FeO2 staining	
	40			S-9	9 14 26			ML	@40' SANDY SILT, hard, olive brown, moist, large amount of FeO2 staining	
810										
	45			S-10	6 8 9			ML	@45' SILT, very stiff, light brown, large amount of FeO2 staining, homogenous	
805										
	50			S-11	14 14 20			SP	@50' SAND, dense, light gray brown, dry to moist, fine sand, trace fines	
								ML	@51.2' SILT, light brown, large amount of FeO2 staining	
800									Total depth of 51.5' No groundwater encountered Backfilled with soil cuttings	
	55									
795										
	60									

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

GEOTECHNICAL BORING LOG LB-4

Project No.	10557.004	**Date Drilled**	12-13-13
Project	Coastal Commerce Chino	**Logged By**	JMD
Drilling Co.	2R Drilling	**Hole Diameter**	9.5"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	850'
Location	See Figure 2	**Sampled By**	JMD

This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
850	0			BULK					@Surface: dirt	MD, CR
				R-1	5 8 8			SM	@2.5' SILTY SAND, loose, light brown to brown, dry, 20% fines (field estimate), with some gravel, 2" maximum gravel size	
845	5			R-2	6 11 10	121	2	SP-SM	@5' SAND to SILTY SAND, medium dense, light brown, dry to moist, 10% fines (field estimate), with some gravel, 1.25" maximum gravel size	-200
840	10			R-3	20 30 40			SP	@10' SAND, dense, gray, dry to moist, trace fines, some fine and medium sand, some gravel, 1.5" maximum gravel size	
835	15			S-4	6 7 7			SM	@15' SILTY SAND, medium dense, olive, moist, fine sand, 40% fines (field estimate), trace coarse sand	
830	20			R-5	10 19 26	100	19	CL	@20' CLAY, very stiff, olive, moist, with FeO2 staining. homogenous	
									Total depth of 21.5' No groundwater encountered Backfilled with soil cuttings	
825	25									
820	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

*** This log is a part of a report by Leighton and should not be used as a stand-alone document. ***

GEOTECHNICAL BORING LOG LB-5

Project No.	10557.004	**Date Drilled**	12-13-13
Project	Coastal Commerce Chino	**Logged By**	JMD
Drilling Co.	2R Drilling	**Hole Diameter**	10"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	**Ground Elevation**	848'
Location	See Figure 2	**Sampled By**	JMD

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION *This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.*	Type of Tests
	0			BULK					@Surface: dirt	
845										
	5			R-2	12 28 34	117	3	SP	@5' SAND, dense, gray brown, moist, medium sand, with some gravel, 1" maximum gravel size	
840										
	10			R-3	12 16 18	106	3	SP	@10' SAND, medium dense, gray to reddish brown, moist, medium sand, trace gravel, 2" maximum gravel size	
835										
	15			R-4	17 14 17	105	2	SP	@15' SAND, medium dense, olive, moist, trace fines, trace fine gravel, trace FeO2 staining	
830										
	20			S-5	7 6 6			SM	@20' SILTY SAND, medium dense, olive, dry to moist, fine sand, 40% fines (field estimate), some FeO2 staining	
									Total depth of 21.5' No groundwater encountered Backfilled with soil cuttings	
825										
	25									
820										
	30									

SAMPLE TYPES:
- B BULK SAMPLE
- C CORE SAMPLE
- G GRAB SAMPLE
- R RING SAMPLE
- S SPLIT SPOON SAMPLE
- T TUBE SAMPLE

TYPE OF TESTS:
- -200 % FINES PASSING
- AL ATTERBERG LIMITS
- CN CONSOLIDATION
- CO COLLAPSE
- CR CORROSION
- CU UNDRAINED TRIAXIAL
- DS DIRECT SHEAR
- EI EXPANSION INDEX
- H HYDROMETER
- MD MAXIMUM DENSITY
- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

*** This log is a part of a report by Leighton and should not be used as a stand-alone document. ***

General Test Setup Data of Well Permeameter, from USBR 7300-89 Method.

Project: Coastal Commercial Chino, Project No. 10557.004

	Item	Unit	LB-1		LB-2		LB-3		LB-4		LB-5			
	Exploration #/Location:		*LB-1*		*LB-2*		*LB-3*		*LB-4*		*LB-5*			
	Approx. Test Depth (ft):		6		4		6		6		5			
	Date Tested, start/finish:		12/16/2013		12/16/2013		12/16/2013		12/16/2013		12/16/2013			
	Tested by:		JMD		JMD		JMD		JMD		JMD			
	USCS Soil Type:													
	Weather (start to finish)		Warm, clear											
	Liquid Used/pH:		water from garden hose											
	Well Prep:		straight drill, tamp											
a.	**Diameter of barrel (in.):**		22.5		22.5		22.5		22.5		22.5		22.5	
b.	**No. of Supply barrels:**		1		1		1		1		1		1	
c.	**Measured boring diameter**	inches	9.5		9.5		9.5		9.5		10		13	
d.	Approx Depth to groundwater below GS	feet	200		200		200		200		200		200	
	Depths from string line (or top of ex. pavement):													
f.	to ground surface (=0 if no string line used)	ft/in.	0. ft		0. ft		0. ft		0. ft		0. ft			
g.	**to Bot of Boring** (or top of soil over Bentonite)	ft/in.	**6. ft**	**1. in.**	**4. ft**	**3. in.**	**5. ft**	**7. in.**	**5. ft**	**8. in.**	**4. ft**	**10. in.**		
i	to Top of Sand (bot of float assbly) (dry)	ft/in.	**5. ft**	**10. in.**	**4. ft**	**2. in.**	**5. ft**	**4.5 in.**	**5. ft**	**5. in.**	**4. ft**	**6. in.**		
k	to Top of casing after adding water (negative is above string line)	ft/in.	0. ft	-3. in.			0. ft	-0.75 in.			0. ft	-1. in.		
m	Top of Float assembly Rod, when pushed to bottom	ft/in.		34.75 in.				33.5 in.				14.88 in.		
n	**top of float assembly rod, floating, water level stable**	ft/in.		30.5 in.				25.13 in.				26.5 in.		
p	**Float Assembly (choose one)**		Long body				Long body				Long body			
q	Float Assembly extension (0=none)	inches	12				12				0			
s	free play in float assembly (water level stablized)	inches	2.5				1.25				2.5			
t	Length of float assembly (=lookup p)	inches	23		#N/A		23		#N/A		23		#N/A	
u	Length of float assembly plus extension (=q+t)	inches	35		#N/A		35		#N/A		23		#N/A	
v	**Ht from water surface to top of float rod (=lookup p)**	inches	16.75		#N/A		16.75		#N/A		16.75		#N/A	
w	range of float movement (=lookup p)	inches	6.75		#N/A		6.75		#N/A		6.75		#N/A	
x	Depth to Water Surface (=n+v)		47.3 in.		#N/A in.		41.9 in.		#N/A in.		43.3 in.		#N/A in.	
h	**Depth of water in Well, "h" (=g-x)**		**25.8 in=** 2.15 ft		**#N/A** #N/A		**25.1 in=** 2.09 ft		**#N/A** #N/A		**14.8 in=** 1.23 ft		**#N/A** #N/A	
y	Total Area of barrels (in.^2):		397		397		397		397		397		397	
r	**Well Radius, "r" (=c/2)**		**4.8** in.		**4.8** in.		**4.8** in.		**4.8** in.		**5.0** in.		**6.5** in.	

Results of Well Permeameter Test, from USBR 7300-89 Method.

Project: **Coastal Commercial Chino, Project No. 10557.004**

Exploration #/Location: ***LB-1*** Initial Depth to top of float rod (in.) 30.5

Leighton

Field Data **Calculations**

Date (and comments)	Time	**Water Level in Supply Barrel (in.)**	Depth to top of float rod (when changed)		Water Temp in Barrel (deg F)	DL Interpre-tation? ("Y")	DL -- Head of Water in Barrel (in.)	h, Height of Water in Well (in.)	h/r	Total Elapsed Time (minutes)	Δt (min)	Vol Change (in.^3)	Flow (in^3/min)	q, Flow (in^3/hr)	V (Fig 9)	K20, Coef. Of Permeability at 20 deg C (in./hr)	**Infiltration Rate [flow/surf area] (in./hr) (FS=1)**
Start Date	Start time:		**ft**	**in.**													
12/16/2013	12:52:00 PM																
12/16/13	12:52	29.25			**74**			25.75	5.4	0					0.9		
12/16/13	12:53	28						25.75	5.4	0	1	497	497	29805	0.9	10.01	**14.65**
12/16/13	12:54	27						25.75	5.4	0	1	397	397	23844	0.9	8.00	**11.72**
12/16/13	12:55	26.625						25.75	5.4	0	1	149	149	8942	0.9	3.00	**4.39**
12/16/13	12:57	25.875						25.75	5.4	0	2	298	149	8942	0.9	3.00	**4.39**
12/16/13	13:05	20.25						25.75	5.4	0	8	2235	279	16766	0.9	5.63	**8.24**
12/16/13	13:23	10.75			76			25.75	5.4	0	18	3775	210	12585	0.9	4.11	**6.02**
										0							
12/16/13	13:27	31.125			76			25.75	5.4	0					0.9		
12/16/13	13:49	20.25						25.75	5.4	0	22	4322	196	11787	0.9	3.85	**5.64**
12/16/13	14:01	14.25			77			25.75	5.4	0	12	2384	199	11922	0.9	3.85	**5.63**
										0							
12/16/13	14:06	31.375			77			25.75	5.4	0					0.9		
12/16/13	14:37	18.5			77			25.75	5.4	0	31	5117	165	9903	0.9	3.20	**4.68**
12/16/13	15:07	7.25			77			25.75	5.4	0	30	4471	149	8942	0.9	2.89	**4.22**
12/16/13	15:20	3						25.75	5.4	0	13	1689	130	7795	0.9	2.52	**3.68**
										0							

Results of Well Permeameter Test, from USBR 7300-89 Method.

Project: **Coastal Commercial Chino, Project No. 10557.004**

Exploration #/Location: ***LB-3***

Initial Depth to top of float rod (in.) 25.125

Leighton

Field Data / **Calculations**

Date (and comments)	Time	**Water Level in Supply Barrel (in.)**	Depth to top of float rod (when changed)		Water Temp in Barrel (deg F)	DL Interpre-tation? ("Y")	DL -- Head of Water in Barrel (in.)	h, Height of Water in Well (in.)	h/r	Total Elapsed Time (minutes)	Δt (min)	Vol Change (in.^3)	Flow (in^3/min)	q, Flow (in^3/hr)	Cumulative Vol (gal)	V (Fig 9)	K20, Coef. Of Permeability at 20 deg C (in./hr)	**Infiltration Rate [flow/surf area] (in./hr) (FS=1)**
Start Date	Start time:																	
12/16/2013	10:25:00 AM		ft	in.														
12/16/13	10:25	30.25			**69**			25.125	5.3	0					0	1.0		
12/16/13	11:04	28.375			74			25.125	5.3		39	745	19	1146		0.9	0.40	**0.58**
12/16/13	11:35	27.375			77			25.125	5.3		31	397	13	769		0.9	0.26	**0.37**
12/16/13	12:27	25.75			79			25.125	5.3		52	646	12	745		0.8	0.24	**0.35**
12/16/13	13:09	24.5			81			25.125	5.3		42	497	12	710		0.8	0.23	**0.33**
12/16/13	13:53	23.25			81			25.125	5.3		44	497	11	677		0.8	0.22	**0.31**
12/16/13	14:49	20.75			82			25.125	5.3		56	994	18	1064		0.8	0.34	**0.49**
12/16/13	15:45	19.125			83			25.125	5.3		56	646	12	692		0.8	0.22	**0.31**

Results of Well Permeameter Test, from USBR 7300-89 Method.

Project: **Coastal Commercial Chino, Project No. 10557.004**

Exploration #/Location: ***LB-5***

Initial Depth to top of float rod (in.) 26.5

Leighton

Field Data | **Calculations**

Date (and comments)	Time	**Water Level in Supply Barrel (in.)**	Depth to top of float rod (when changed)		Water Temp in Barrel (deg F)	DL Interpre-tation? ("Y")	DL -- Head of Water in Barrel (in.)	h, Height of Water in Well (in.)	h/r	Total Elapsed Time (minutes)	Δt (min)	Vol Change (in.^3)	Flow (in^3/min)	q, Flow (in^3/hr)	Cumulative Vol (gal)	V (Fig 9)	K20, Coef. Of Permeability at 20 deg C (in./hr)	**Infiltration Rate [flow/surf area] (in./hr) (FS=1)**
Start Date	Start time:																	
12/16/2013	2:25:00 PM		**ft**	**in.**						F	G	H						
12/16/13	14:25	31			**77**			14.75	3.0	0					0	0.9		
12/16/13	14:26	30						14.75	3.0		1	397	397	23844		0.9	16.33	**17.75**
12/16/13	14:27	29.125						14.75	3.0		1	348	348	20864		0.9	14.29	**15.53**
12/16/13	14:28	28.125						14.75	3.0		1	397	397	23844		0.9	16.33	**17.75**
12/16/13	14:29	27.25						14.75	3.0		1	348	348	20864		0.9	14.29	**15.53**
12/16/13	14:30	26.25						14.75	3.0		1	397	397	23844		0.9	16.33	**17.75**
12/16/13	14:32	24.375						14.75	3.0		2	745	373	22354		0.9	15.31	**16.64**
12/16/13	14:42	15.375			77			14.75	3.0		10	3577	358	21460		0.9	14.70	**15.97**
12/16/13	14:53	6						14.75	3.0		11	3726	339	20322		0.9	13.92	**15.13**
12/16/13	15:01	25.125			79			14.75	3.0							0.8		
12/16/13	15:02	24.5						14.75	3.0		1	248	248	14903		0.8	9.96	**10.82**
12/16/13	15:24	7.75			79			14.75	3.0	0	22	6657	303	18154	0	0.8	12.13	**13.18**
12/16/13	15:31	2.375						14.75	3.0	0	7	2136	305	18309	0	0.8	12.23	**13.30**
										0					0			

APPENDIX C

LABORATORY TEST RESULTS

TESTS for SULFATE CONTENT
CHLORIDE CONTENT and pH of SOILS

Leighton

Project Name:	Coastal Commercial Chino	Tested By :	G. Berdy	Date: 12/26/13
Project No. :	10557.004	Data Input By:	J. Ward	Date: 01/03/14

Boring No.	LB-4			
Sample No.	B-4			
Sample Depth (ft)	0-5			
Soil Identification:	Olive brown (SP-SM)g			
Wet Weight of Soil + Container (g)	301.40			
Dry Weight of Soil + Container (g)	299.00			
Weight of Container (g)	64.80			
Moisture Content (%)	1.02			
Weight of Soaked Soil (g)	100.50			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	31			
Crucible No.	28			
Furnace Temperature (°C)	820			
Time In / Time Out	8:50/9:35			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	21.1490			
Wt. of Crucible (g)	21.1467			
Wt. of Residue (g) (A)	0.0023			
PPM of Sulfate (A) x 41150	94.65			
PPM of Sulfate, Dry Weight Basis	**96**			

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	15			
ml of AgNO3 Soln. Used in Titration (C)	1.2			
PPM of Chloride (C -0.2) * 100 * 30 / B	200			
PPM of Chloride, Dry Wt. Basis	**202**			

pH TEST, DOT California Test 532/643

pH Value	**6.94**			
Temperature °C	21.0			

SOIL RESISTIVITY TEST

DOT CA TEST 532 / 643

Leighton

Project Name:	Coastal Commercial Chino	Tested By :	G. Berdy	Date:	12/31/13
Project No. :	10557.004	Data Input By:	J. Ward	Date:	01/03/14
Boring No.:	LB-4	Depth (ft.) :	0-5		
Sample No. :	B-4				

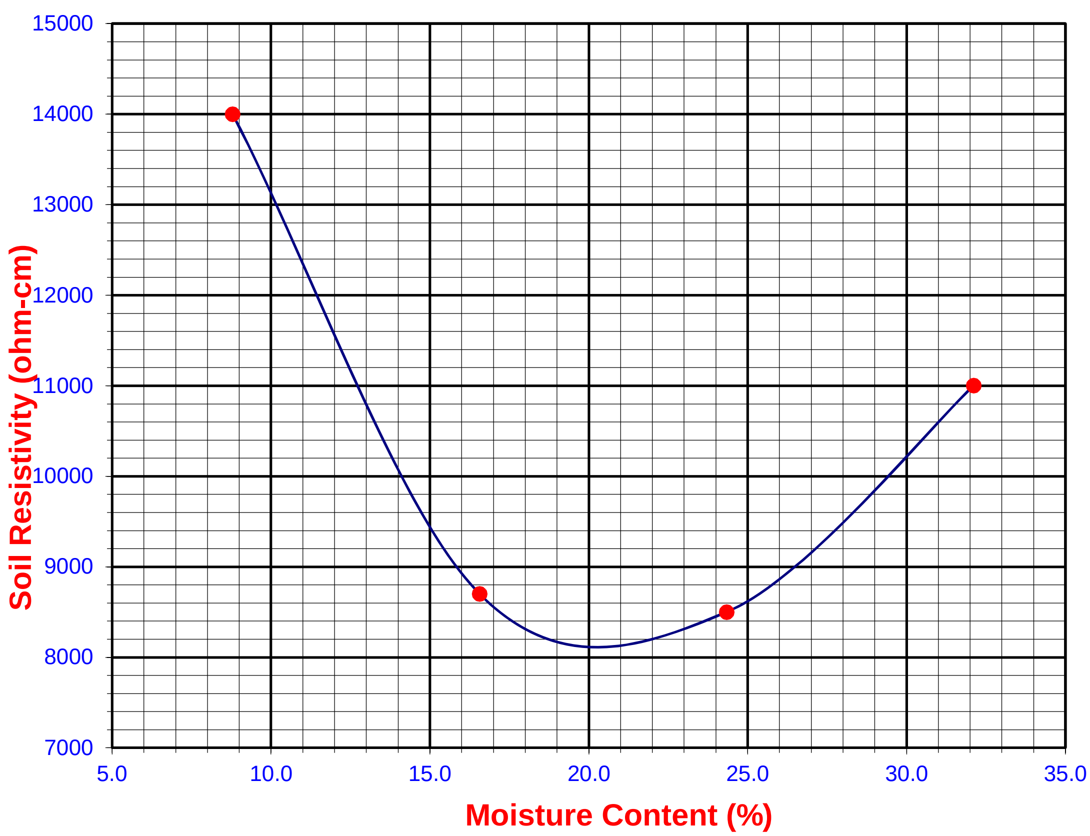
Soil Identification:* Olive brown (SP-SM)g

*California Test 643 requires soil specimens to consist only of portions of samples passing through the No. 8 US Standard Sieve before resistivity testing. Therefore, this test method may not be representative for coarser materials.

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	10	8.80	14000	14000
2	20	16.57	8700	8700
3	30	24.34	8500	8500
4	40	32.11	11000	11000
5				

Moisture Content (%) (MCi)	1.02
Wet Wt. of Soil + Cont. (g)	301.40
Dry Wt. of Soil + Cont. (g)	299.00
Wt. of Container (g)	64.80
Container No.	
Initial Soil Wt. (g) (Wt)	130.00
Box Constant	1.000
MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH: pH	Soil pH: Temp. (°C)
DOT CA Test 532 / 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 532 / 643	
8100	**20.3**	**96**	**202**	**6.94**	**21.0**



TESTS for SULFATE CONTENT CHLORIDE CONTENT and pH of SOILS

Leighton

Project Name:	Coastal Commercial Chino	Tested By :	G. Berdy	Date:	12/26/13
Project No. :	10557.004	Data Input By:	J. Ward	Date:	01/03/14

Boring No.	LB-4			
Sample No.	B-4			
Sample Depth (ft)	0-5			
Soil Identification:	Olive brown (SP-SM)g			
Wet Weight of Soil + Container (g)	301.40			
Dry Weight of Soil + Container (g)	299.00			
Weight of Container (g)	64.80			
Moisture Content (%)	1.02			
Weight of Soaked Soil (g)	100.50			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	31			
Crucible No.	28			
Furnace Temperature (°C)	820			
Time In / Time Out	8:50/9:35			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	21.1490			
Wt. of Crucible (g)	21.1467			
Wt. of Residue (g) (A)	0.0023			
PPM of Sulfate (A) x 41150	94.65			
PPM of Sulfate, Dry Weight Basis	**96**			

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	15			
ml of AgNO3 Soln. Used in Titration (C)	1.2			
PPM of Chloride (C -0.2) * 100 * 30 / B	200			
PPM of Chloride, Dry Wt. Basis	**202**			

pH TEST, DOT California Test 532/643

pH Value	**6.94**			
Temperature °C	21.0			

SOIL RESISTIVITY TEST

DOT CA TEST 532 / 643

Project Name:	Coastal Commercial Chino	Tested By :	G. Berdy
Project No. :	10557.004	Data Input By:	J. Ward
Boring No.:	LB-4	Depth (ft.) :	0-5
Sample No. :	B-4		

Date: 12/31/13

Date: 01/03/14

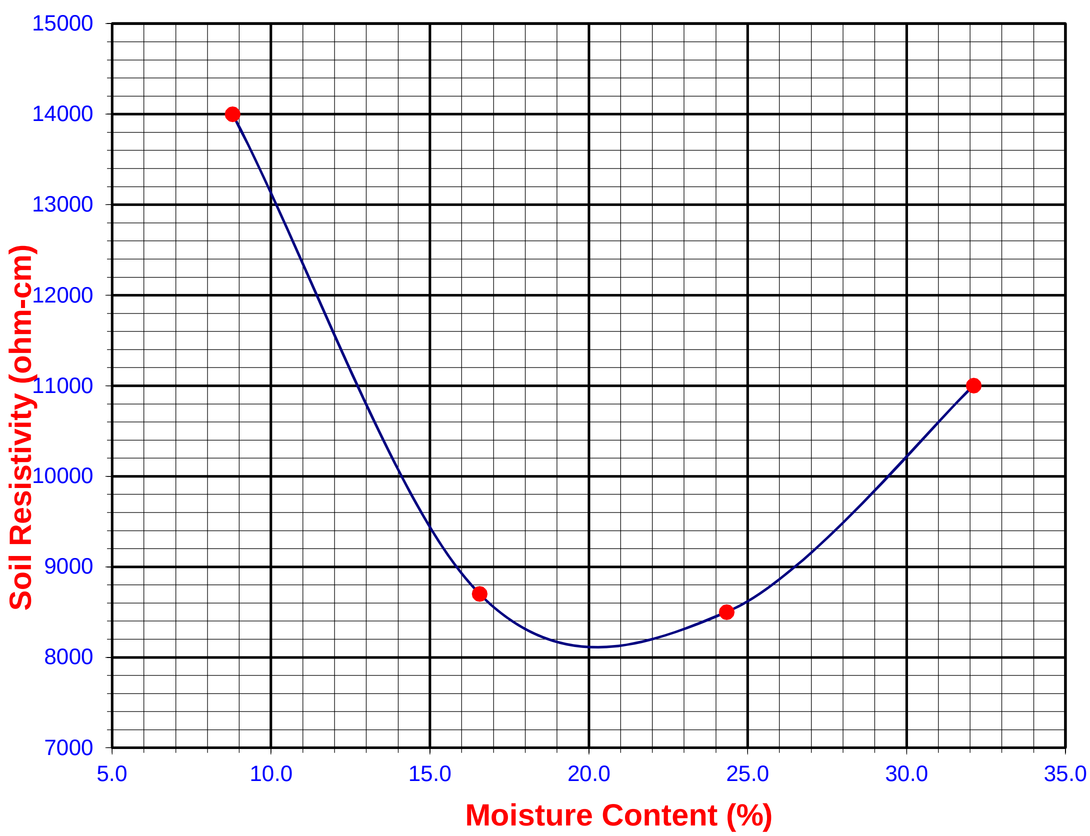
Soil Identification:* Olive brown (SP-SM)g

*California Test 643 requires soil specimens to consist only of portions of samples passing through the No. 8 US Standard Sieve before resistivity testing. Therefore, this test method may not be representative for coarser materials.

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	10	8.80	14000	14000
2	20	16.57	8700	8700
3	30	24.34	8500	8500
4	40	32.11	11000	11000
5				

Moisture Content (%) (MCi)	1.02
Wet Wt. of Soil + Cont. (g)	301.40
Dry Wt. of Soil + Cont. (g)	299.00
Wt. of Container (g)	64.80
Container No.	
Initial Soil Wt. (g) (Wt)	130.00
Box Constant	1.000
MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 532 / 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 532 / 643	
8100	**20.3**	**96**	**202**	**6.94**	**21.0**



MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Leighton

Project Name:	Coastal Commercial Chino	Tested By:	O. Figueroa	Date:	12/27/13
Project No.:	10557.004	Input By:	J. Ward	Date:	01/03/14
Boring No.:	LB-4	Depth (ft.):	0-5		
Sample No.:	B-4				
Soil Identification:	Olive brown poorly-graded sand with silt and gravel (SP-SM)g				

Preparation Method:	X	Moist
		Dry
Compaction Method	X	Mechanical Ram
		Manual Ram

Scalp Fraction (%)	
#3/4	
#3/8	
#4	15.4

Rammer Weight (lb.) = 10.0

Height of Drop (in.) = 18.0

Mold Volume (ft^3) 0.03310

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3773.0	3834.0	3898.0	3899.0		
Weight of Mold (g)	1859.0	1859.0	1859.0	1859.0		
Net Weight of Soil (g)	1914.0	1975.0	2039.0	2040.0		
Wet Weight of Soil + Cont. (g)	475.80	450.50	423.80	506.90		
Dry Weight of Soil + Cont. (g)	462.20	428.90	395.80	463.60		
Weight of Container (g)	48.50	51.30	54.80	52.70		
Moisture Content (%)	3.29	5.72	8.21	10.54		
Wet Density (pcf)	127.5	131.5	135.8	135.9		
Dry Density (pcf)	123.4	124.4	125.5	122.9		

Maximum Dry Density (pcf) 125.5 **Optimum Moisture Content (%)** 8.0

Corrected Dry Density (pcf) 130.5 **Corrected Moisture Content (%)** 7.0

☒ **Procedure A**
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
May be used if +#4 is 20% or less

☐ **Procedure B**
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
Use if +#4 is >20% and +3/8 in. is 20% or less

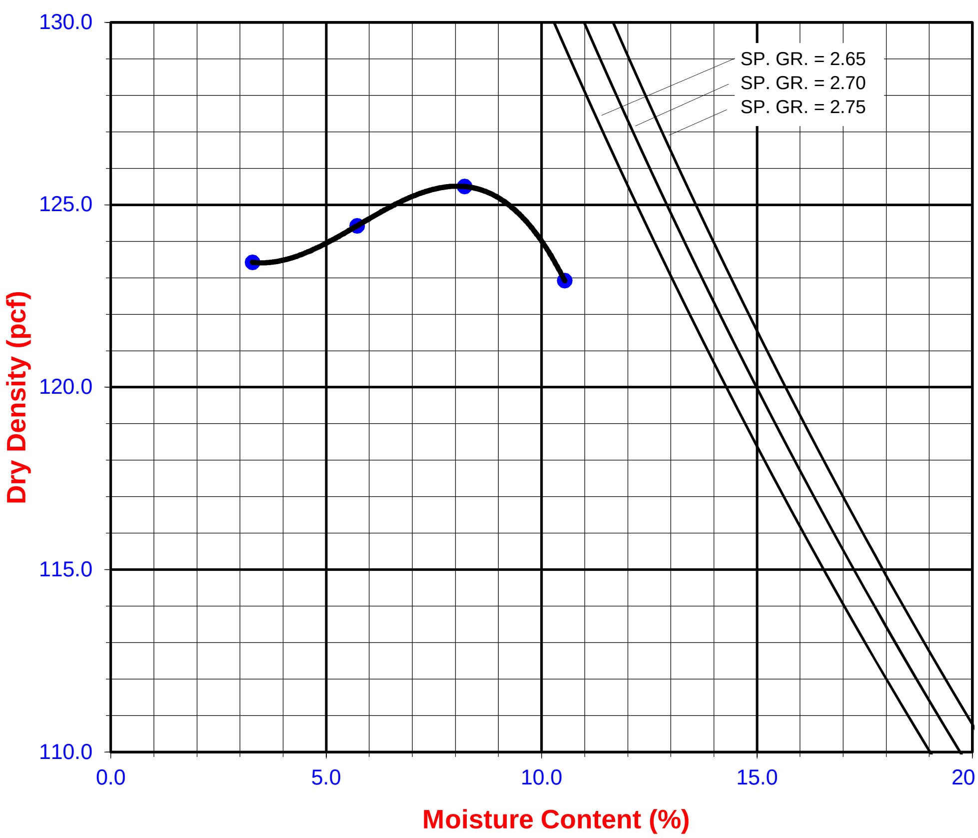
☐ **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL,PL,PI



ATTERBERG LIMITS

ASTM D 4318

Leighton

Project Name: Coastal Commercial Chino
Tested By: G. Bathala
Date: 12/30/13

Project No. : 10557.004
Input By: J. Ward
Date: 01/03/13

Boring No.: LB-3
Checked By: J. Ward

Sample No.: R-4
Depth (ft.) 15.0

Soil Identification: Olive sandy silt s(ML)

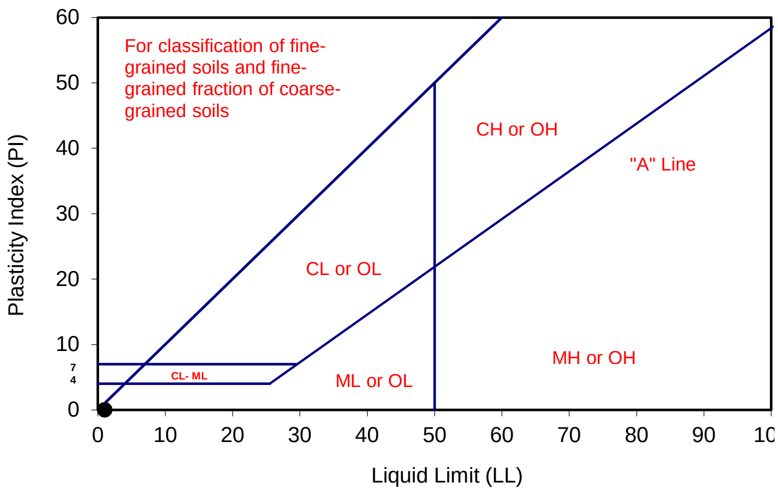
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			5			
Wet Wt. of Soil + Cont. (g)	18.01	16.94	35.69	**Cannot get more than 5 blows:**		
Dry Wt. of Soil + Cont. (g)	17.10	16.27	30.12	**NonPlastic**		
Wt. of Container (g)	13.51	13.61	13.51			
Moisture Content (%) [Wn]	25.35	25.19	33.53			

Liquid Limit	**NP**
Plastic Limit	**25**
Plasticity Index	**NP**
Classification	**NP**

PI at "A" - Line = 0.73(LL-20) =

One - Point Liquid Limit Calculation

$LL = Wn(N/25)^{0.121}$



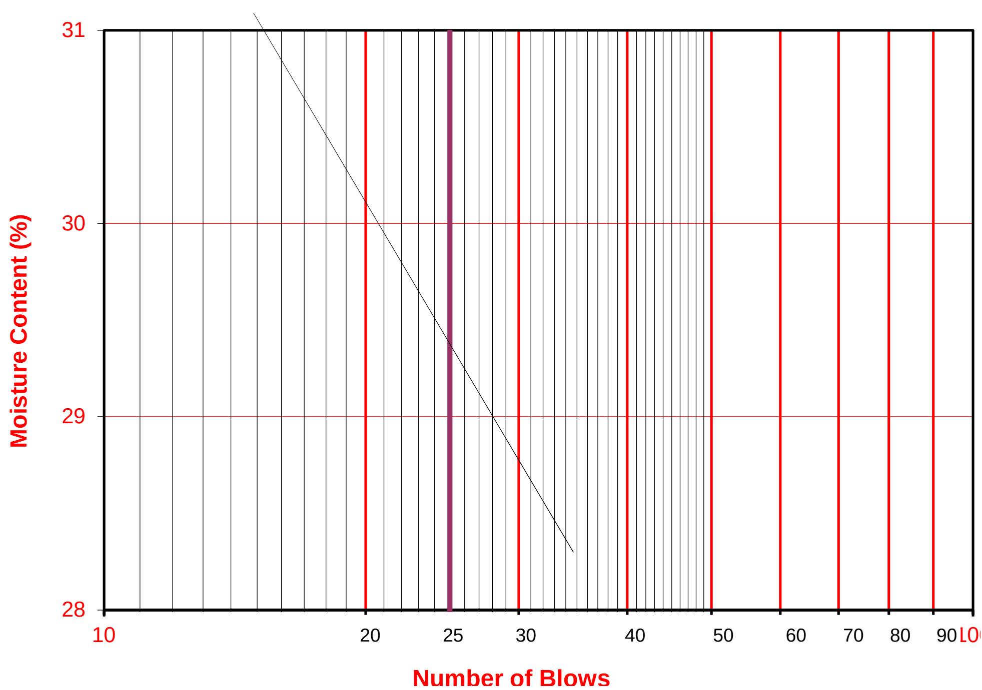
PROCEDURES USED

☐ Wet Preparation
Multipoint - Wet

X Dry Preparation
Multipoint - Dry

X Procedure A
Multipoint Test

☐ Procedure B
One-point Test



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ONE-DIMENSIONAL SWELL OR SETTLEMENT POTENTIAL OF COHESIVE SOILS
(ASTM D 4546)

Project Name: Coastal Commercial Chino
Project No.: 10557.004
Boring No.: LB-3
Sample No.: R-3
Sample Description: Olive silty sand (SM)

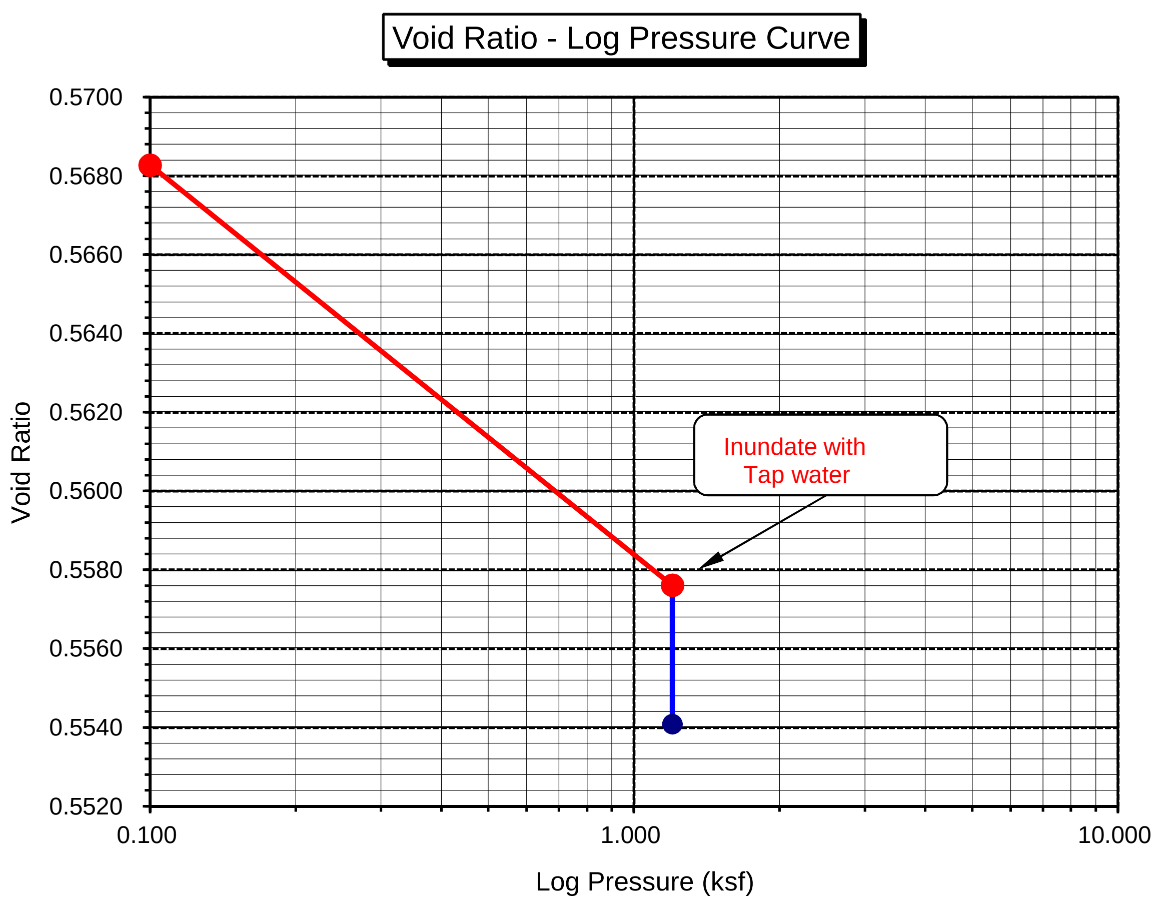
Tested By: G. Bathala   Date: 12/20/13
Checked By: J. Ward   Date: 01/03/14
Sample Type: Ring
Depth (ft.) 10.0

Initial Dry Density (pcf):	107.5
Initial Moisture (%):	3.61
Initial Length (in.):	1.0000
Initial Dial Reading:	0.3063
Diameter(in):	2.416

Final Dry Density (pcf):	108.5
Final Moisture (%) :	17.2
Initial Void Ratio:	0.5683
Specific Gravity(assumed):	2.70
Initial Saturation (%)	17.2

Pressure (p) (ksf)	Final Reading (in)	Apparent Thickness (in)	Load Compliance (%)	Swell (+) Settlement (-) % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.100	0.3063	1.0000	0.00	0.00	0.5683	0.00
1.200	0.2983	0.9920	0.12	-0.80	0.5576	-0.68
H2O	0.2961	0.9898	0.12	-1.03	0.5541	-0.91

Percent Swell (+) / Settlement (-) After Inundation = -0.23



Boring No.	LB-1	LB-4						
Sample No.	R-4	R-2						
Depth (ft.)	15.0	5.0						
Sample Type	Ring	Ring						
Soil Identification	Brown poorly-graded sand with silt and gravel (SP-SM)g	Olive brown sandy silt s(ML)						
Moisture Correction								
Wet Weight of Soil + Container (g)	0.0	0.0						
Dry Weight of Soil + Container (g)	0.0	0.0						
Weight of Container (g)	1.0	1.0						
Moisture Content (%)	0.00	0.00						
Sample Dry Weight Determination								
Weight of Sample + Container (g)	822.7	915.4						
Weight of Container (g)	250.0	252.4						
Weight of Dry Sample (g)	572.7	663.0						
Container No.:								
After Wash								
Method (A or B)	B	B						
Dry Weight of Sample + Cont. (g)	782.9	519.1						
Weight of Container (g)	250.0	252.4						
Dry Weight of Sample (g)	532.9	266.7						
% Passing No. 200 Sieve	**6.9**	**59.8**						
% Retained No. 200 Sieve	93.1	40.2						

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PERCENT PASSING
No. 200 SIEVE
ASTM D 1140

Project Name:	Coastal Commercial Chino
Project No.:	10557.004
Client Name:	L&A/Rancho Cucamonga
Tested By:	S. Felter
Date:	12/23/13

APPENDIX D

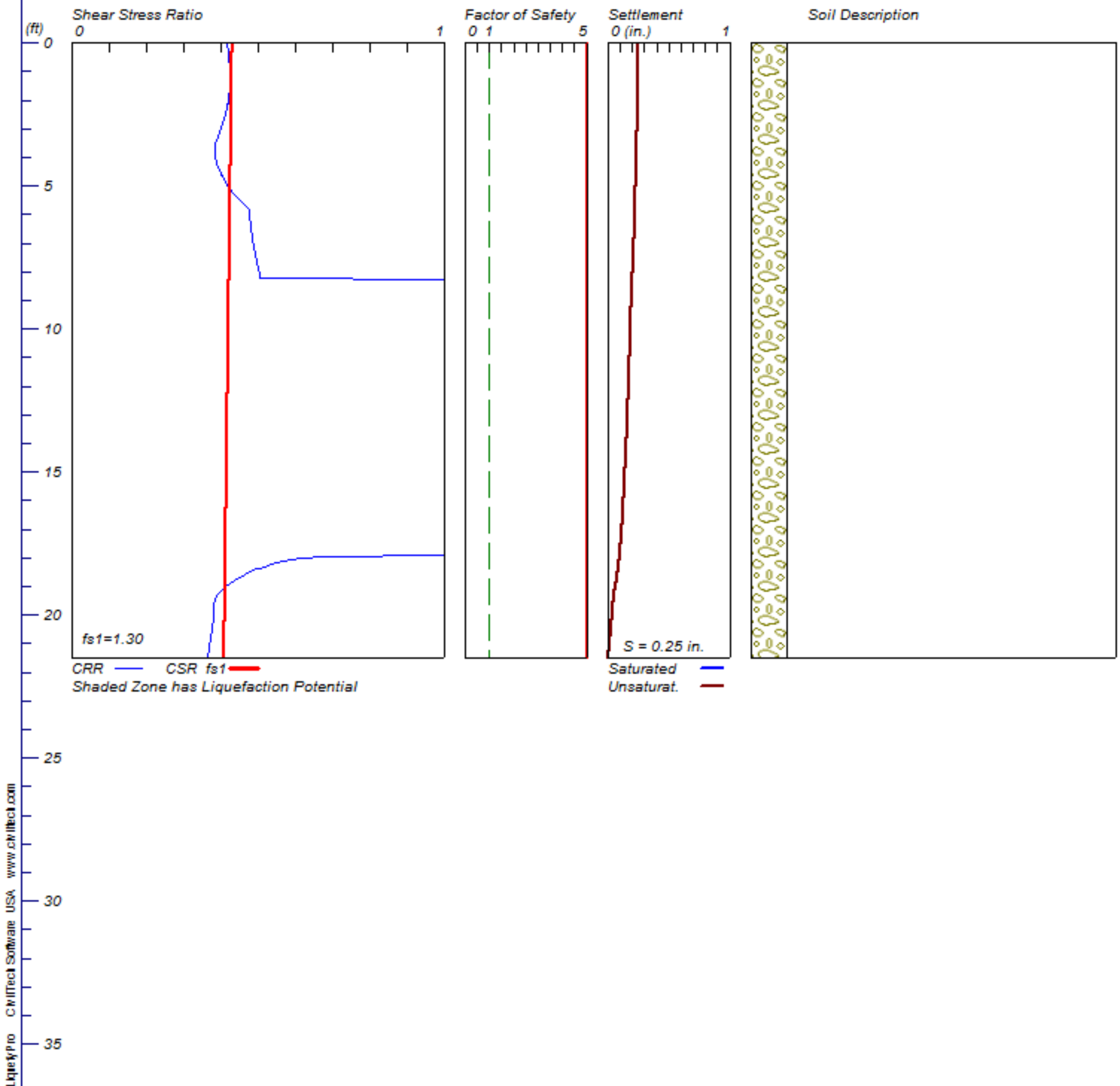
SUMMARY OF SECONDARY SEISMIC HAZARD ANALYSIS

SEISMIC SETTLEMENT ANALYSIS

Coastal Commerce Chino

Hole No.=LB-1 Water Depth=100 ft Surface Elev.=849

Magnitude=6.57
Acceleration=0.51g

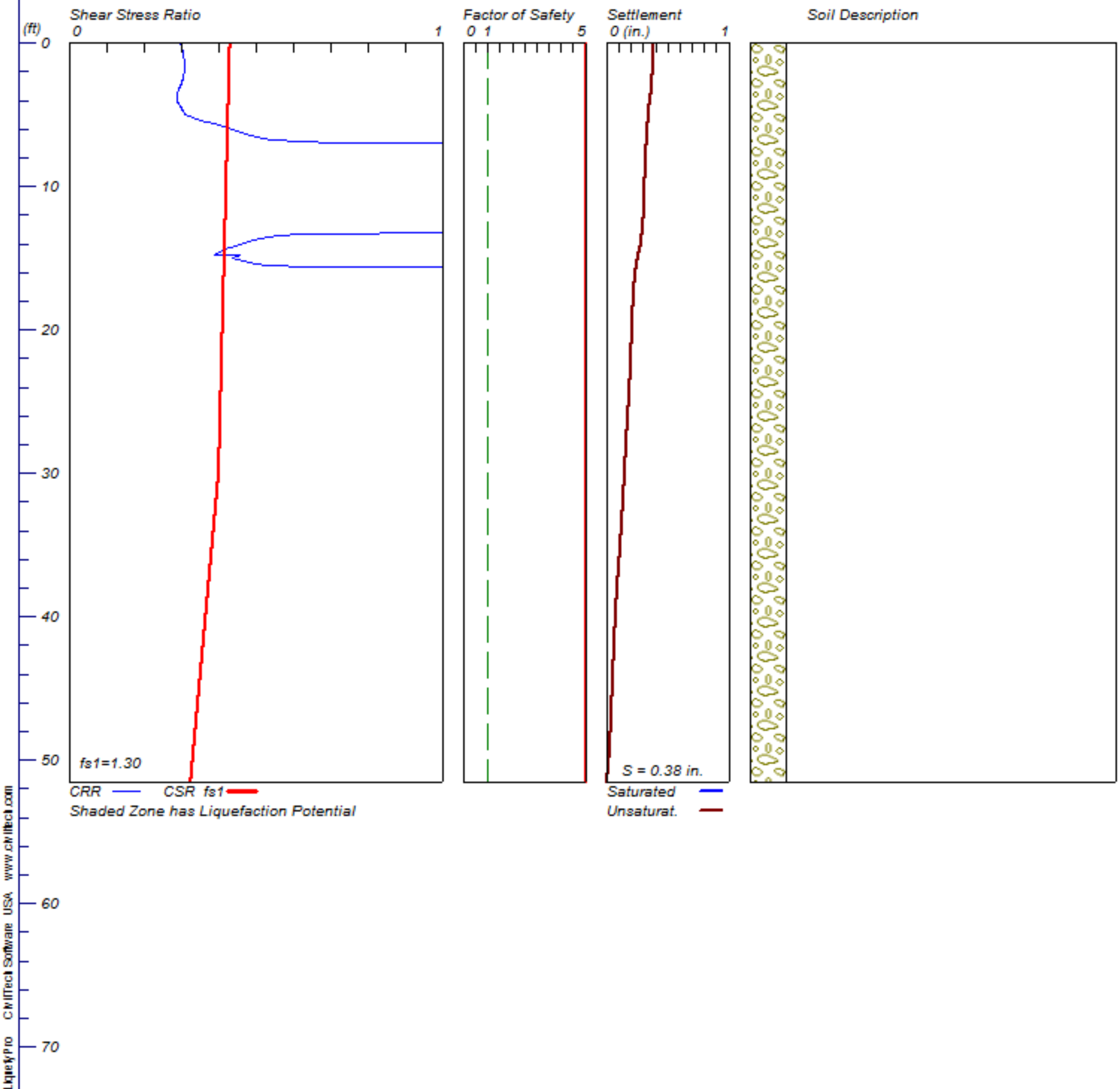


SEISMIC SETTLEMENT ANALYSIS

Coastal Commerce Chino

Hole No.=LB-2 Water Depth=100 ft Surface Elev.=844

Magnitude=6.57
Acceleration=0.51g

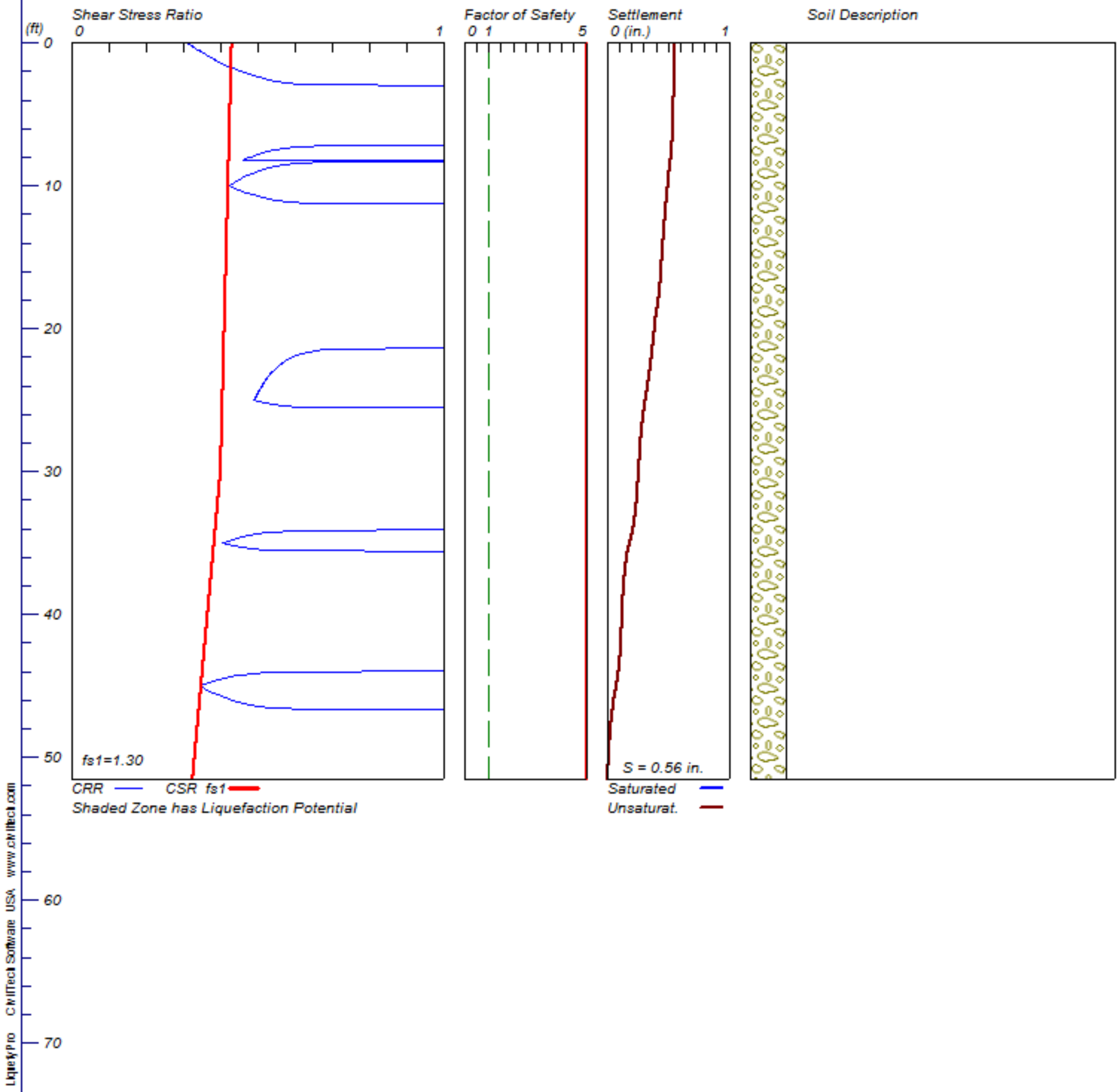


SEISMIC SETTLEMENT ANALYSIS

Coastal Commerce Chino

Hole No.=LB-3   Water Depth=100 ft   Surface Elev.=852

Magnitude=6.57
Acceleration=0.51g

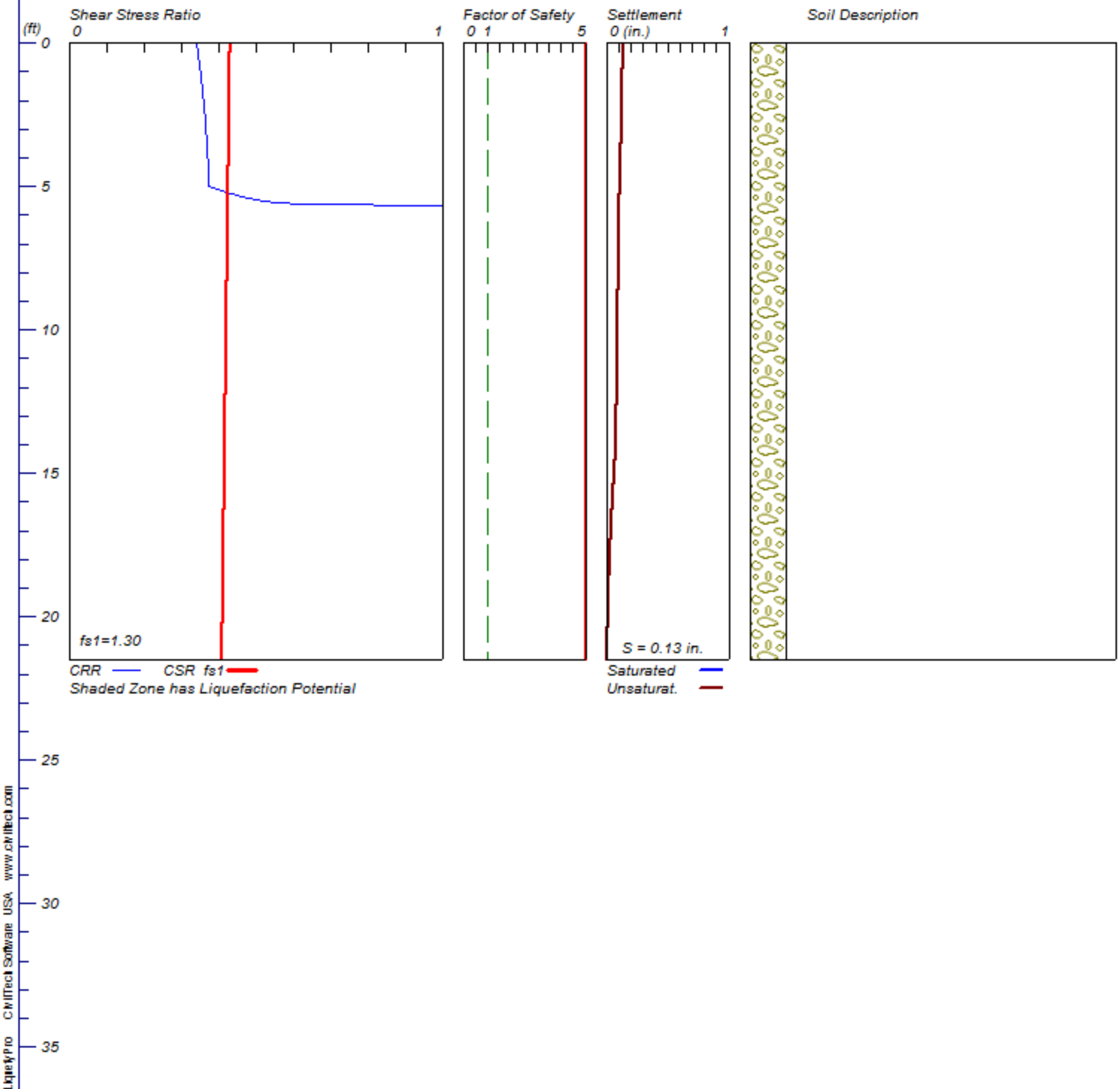


SEISMIC SETTLEMENT ANALYSIS

Coastal Commerce Chino

Hole No.=LB-4 Water Depth=100 ft Surface Elev.=850

Magnitude=6.57
Acceleration=0.51g

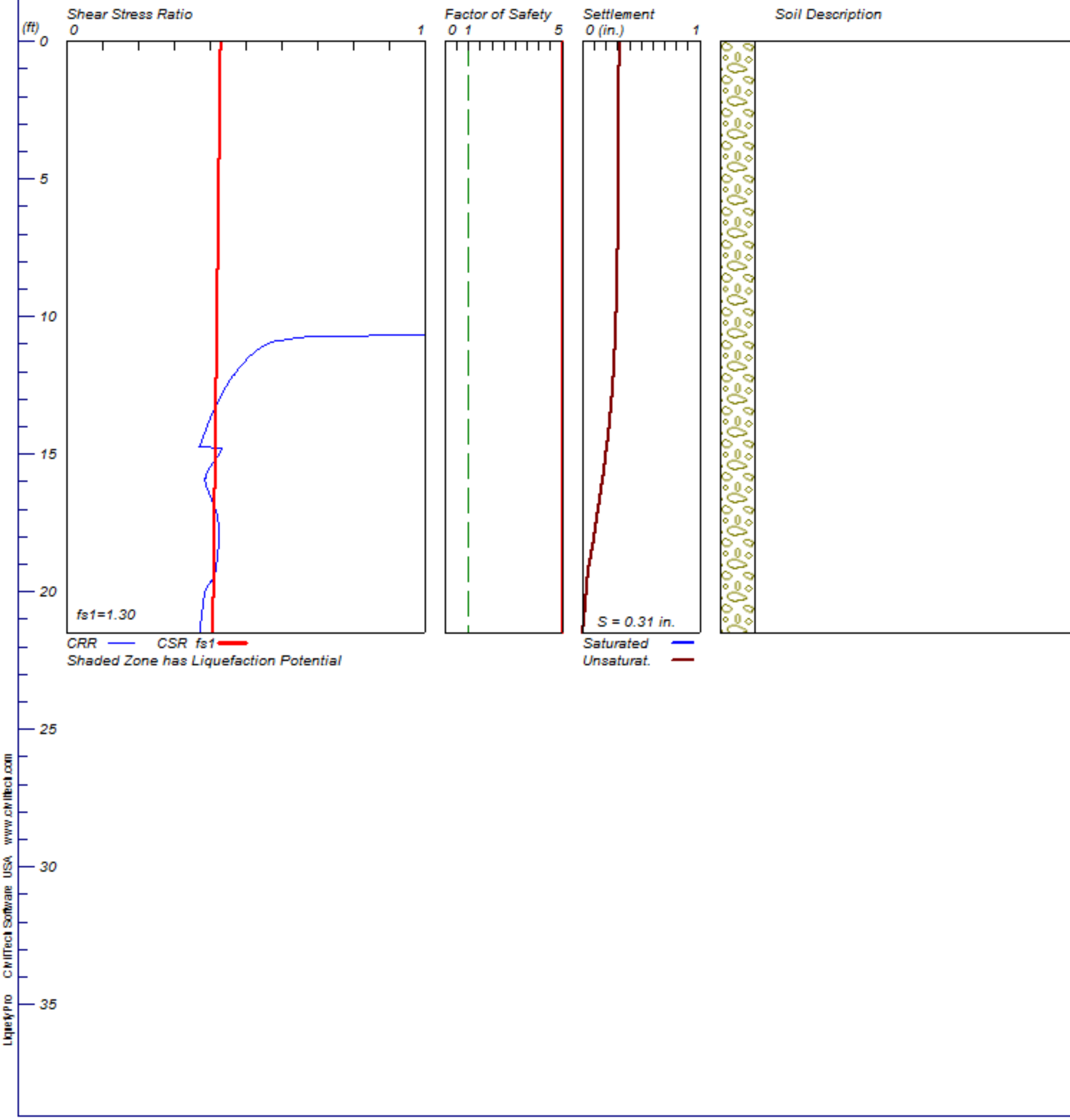


SEISMIC SETTLEMENT ANALYSIS

Coastal Commerce Chino

Hole No.=LB-5   Water Depth=100 ft   Surface Elev.=848

Magnitude=6.57
Acceleration=0.51g



APPENDIX E

GENERAL EARTHWORK AND GRADING SPECIFICATIONS

Leighton

LEIGHTON AND ASSOCIATES, INC.

GENERAL EARTHWORK AND GRADING SPECIFICATIONS FOR ROUGH GRADING

Table of Contents

1.0 General

1.1 Intent: These General Earthwork and Grading Specifications are for the grading and earthwork shown on the approved grading plan(s) and/or indicated in the geotechnical report(s). These Specifications are a part of the recommendations contained in the geotechnical report(s). In case of conflict, the specific recommendations in the geotechnical report shall supersede these more general Specifications. Observations of the earthwork by the project Geotechnical Consultant during the course of grading may result in new or revised recommendations that could supersede these specifications or the recommendations in the geotechnical report(s).

1.2 The Geotechnical Consultant of Record: Prior to commencement of work, the owner shall employ the Geotechnical Consultant of Record (Geotechnical Consultant). The Geotechnical Consultants shall be responsible for reviewing the approved geotechnical report(s) and accepting the adequacy of the preliminary geotechnical findings, conclusions, and recommendations prior to the commencement of the grading.

Prior to commencement of grading, the Geotechnical Consultant shall review the "work plan" prepared by the Earthwork Contractor (Contractor) and schedule sufficient personnel to perform the appropriate level of observation, mapping, and compaction testing.

During the grading and earthwork operations, the Geotechnical Consultant shall observe, map, and document the subsurface exposures to verify the geotechnical design assumptions. If the observed conditions are found to be significantly different than the interpreted assumptions during the design phase, the Geotechnical Consultant shall inform the owner, recommend appropriate changes in design to accommodate the observed conditions, and notify the review agency where required. Subsurface areas to be geotechnically observed, mapped, elevations recorded, and/or tested include natural ground after it has been cleared for receiving fill but before fill is placed, bottoms of all "remedial removal" areas, all key bottoms, and benches made on sloping ground to receive fill.

The Geotechnical Consultant shall observe the moisture-conditioning and processing of the subgrade and fill materials and perform relative compaction testing of fill to determine the attained level of compaction. The Geotechnical Consultant shall provide the test results to the owner and the Contractor on a routine and frequent basis.

1.3 <u>The Earthwork Contractor</u>: The Earthwork Contractor (Contractor) shall be qualified, experienced, and knowledgeable in earthwork logistics, preparation and processing of ground to receive fill, moisture-conditioning and processing of fill, and compacting fill. The Contractor shall review and accept the plans, geotechnical report(s), and these Specifications prior to commencement of grading. The

Contractor shall be solely responsible for performing the grading in accordance with the plans and specifications.

The Contractor shall prepare and submit to the owner and the Geotechnical Consultant a work plan that indicates the sequence of earthwork grading, the number of "spreads" of work and the estimated quantities of daily earthwork contemplated for the site prior to commencement of grading. The Contractor shall inform the owner and the Geotechnical Consultant of changes in work schedules and updates to the work plan at least 24 hours in advance of such changes so that appropriate observations and tests can be planned and accomplished. The Contractor shall not assume that the Geotechnical Consultant is aware of all grading operations.

The Contractor shall have the sole responsibility to provide adequate equipment and methods to accomplish the earthwork in accordance with the applicable grading codes and agency ordinances, these Specifications, and the recommendations in the approved geotechnical report(s) and grading plan(s). If, in the opinion of the Geotechnical Consultant, unsatisfactory conditions, such as unsuitable soil, improper moisture condition, inadequate compaction, insufficient buttress key size, adverse weather, etc., are resulting in a quality of work less than required in these specifications, the Geotechnical Consultant shall reject the work and may recommend to the owner that construction be stopped until the conditions are rectified.

2.0 <u>Preparation of Areas to be Filled</u>

2.1 <u>Clearing and Grubbing</u>: Vegetation, such as brush, grass, roots, and other deleterious material shall be sufficiently removed and properly disposed of in a method acceptable to the owner, governing agencies, and the Geotechnical Consultant.

The Geotechnical Consultant shall evaluate the extent of these removals depending on specific site conditions. Earth fill material shall not contain more than 1 percent of organic materials (by volume). No fill lift shall contain more than 5 percent of organic matter. Nesting of the organic materials shall not be allowed.

If potentially hazardous materials are encountered, the Contractor shall stop work in the affected area, and a hazardous material specialist shall be informed immediately for proper evaluation and handling of these materials prior to continuing to work in that area.

As presently defined by the State of California, most refined petroleum products (gasoline, diesel fuel, motor oil, grease, coolant, etc.) have chemical constituents that are considered to be hazardous waste. As such, the indiscriminate dumping or spillage of these fluids onto the ground may constitute a misdemeanor, punishable by fines and/or imprisonment, and shall not be allowed.

2.2 Processing: Existing ground that has been declared satisfactory for support of fill by the Geotechnical Consultant shall be scarified to a minimum depth of 6 inches. Existing ground that is not satisfactory shall be overexcavated as specified in the following section. Scarification shall continue until soils are broken down and free of large clay lumps or clods and the working surface is reasonably uniform, flat, and free of uneven features that would inhibit uniform compaction.

2.3 Overexcavation: In addition to removals and overexcavations recommended in the approved geotechnical report(s) and the grading plan, soft, loose, dry, saturated, spongy, organic-rich, highly fractured or otherwise unsuitable ground shall be overexcavated to competent ground as evaluated by the Geotechnical Consultant during grading.

2.4 Benching: Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical units), the ground shall be stepped or benched. Please see the Standard Details for a graphic illustration. The lowest bench or key shall be a minimum of 15 feet wide and at least 2 feet deep, into competent material as evaluated by the Geotechnical Consultant. Other benches shall be excavated a minimum height of 4 feet into competent material or as otherwise recommended by the Geotechnical Consultant. Fill placed on ground sloping flatter than 5:1 shall also be benched or otherwise overexcavated to provide a flat subgrade for the fill.

2.5 Evaluation/Acceptance of Fill Areas: All areas to receive fill, including removal and processed areas, key bottoms, and benches, shall be observed, mapped, elevations recorded, and/or tested prior to being accepted by the Geotechnical Consultant as suitable to receive fill. The Contractor shall obtain a written acceptance from the Geotechnical Consultant prior to fill placement. A licensed surveyor shall provide the survey control for determining elevations of processed areas, keys, and benches.

3.0 Fill Material

3.1 General: Material to be used as fill shall be essentially free of organic matter and other deleterious substances evaluated and accepted by the Geotechnical Consultant prior to placement. Soils of poor quality, such as those with unacceptable gradation, high expansion potential, or low strength shall be placed in areas acceptable to the Geotechnical Consultant or mixed with other soils to achieve satisfactory fill material.

3.2 Oversize: Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 8 inches, shall not be buried or placed in fill unless location, materials, and placement methods are specifically accepted by the Geotechnical Consultant. Placement operations shall be such that nesting of oversized material does not occur and such that oversize material is completely surrounded by compacted or densified fill. Oversize material shall not be placed within 10 vertical feet of finish grade or within 2 feet of future utilities or underground construction.

3.3 Import: If importing of fill material is required for grading, proposed import material shall meet the requirements of Section 3.1. The potential import source shall be given to the Geotechnical Consultant at least 48 hours (2 working days) before importing begins so that its suitability can be determined and appropriate tests performed.

4.0 Fill Placement and Compaction

4.1 Fill Layers: Approved fill material shall be placed in areas prepared to receive fill (per Section 3.0) in near-horizontal layers not exceeding 8 inches in loose thickness. The Geotechnical Consultant may accept thicker layers if testing indicates the grading procedures can adequately compact the thicker layers. Each layer shall be spread evenly and mixed thoroughly to attain relative uniformity of material and moisture throughout.

4.2 Fill Moisture Conditioning: Fill soils shall be watered, dried back, blended, and/or mixed, as necessary to attain a relatively uniform moisture content at or slightly over optimum. Maximum density and optimum soil moisture content tests shall be performed in accordance with the American Society of Testing and Materials (ASTM Test Method D1557-91).

4.3 Compaction of Fill: After each layer has been moisture-conditioned, mixed, and evenly spread, it shall be uniformly compacted to not less than 90 percent of maximum dry density (ASTM Test Method D1557-91). Compaction equipment shall be adequately sized and be either specifically designed for soil compaction or of proven reliability to efficiently achieve the specified level of compaction with uniformity.

4.4 Compaction of Fill Slopes: In addition to normal compaction procedures specified above, compaction of slopes shall be accomplished by backrolling of slopes with sheepsfoot rollers at increments of 3 to 4 feet in fill elevation, or by other methods producing satisfactory results acceptable to the Geotechnical Consultant. Upon completion of grading, relative compaction of the fill, out to the slope face, shall be at least 90 percent of maximum density per ASTM Test Method D1557-91.

4.5 Compaction Testing: Field tests for moisture content and relative compaction of the fill soils shall be performed by the Geotechnical Consultant. Location and frequency of tests shall be at the Consultant's discretion based on field conditions encountered. Compaction test locations will not necessarily be selected on a random basis. Test locations shall be selected to verify adequacy of compaction levels in areas that are judged to be prone to inadequate compaction (such as close to slope faces and at the fill/bedrock benches).

4.6 Frequency of Compaction Testing: Tests shall be taken at intervals not exceeding 2 feet in vertical rise and/or 1,000 cubic yards of compacted fill soils embankment. In addition, as a guideline, at least one test shall be taken on slope faces for each 5,000 square feet of slope face and/or each 10 feet of vertical height of slope. The Contractor shall assure that fill construction is such that the testing schedule can be accomplished by the Geotechnical Consultant. The Contractor shall stop or slow down the earthwork construction if these minimum standards are not met.

4.7 Compaction Test Locations: The Geotechnical Consultant shall document the approximate elevation and horizontal coordinates of each test location. The Contractor shall coordinate with the project surveyor to assure that sufficient grade stakes are established so that the Geotechnical Consultant can determine the test locations with sufficient accuracy. At a minimum, two grade stakes within a horizontal distance of 100 feet and vertically less than 5 feet apart from potential test locations shall be provided.

5.0 Subdrain Installation

Subdrain systems shall be installed in accordance with the approved geotechnical report(s), the grading plan, and the Standard Details. The Geotechnical Consultant may recommend additional subdrains and/or changes in subdrain extent, location, grade, or material depending on conditions encountered during grading. All subdrains shall be surveyed by a land surveyor/civil engineer for line and grade after installation and prior to burial. Sufficient time should be allowed by the Contractor for these surveys.

6.0 Excavation

Excavations, as well as over-excavation for remedial purposes, shall be evaluated by the Geotechnical Consultant during grading. Remedial removal depths shown on geotechnical plans are estimates only. The actual extent of removal shall be determined by the Geotechnical Consultant based on the field evaluation of exposed conditions during grading. Where fill-over-cut slopes are to be graded, the cut portion of the slope shall be made, evaluated, and accepted by the Geotechnical Consultant prior to placement of materials for construction of the fill portion of the slope, unless otherwise recommended by the Geotechnical Consultant.

7.0 Trench Backfills

7.1 Safety: The Contractor shall follow all OHSA and Cal/OSHA requirements for safety of trench excavations.

7.2 Bedding and Backfill: All bedding and backfill of utility trenches shall be done in accordance with the applicable provisions of Standard Specifications of Public Works Construction. Bedding material shall have a Sand Equivalent greater than 30 (SE>30). The bedding shall be placed to 1 foot over the top of the conduit and densified by jetting. Backfill shall be placed and densified to a minimum of 90 percent of maximum from 1 foot above the top of the conduit to the surface.

The Geotechnical Consultant shall test the trench backfill for relative compaction. At least one test should be made for every 300 feet of trench and 2 feet of fill.

7.3 Lift Thickness: Lift thickness of trench backfill shall not exceed those allowed in the Standard Specifications of Public Works Construction unless the Contractor can demonstrate to the Geotechnical Consultant that the fill lift can be compacted to the minimum relative compaction by his alternative equipment and method.

7.4 Observation and Testing: The jetting of the bedding around the conduits shall be observed by the Geotechnical Consultant.