GEOTECHNICAL EXPLORATION REPORT
PROPOSED RESIDENTIAL DEVELOPMENT
2880 MESA VERDE DRIVE EAST
COSTA MESA, CALIFORNIA

Prepared for:

Pinnacle Residential

20 Enterprise, Suite 320
Aliso Viejo, California 92656

Project No. 10646-001

April 2, 2014

Leighton and Associates, Inc.
A LEIGHTON GROUP COMPANY

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20 Enterprise, Suite 320
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Attention: Mr. David Kinnet, Principal

Subject: Geotechnical Exploration Report
Proposed Residential Development
2880 Mesa Verde Drive East
Costa Mesa, California

Per your request and authorization, Leighton and Associates, Inc. (Leighton) is pleased to present this geotechnical exploration report for the subject redevelopment project. Based on the information provided by you, Leighton understands the proposed residential development for the site will include grading to create level pads for construction of approximately 10 single-family residences two stories in height, with associated access drives, utilities, and other ancillary improvements. No subterranean structures are currently planned. The purpose of our study was to evaluate the existing onsite geotechnical conditions and to provide geotechnical recommendations relative to the proposed project.

No grading plans were provided for our review in preparation of this report. Based on our exploration and analysis, the proposed project is considered feasible from a geotechnical standpoint. Geotechnical recommendations with respect to site grading and foundation design are presented in this report.

Respectfully submitted,

LEIGHTON AND ASSOCIATES, INC.

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1.0 INTRODUCTION

1.1 Site Description and Proposed Development

The project site is located at 2880 Mesa Verde Drive East in the city of Costa Mesa. The site location (latitude 33.6749°, longitude -117.9261°) and immediate vicinity are shown on Figure 1, *Site Location Map* The subject site is bordered by single family residential development to the north, Andros Street to the east, Life Coach Institute (single-story wood framed structures) to the south, and Mesa Verde Drive East to the west. Topographically, the site is relatively flat with approximate ground surface elevation ranging from 56 to 60 feet above mean sea level (msl). Current drainage is accomplished as sheet flow over paved surfaces to Mesa Verde Drive East.

The project site currently consists of church buildings with paved parking areas and underground utilities. The structures were occupied at the time of this report preparation and consist of wood-framed, single-story buildings built in 1968. Based on review of historical aerial photographs (NETR, 2014), the site previously was used for agriculture prior to construction of the church buildings.

No grading plan was provided for our review, however we understand based on review of information provided by you that the proposed residential development for the site includes grading to facilitate construction of ten single-family residences up to two stories in height with associated access, utilities, and other ancillary improvements.

1.2 Purpose and Scope of Exploration

The purpose of our geotechnical exploration was to evaluate the subsurface conditions at the site and provide geotechnical recommendations to aid in design and construction for the project as currently proposed.

The scope of this geotechnical report included the following tasks:

- *Background Review* – A background review was performed of readily available, relevant geotechnical and geological literature pertinent to the project site. References used in preparation of this report are listed in Section 7.0.

- *Field Exploration* – Our field exploration was performed on March 11, 2014, and consisted of four hollow-stem auger borings (designated as B-1, B-2, B-3, and B-4) drilled to depths between 26½ and 51½ feet below existing ground surface (bgs). The approximate locations of the explorations performed by Leighton are shown on Figure 2, *Boring Location Map*. Prior to the field exploration, the boring locations were marked and Underground Service Alert (USA) was notified for utility clearance.

  During drilling, both bulk and relatively undisturbed drive samples were obtained from the borings for geotechnical laboratory testing. Relatively undisturbed samples were collected from the borings using a Modified California Ring sampler conducted in accordance with ASTM Test Method D 3550. Standard Penetration Tests (SPT) were also performed within the hollow-stem auger test borings in accordance with ASTM Test Method D 1586. The samplers were driven for a total penetration of 18 inches, unless practical refusal, using a 140-pound automatic hammer falling freely for 30 inches. The number of blows per 6 inches of penetration was recorded on the boring logs.

  The borings were logged in the field by a certified engineering geologist from our technical staff. Each soil sample collected was reviewed and described in accordance with the Unified Soil Classification System. The samples were sealed and packaged for transportation to our laboratory. After completion of drilling, the borings were backfilled with soils generated during the exploration. The boring logs are presented in Appendix A, *Field Exploration Logs*.

- *Laboratory Testing* – Laboratory tests were performed on representative soil samples to evaluate geotechnical engineering properties of subsurface materials. The following laboratory tests were performed:

  - In-situ Moisture Content and Dry Density (ASTM D 2216 and ASTM D 2937);
  - Atterberg Limits (ASTM D 4318);
  - Consolidation (ASTM D 2435);
  - Percent Passing No. 200 Sieve (ASTM D 1140);
  - Expansion Index (ASTM D 4829); and
  - Direct Shear (ASTM D 3080).

The results of the moisture and density determination are shown on the borings logs included in Appendix A. The results of the remaining laboratory tests are presented in Appendix B, *Laboratory Test Results.*

- *Engineering Analysis* – Geotechnical analysis was performed on the collected data to develop conclusions and recommendations for design and construction presented in this report.

- *Report Preparation* - This geotechnical report presents our findings, conclusions, and recommendations.

It should be noted that the recommendations in this report are subject to the limitations presented in Section 6.0. An information sheet prepared by ASFE (the Association of Engineering Firms Practicing in the Geosciences) is also included at the rear of the text. We recommend that all individuals using this report read the limitations along with the attached document.

2.0 GEOTECHNICAL FINDINGS

2.1 Geologic Setting

The proposed development is located at the southern margin of the Los Angeles Basin in the northwestern region of Newport Mesa, a geographically distinct topographic feature that is traceable from south of San Onofre northward almost continuously to Dana Point. From Dana Point to Newport Beach the terrace becomes semi continuous due to erosion. This wave-cut bench in Miocene and Pliocene shale deposits (Monterey Formation) has been overlain by middle to early Pleistocene paralic deposits consisting of marine strandline, beach, estuarine and non-marine colluvial deposits composed of silt, sand and cobbles (Figure 3, *Regional Geology Map*).

The Newport Mesa is characterized by an upper surface sloping gently inland from an 85- to 105-foot high cliff that faces the sea along its southern edge. The Newport-Inglewood fault zone forms an important element of the regional tectonic structure, resulting in the broad up-arching and disruption of the subsurface formations before extending out to sea beneath the southeastern corner of the mesa. The landward tilt of the mesa surface is the southernmost on-land expression of deformation along the Newport-Inglewood fault zone (Barrows, 1974).

2.1.1 Geologic Structure

The Newport-Inglewood fault zone (NIFZ) is northwest-trending, right–lateral, strike-slip zone of approximately a 2- to 4-mile wide belt of anticlinal folds and faults disrupting early Holocene to Late Pleistocene-age and older deposits (Barrows, 1974) characterized by structural trends attributable to right-lateral shearing of basement rocks at depth (Moody and Hill, 1956). The zone defines the boundary between the western basement complex of Catalina type schist and related rocks to the southwest and the eastern basement complex of metasedimentary, metavolcanic, and plutonic rocks to the northeast (Yerkes et al., 1965). Right-lateral, strike-slip displacement of 3,000 to 5,000 feet has been measured in Lower Pliocene strata along the Newport-Inglewood structural zone (Dudley, 1954). Apparent vertical offset across faults of the Newport-Inglewood structural zone ranges from 4,000 feet at the basement interface, to 1,000 feet in the Pliocene strata, and 200 feet at the Plio-Pliestocene boundary (Yerkes et al., 1965). Movement along this structural zone is inferred to have been initiated during middle Miocene time (approximately 15 million years ago), with seismic activity continuing up to present time (Figure 5, *Historical Seismicity Map*). Tilted and structurally deformed sediments have also been observed within the structural Newport-Inglewood zone (Barrows, 1974).

2.2 Subsurface Soil Conditions

The field explorations (hollow stem auger borings) indicate the site is underlain by undocumented artificial fill and Quaternary age Pleistocene terrace deposits.

Artificial Fill, Undocumented: Map Symbol (Afu)

The artificial fill soils form a relatively thin mantle (2-3½ feet thick) and consist primarily of dark brown, stiff, silty to dark reddish brown, dense, coarse grained clayey with occasional manmade debris. For purposes of this report all existing fill soils are considered undocumented.

Quaternary Old Paralic Deposits: Map Symbol (Qopf)

The late to middle Pleistocene age terrace deposits consist mostly of interbedded to massive, impermeable clays to moderately permeable sands, reddish brown, interfingered strandline, beach, colluvial, and estuarine deposits composed of firm to hard, clay, sandy clay, silty clay, clayey silt, and sandy silt to medium dense to very dense sand with varying amounts of gravel and silt. Intermixed within the blocky structure of these deposits are varying proportions of calcium carbonate, oxidation staining, and clay development. This unit was deposited along a wave-cut abrasion platform during the late to middle Pleistocene (Morton D.M., and Miller, F.K., 2006,).

A more detailed description of the subsurface soils encountered in the borings is presented in the boring logs (Appendix A). Some of the engineering properties of these soils are described in the following subsections.

2.2.1 Expansive Soil

Expansive soils contain significant amounts of clay particles that swell considerably when wetted and which shrink when dried. Foundations constructed on these soils are subject to uplifting forces caused by the swelling. Without proper mitigation measures, heaving and cracking of both building foundations and slabs-on-grade could result. Based on our exploration, the near surface onsite soils consist predominantly of clayey sand to sandy clay. The onsite near surface soils are generally considered to have a moderate to high potential for expansion. The laboratory test result of a representative composite sample from Leighton boring LB-2 showed moderate expansion potential when wetted (EI = 35).

Variance in expansion potential of onsite soil is anticipated, therefore additional testing is recommended upon completion of rough grading to confirm the expansion potential result presented in this report.

2.2.2 Compressibility/Collapse Potential

Based on the results of consolidation tests, the onsite soils exhibit low compressibility characteristics when subject to the anticipated loading. Potential for collapse is not a design factor for this project.

2.3 Groundwater

Groundwater was not encountered during our field exploration to a depth explored of 51½ feet bgs. According to groundwater information obtained through the California Geological Survey (CGS), formerly the California Division of Mines and Geology, in the vicinity of the site, historically shallowest groundwater depth is approximately 30 feet below the existing ground surface (CGS, 1997). Based on the current proposed residential development scheme, groundwater is not expected to pose a constraint during construction.

Based on groundwater data presented in this report, seasonal fluctuations in groundwater elevations should be anticipated over time. Local perched groundwater conditions or surface seepage may develop once site development is completed and landscape irrigation commences.

3.0 GEOLOGIC/SEISMIC HAZARDS

Geologic and seismic hazards include surface faulting, seismic shaking, landslides, liquefaction, seismically induced settlement, lateral spreading, seismically induced landslides, seiches and tsunamis, and flooding. The following sections discuss these hazards and their potential impact at the project site.

3.1 Surface Fault Rupture

Our review of available in-house literature indicates that no known active faults have been mapped across the site, and the site is not located within a designated Alquist-Priolo Earthquake Fault Zone (CGS, 1986; Hart and Bryant, 2007). Therefore, a surface fault rupture hazard evaluation is not mandated for this site. There are no currently known active surface faults at this site (Figure 4, *Regional Fault Map*).

Presently, several sections of the Newport-Inglewood zone of deformation south and west of the site are included in the Alquist-Priolo Earthquake Fault Zone. However, from Huntington Beach Mesa southward, the Newport-Inglewood zone has not been designated as part of the Alquist-Priolo Earthquake Fault Zone, mainly because of the lack of evidence for faulting in young sediments. The South Branch of the Newport Inglewood fault zone trends just south the site. However, this fault is not considered active by State of California definition; therefore, the potential risk for surface fault rupture at this site is currently deemed low.

The location of the closest active faults to the site was generated using the United States Geological Survey (USGS) Earthquake Hazards Program (USGS, 2008a). The closest active faults to the site are the San Joaquin Hills blind thrust fault and the Newport-Inglewood Fault Zone, located approximately 1.4 miles and 3.0 miles, respectively, from the site. The San Andreas fault, which is the largest active fault in California, is approximately 49.2 miles northeast of the site.

3.2 Secondary Seismic Hazards

In general, secondary seismic hazards for the site could include soil liquefaction, seismically induced settlement, lateral spreading, seismically induced landsliding, seiches and tsunamis. These potential secondary seismic hazards are discussed below.

3.2.1 Liquefaction Potential

Liquefaction is the loss of soil strength or stiffness due to increasing pore-water pressure during severe ground shaking. Liquefaction is associated primarily with loose (low density), saturated, fine- to medium-grained, cohesionless soils.

As shown on the State of California Seismic Hazard Zones Map for the Newport Beach Quadrangle (CGS, 1998), this site is not located within an area that has been identified by the State of California as being potentially susceptible to liquefaction (Figure 6, *Seismic Hazard Map*). Furthermore, the blow counts recorded during our exploration did not suggest the site soil is prone to liquefaction. Therefore, it is our opinion that the potential for liquefaction occurring at the site is low.

3.2.2 Seismically Induced Settlement

During a strong seismic event, seismically induced settlement can occur within loose to moderately dense, unsaturated granular soils, separate from liquefaction. Settlement caused by ground shaking is often non-uniformly distributed, which can result in differential settlement. Based on blow count records, the seismically induced settlement under the building is anticipated to be less than one inch.

3.2.3 Lateral Spreading

Lateral spreading is a phenomenon in which large blocks of intact, non-liquefied soil move downslope on a liquefied soil layer. Lateral spreading is often a regional event. For lateral spreading to occur, the liquefiable soil zone must be laterally continuous, unconstrained laterally, and free to move along sloping ground. Due to the low susceptibility for liquefaction, the potential for lateral spreading is considered very low.

3.2.4 Seismically Induced Landslides

Significant slopes are not located on or near the site. Based on the State of California Seismic Hazard Zones Map for the Newport Beach Quadrangle (CGS, 1998), the site is not located within an area that has been identified by the State of California as being potentially susceptible to seismically induced landslides (Figure 6).

3.2.5 Seiches and Tsunamis

Seiches are large waves generated in very large enclosed bodies of water or partially enclosed arms of the sea in response to ground shaking. Tsunamis are waves generated in large bodies of water by fault displacement or major ground movement. According to the State of California Tsunami Inundation Map for Emergency Planning Newport Beach Quadrangle (CGS, 2009), the Site is situated well above the tsunami inundation line, therefore the risk of tsunami inundation is very low. Additionally, based on the lack of large enclosed water bodies nearby, seiche risks are considered very low.

3.3 Flooding Hazards

According to a Federal Emergency Management Agency (FEMA) flood insurance rate map (FEMA, 2008), the site is not located within a flood zone (Figure 7, *Flood Hazard Zone Map*).

4.0 DESIGN RECOMMENDATIONS

Geotechnical recommendations for the proposed development are presented in the following sections and are intended to provide sufficient geotechnical information to develop the project in general accordance with 2013 California Building Code (CBC) requirements. The following recommendations are considered preliminary and should be considered minimal from a geotechnical viewpoint as there may be more restrictive requirements of the architect, structural engineer, governing agencies and the City of Costa Mesa.

The geotechnical consultant should review the grading plan, foundation plan and specifications as they become available to verify that the recommendations presented in this report have been incorporated into the plans prepared for the project.

4.1 Earthwork

We recommend all earthwork for the project be performed in accordance with the following recommendations, future grading plan review report(s), the City of Costa Mesa grading requirements and the *General Earthwork and Grading Specifications* included in Appendix C. In case of conflict the following recommendations shall supersede those provided in Appendix C.

4.1.1 Site Preparation

Prior to construction, the areas proposed for residential development and improvements should be cleared of any existing improvements associated with the former land use (demolition of structures, concrete pads and asphalt) and properly disposed of offsite. Efforts should be made to locate any existing utility lines to be removed or rerouted where interfering with the proposed construction. Any resulting cavities should be properly backfilled and compacted. After the areas are cleared, the soils should be carefully observed for the removal of all potentially unsuitable deposits.

4.1.2 Overexcavation and Recommpaction

The existing undocumented artificial fill should be removed to expose competent native terrace deposits and replaced as engineered fill. The structural elements for the proposed residential structures and improvements may be supported on conventional shallow footing foundation systems established on at least three feet of engineered fill soils established on competent native soils (terrace deposits). All other incidental improvements (such as flatwork and hardscape) may be supported on one foot of engineered fill established on competent native soils. Overexcavation and recompaction should extend a minimum horizontal distance equal to the vertical distance between the proposed footing bottom and depth of overexcavation. However, care should be used to avoid undermining existing improvements surrounding the project site. Excavation adjacent to existing wall foundations in the north and south portions of the site that extend below bearing elevation may require slot-cutting techniques or shoring to perform the excavation and protect the foundations.

The "ABC" slot cut method may be used for construction of the new foundation located immediately adjacent to existing foundations. The initial cut along the excavation should not be steeper than 1H to 1V (horizontal to vertical). The maximum width and height of the slots should not exceed eight feet. The width of the earth buttress on either side of the slot should be maintained at a minimum of 12 feet.

After completion of the overexcavation and prior to fill placement or other improvements such as flatwork and hardscape, the exposed soils should be scarified to a minimum depth of six inches, moisture conditioned 3 to 4 percentage points above optimum moisture content and compacted to a minimum of 90 percent relative compaction (ASTM D 1557).

4.1.3 Fill Placement

The onsite soils, less any deleterious material (construction debris) or organic matter, can be used in required fills. Oversized material greater than 6 inches in maximum dimension should not be placed in the fill. Areas prepared to receive structural fill and/or other surface improvements should be scarified, brought to 3 to 4 percent over optimum moisture content and recompacted to at least 90 percent relative compaction per ASTM Test Method D 1557.

Any required import material should consist of non-corrosive and relatively non-expansive soils with an Expansion Index (EI) less than 20. The imported materials should contain sufficient fines (binder material) so as to result in a stable subgrade when compacted. All proposed import materials should be approved by the geotechnical engineer of record prior to being placed at the site. The use of free-draining granular soils as structural compacted fill adjacent to or within the proposed buildings is generally not recommended since soils of this type can allow the accumulation of water infiltration, which may activate the expansive characteristics of the underlying soils.

With the anticipation that all fill soil will be derived from onsite moderate to highly expansive clay earth material all fill should be placed in thin, loose lifts, with each lift properly moisture conditioned 3 to 4 percentage points above optimum moisture content and compacted to a minimum of 90 percent relative compaction (ASTM D 1557). The optimum lift thickness to produce a uniformly compacted fill will depend on the type and size of compaction equipment used. Proper moisture conditioning of the soils is vital in reducing expansion potential and reducing the potential for post-construction heave that may result in distortion and possibly damage to new improvements. Aggregate base should be compacted to a minimum of 95 percent relative compaction (ASTM D 1557).

4.1.4 Pipe Bedding

Any proposed pipe should be placed on properly placed bedding materials. Pipe bedding should extend to a depth in accordance with the pipe manufacturer's specification. The pipe bedding should extend to at least 12 inches over the top of the pipeline. The bedding material may consist of compacted free-draining sand, gravel, or crushed rock and should be densified by mechanical means (flooding or jetting are not appropriate at this site). Pipe bedding material should have a Sand Equivalent (SE) of at least 30 per California Test Method CTM-217. A 5-foot-long seepage plug consisting of clay soil or CLSM slurry should be placed as backfill where the trench enters under the building slab, with the purpose of preventing water from within the trench bedding from seeping into/under the building pad.

4.1.5 Trench Backfill

Trench excavations above pipe bedding zone may be backfilled with onsite soils under the observation of the geotechnical consultant. All fill soils should be placed in loose lifts, moisture conditioned as required and compacted to a minimum of 90 percent relative compaction based on ASTM Test Method D 1557. Lift thickness will be dependent on the equipment used as suggested in the latest edition of the Standard Specifications for Public Works Construction (Greenbook). The fill soils should extend to the bottom of the aggregate base for new pavement, or to finished grade in non-paved areas.

4.1.6 Surface Drainage

Positive drainage of surface water away from structures is very important. Water should not be allowed to pond adjacent to buildings. Positive drainage may be accomplished by providing drainage away from buildings a minimum of 2 percent for earthen surfaces for a lateral distance of at least five feet and further maintained by a swale or drainage path at a gradient of at least 1 percent. Where necessary, drainage paths may be shortened by the use of area drains and collector pipes. Eave gutters are recommended and should reduce water infiltration into the subgrade materials. Downspouts should be connected to appropriate outlet devices.

Irrigation of landscaping should be controlled to maintain, as much as possible, consistent moisture content sufficient to provide healthy plant growth without over watering.

4.2 Seismic Design Parameters

To accommodate effects of ground shaking produced by regional seismic events, seismic design can, at the discretion of the designing Structural Engineer, be performed in accordance with the 2013 edition of the California Building Code (CBC). Table 1, *2013 CBC Seismic Parameters*, lists seismic design parameters based on the 2013 CBC methodology, which is based on ASCE/SEI 7-10:

2013 CBC Seismic Parameters

Seismic Design Parameters	Value
Site Latitude (decimal degrees)	33.6749
Site Longitude (decimal degrees)	-117.9261
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))	1.601
Mapped Spectral Response Acceleration at 1s Period, S_1 (Figure 1613.3.1(2))	0.592
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)	1.601
Adjusted Spectral Response Acceleration at 1s Period, S_{M1} (Eq. 16-38)	0.888
Design Spectral Response Acceleration at 0.2s Period, S_{DS} (Eq. 16-39)	1.067
Design Spectral Response Acceleration at 1s Period, S_{D1} (Eq. 16-40)	0.592

4.3 Footing Foundations

New shallow spread footings established on engineered fill may be used to support the proposed residential structures. It is anticipated that a perimeter property line free standing wall will be constructed around a majority of the site. Earthwork removals may be limited due to the proximity of the adjacent property line. Footings may need to be deepened due to grading limitations from property line constraints. Recommendations for deepened footings can be provided in a geotechnical grading plan review report based on the proposed grading plan.

Due to the variance in expansion potential anticipated, additional testing is recommended upon completion of rough grading to verify the expansion potential results presented in this report. These recommendations may need to be revised based on results of future testing.

4.3.1 Minimum Embedment and Width

Continuous strip footings should have a minimum width of 18 inches. Isolated square pad column footings are recommended to be a minimum of 24 inches in width. The bottom of the footing should be at least 24 inches below lowest adjacent grade or finish floor elevation.

4.3.2 Allowable Bearing Pressure

The footings may be designed for a maximum net allowable soil bearing pressure of 2,000 pounds per square foot (psf) for isolated column footings and 3,000 psf continuous strip footings. The soil bearing pressure may be increased by one-third for transient loads such as wind and seismic forces.

4.3.3 Lateral Load Resistance

Resistance to lateral loads will be provided by a combination of friction between the soil and foundation interface and passive pressure acting against the vertical portion of the footings. For calculating allowable lateral resistance, a passive pressure of 250 psf per foot of depth to a maximum of 2,500 psf and a frictional coefficient of 0.30 may be used provided the foundations are supported within structural compacted fill as previously described. No reduction is necessary when combining frictional and passive resistance.

4.3.4 Settlement

The estimated total settlement of the structures supported on spread footings as recommended above is less than 1 inch. The differential settlement between adjacent columns is estimated to be less than ½ inch over a horizontal distance of 30 feet.

4.4 Conventional Slab-On-Grade

Concrete slabs may be designed using a modulus of subgrade reaction of 100 pci provided the subgrade is prepared as described in Section 4.1 of this report, which includes proper moisture conditioning and recompaction of the soils. Moisture content in the finish subgrade within the building footprint should be maintained at 120 percent above the optimum moisture content to a depth of at least 24 inches. For areas 5 feet laterally outside the building footprint, the moisture content within the top 16 inches of finish grade should be maintained at 120 percent above optimum moisture content. The subgrade soils should be evaluated by the geotechnical engineer at the time of construction to verify adequate moisture conditioning has been performed and maintained. From a geotechnical standpoint, we recommend slab-on-grade be a minimum four inches thick with No. 4 rebar placed at the center of the slab at 18 inches on center in each direction. The structural engineer should design the actual thickness and reinforcement based on anticipated loading conditions and expansive characteristics of the onsite soil.

Minor cracking of concrete after curing due to drying and shrinkage is normal and should be expected; however, concrete 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 should also be expected. The use of low-slump concrete or low water/cement ratios can reduce the potential for shrinkage cracking. Additionally, our experience indicates that the use of reinforcement in slabs and foundations can generally reduce the potential for concrete cracking. To reduce the potential for excessive cracking, concrete slabs-on-grade should be provided with construction or weakened plane joints at frequent intervals. Joints should be laid out to form approximately square panel

4.5 Post-Tensioned Foundation Recommendations

As an alternative to conventional slab-on-grade, a post-tensioned slab may be used. Based on results of this investigation, preliminary recommendations for post-tensioned slabs design are as follows:

Condition	Center Lift	Edge Lift
Edge Moisture Variance Distance, e_m (feet)	5.3	3.7
Differential Soil Movement, y_m (inches)	4.5	1.6

The slabs may be designed for an average allowable bearing pressure of 1,500 pounds per square foot (psf) for dead plus live loads with maximum localized bearing pressure of 2,000 psf for column or wall loads. The allowable bearing pressure may be increased by one-third for short-term loading including wind and seismic loads. The structural engineer should also design the post-tensioned slabs with adequate stiffness to minimize potential cracking in the slabs. A minimum thickness of 12 inches should be maintained around the outer edge of the slab below the lowest adjacent grade.

We also recommend that the moisture content in the finish subgrade within the building footprint be maintained at 120 percent above the optimum moisture content to a depth of at least 24 inches. For areas five feet laterally outside the building footprint, the moisture content within the top 16 inches of the finish subgrade should be maintained at 120 percent above the optimum moisture content. Adequate observation and testing should be performed during future site grading to verify the moisture and density of the in-place fill and new fill meet the desired requirements.

We recommend additional Expansion Index tests be conducted prior to the home construction phase. The above recommended design criteria may subject to change if the expansion potential of the subgrade soil is found to be different than assumed herein.

4.6 Vapor Retarder

The following recommendations are for informational purposes since they are unrelated to the geotechnical performance of the foundation. Post construction moisture migration should be expected below the foundation.

In general, interior floor slabs with moisture sensitive floor coverings are recommended to be underlain by a minimum 10-mil thick vapor retarder that has a permeance of less than 0.3 perms, as determined by ASTM E 96, and meets the applicable code requirements (ASTM E 1745). The foundation engineer/architect should determine whether the use of a capillary break (crushed gravel layer) in conjunction with a vapor retarder is necessary or required by code. Sand layer thickness above the barrier should also be determined by the foundation engineer/architect. Sand layers should be installed where applicable in accordance with ACI Publication 302 Guide for Concrete Floor and Slab Construction.

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 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.

4.7 Stormwater Infiltration

No plans regarding the design of stormwater infiltration devices were presented for our review. Often, a combination of methods is implemented to reduce storm water runoff and increase infiltration including permeable pavements, grass-lined swales, retention areas and/or drywells.

Due to the thick, expansive clay layer encountered in our borings that extends 15 feet to 25 feet bgs, stormwater infiltration is not feasible for this project site.

4.8 Lateral Earth Pressures

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 8, *Retaining Wall Backfill and Subdrain Detail.* Using expansive soil as retaining wall backfill will result in higher lateral earth pressures exerted on the wall and are, therefore, not recommended. Based on these recommendations, the following parameters may be used for the design of conventional retaining walls.

Static Equivalent Fluid Pressure (pcf)	
Condition	Level Backfill
Active	40
At-Rest	55
Passive (ultimate)	250 (Maximum 2,500 psf)

The above values do not contain an appreciable factor of safety, so the structural engineer should apply the applicable factors of safety and/or load factors during design. 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 0.30 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. A soil unit weight of 125 pcf may be assumed for calculating the actual weight of the soil over the wall footing.

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 third of uniform vertical surcharge loads should be applied at the surface as a horizontal pressure on cantilever (active) retaining walls, while half of uniform vertical surcharge-loads should be applied as a horizontal pressure on braced (at-rest) retaining walls. To account for automobile parking surcharge, we suggest that a uniform horizontal pressure of 100 psf (for restrained walls) or 70 psf (for cantilever walls) be added for design, where autos are parked within a horizontal distance behind the retaining wall less than the height of the retaining wall stem.

For walls with a retained height over 12 feet, or where otherwise required by Code or deemed appropriate by the structural engineer, we recommend that the wall designs be checked seismically using an *additive seismic* Equivalent Fluid Pressure (EFP) of 15 pcf, which is added to the *active* EFP. Such walls that are to be designed in the static case assuming the *at-rest* condition should be checked seismically using this *additive seismic* EFP added to the *active* condition (i.e., the *additive seismic* EFP is not added to the *at-rest* EFP). The *additive seismic* EFP should be applied with a standard EFP pressure distribution (i.e., it is not an inverted triangle).

Conventional retaining wall footings should have a minimum width of 18 inches and a minimum embedment of 24 inches below the lowest adjacent grade. An allowable bearing pressure of 2,000 psf may be used for retaining wall footing design, based on the minimum footing width and depth. This bearing value may be increased by 250 psf per foot increase in width or depth to a maximum allowable bearing pressure of 3,000 psf.

4.9 Pavement Design

Based on the design procedures outlined in the current Department of Transportation (Caltrans) Highway Design Manual, and using an assumed R-value of 15 for subgrade and 78 for crushed aggregate base course, the following flexible pavement sections may be used for various Traffic Indices.

Traffic Index	Asphalt Concrete (inches)	Base Course (inches)
5.0 or less	3	8
6.0	4	10

In areas where Portland Cement Concrete (PCC) pavements are planned, such as fire-truck access road, the pavement is recommended to be a minimum of six inches in thickness underlain by a minimum six inches of base course.

All pavement construction should be performed in accordance with the latest edition of the Standard Specifications for Public Works Construction (SSPWC). Field observation 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 base course, the subgrade soil should be scarified to a minimum depth of six inches, moisture-conditioned to 3 to 4 percent above optimum moisture content, and recompacted to a minimum of 90 percent relative compaction. Base material should be placed in thin lifts, moisture conditioned as necessary, and compacted to a minimum of 95 percent relative compaction.

Upon completion of rough grading samples of street subgrade should be collected and tested for R-value to verify the assumed value used in design of structural sections in this report. Additionally, if paver construction is considered, the concrete paver type should be provided to Leighton along with the appropriate Traffic Index (TI) values to generate appropriate recommendations for structural paver support.

4.10 Grading and Foundation Plan Review

When available, grading and foundation plans should be reviewed by Leighton in order to verify our geotechnical recommendations are properly implemented into design of the project. Updated recommendations and/or field work may be necessary.

5.0 CONSTRUCTION CONSIDERATIONS

5.1 Temporary Excavations

All temporary excavations, including footings, utility trenches, should be performed in accordance with project plans, specifications, and all OSHA requirements. Excavations 5 feet or deeper should be laid back or shored in accordance with OSHA requirements before personnel are allowed to enter.

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 cut, unless the cut is shored appropriately.

During construction, the soil conditions should be regularly evaluated to verify that conditions are as anticipated. The contractor shall be responsible for providing the “competent person” required by OSHA standards to evaluate soil conditions. Soil types will vary, but Type C soils can be expected at shallow depths. Close coordination between the competent person and the geotechnical engineer should be maintained to facilitate construction while providing safe excavations.

5.2 Additional Geotechnical Services

The geotechnical recommendations presented in this report are based on subsurface conditions as interpreted from limited subsurface explorations and limited laboratory testing. Our conclusions and recommendations presented in this report should be reviewed and verified by Leighton during site grading and construction and revised accordingly, if exposed geotechnical conditions vary from our preliminary findings and interpretations. The recommendations presented in this report are only valid if Leighton verifies the site conditions during construction.

Geotechnical observation and testing should be provided during the following activities:

- Grading and excavation of the site;
- Subgrade preparation;
- Compaction of all fill materials;

- Utility trench backfilling and compaction;
- Footing excavation and slab-on-grade preparation;
- During installation of temporary shoring, wherever needed; and
- When any unusual conditions are encountered.

6.0 LIMITATIONS

This report was based solely on data obtained from a limited number of geotechnical exploration, and soil 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, the findings, conclusions, and recommendations presented in this report are only valid if Leighton and Associates, Inc. has the opportunity to observe subsurface conditions during grading and construction, to confirm that our preliminary data are representative for the site. Leighton and Associates, Inc. should also review the construction plans and project specifications, when available, to comment on the geotechnical aspects.

This report was prepared using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants practicing in this or similar localities. The findings, conclusion, and recommendations included in this report are considered preliminary and are subject to verification. We do not make any warranty, either expressed or implied.

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Important Information about Your

Geotechnical Engineering Report

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

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

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

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

Read the Full Report

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

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

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

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

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

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

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

Subsurface Conditions Can Change

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

Most Geotechnical Findings Are Professional Opinions

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

A Report's Recommendations Are *Not* Final

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

A Geotechnical Engineering Report Is Subject to Misinterpretation

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

Do Not Redraw the Engineer's Logs

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

Give Contractors a Complete Report and Guidance

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

Read Responsibility Provisions Closely

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

Geoenvironmental Concerns Are Not Covered

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

Obtain Professional Assistance To Deal with Mold

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

Rely, on Your ASFE-Member Geotechncial Engineer for Additional Assistance

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

ASFE
THE BEST PEOPLE ON EARTH

8811 Colesville Road/Suite G106, Silver Spring, MD 20910
Telephone: 301/565-2733 Facsimile: 301/589-2017
e-mail: info@asfe.org www.asfe.org

Project: 10646.001	Eng/Geol: CR/JAR
Scale:1 " = 2,000 '	Date: March 2014

Base Map: ESRI ArcGIS Online 2014
Thematic Information: Leighton
Author: Leighton Geomatics (mmurphy)

SITE LOCATION MAP

2880 Mesa Verde Drive East
Costa Mesa, California

Figure 1

Leighton

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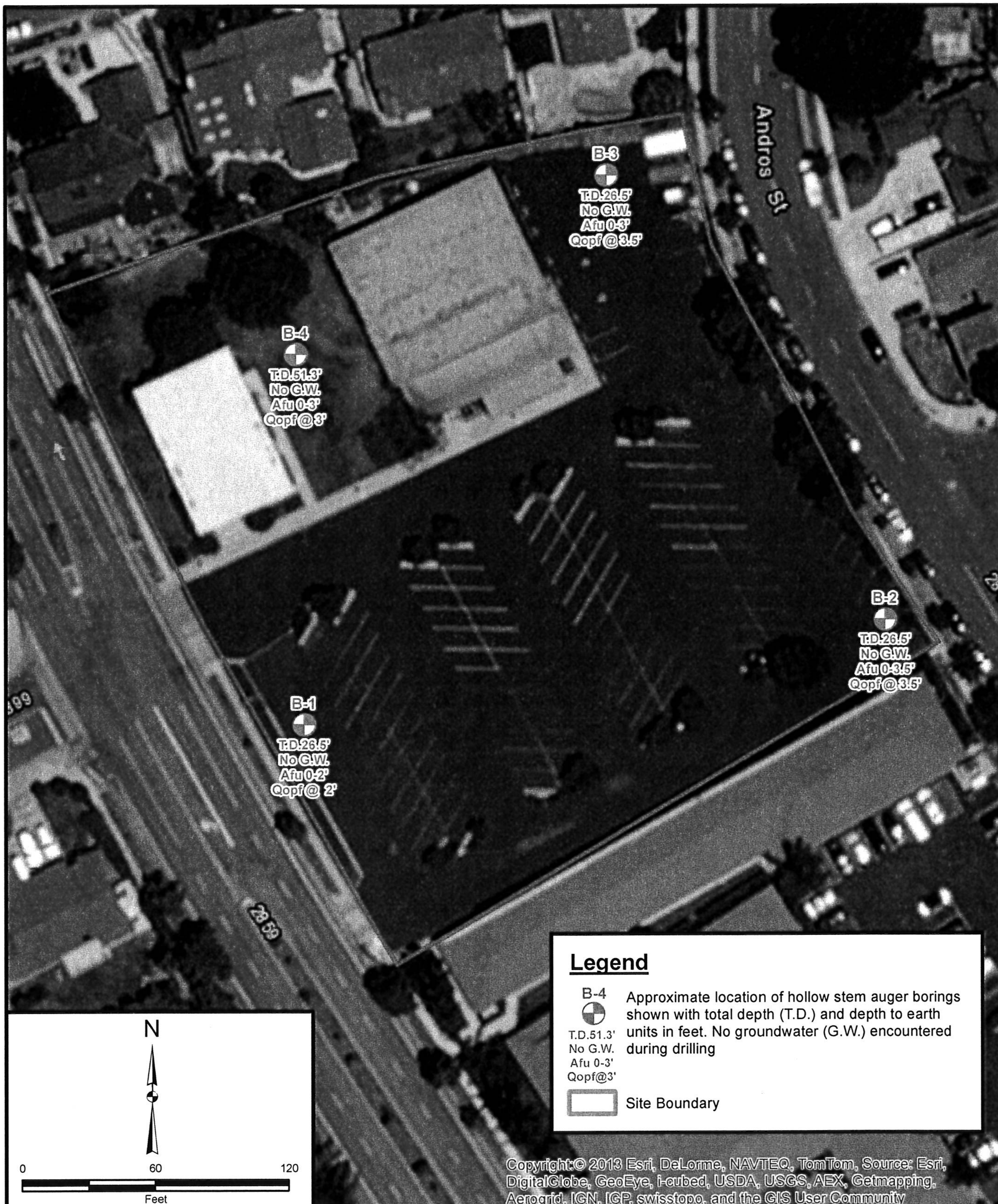


Project: 10646.001	Eng/Geol: CR/JAR	Figure 2
Scale: 1 " = 60 '	Date: March 2014	

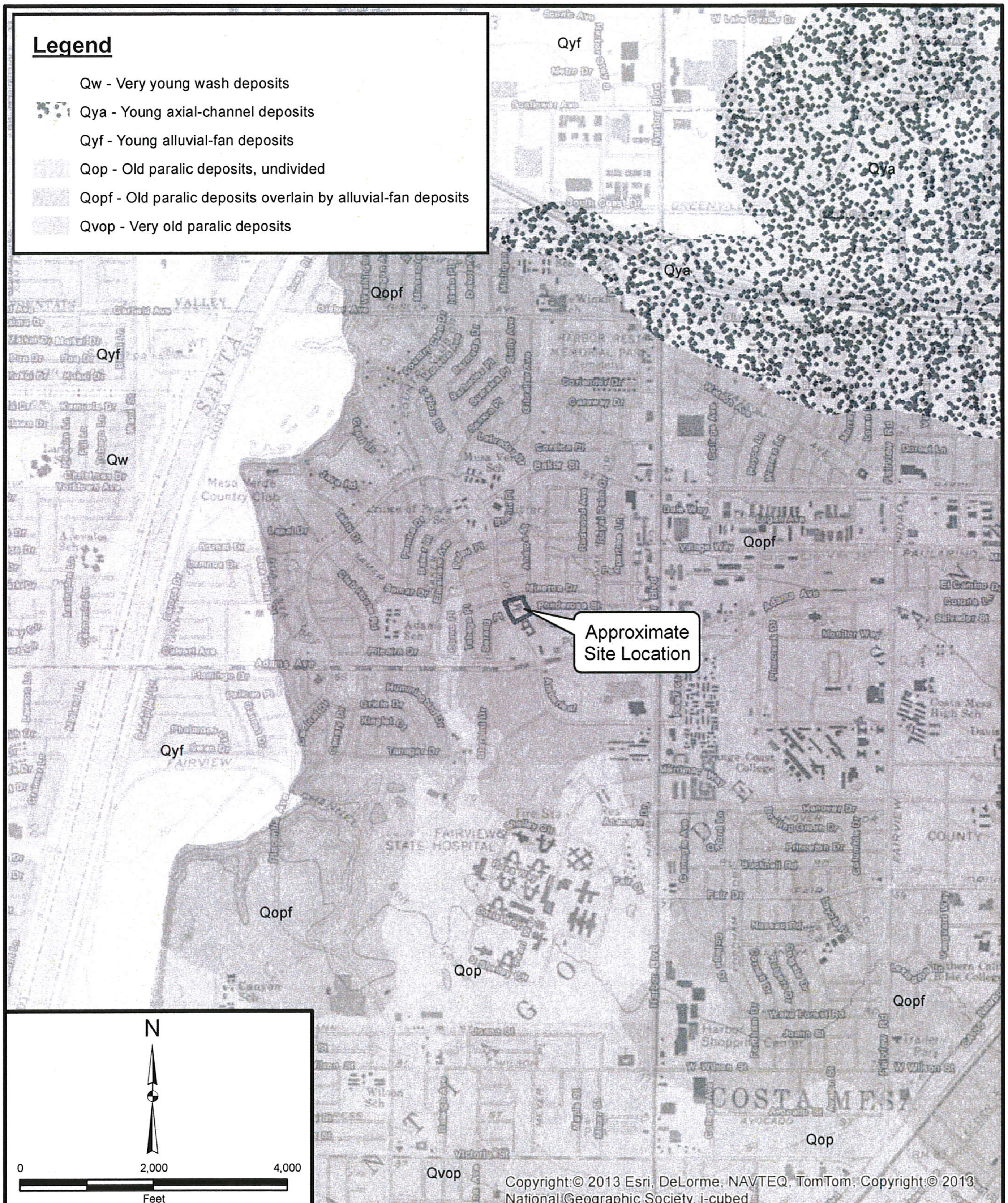
Base Map: ESRI ArcGIS Online 2014
Thematic Information: Leighton
Author: Leighton Geomatics (mmurphy)

BORING LOCATION MAP

2880 Mesa Verde Drive East
Costa Mesa, California

Leighton

Map Saved as P:\drafting\10646\001\GIS\of_2014-03-10\10646.001_figure2_BLM.mxd on 3/25/2014 11:11:01 AM



Project: 10646.001	Eng/Geol: CR/JAR
Scale:1 " = 2,000 '	Date: March 2014

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Author: Leighton Geomatics (mmurphy)

REGIONAL GEOLOGY MAP

2880 Mesa Verde Drive East
Costa Mesa, California

Figure 3

Leighton

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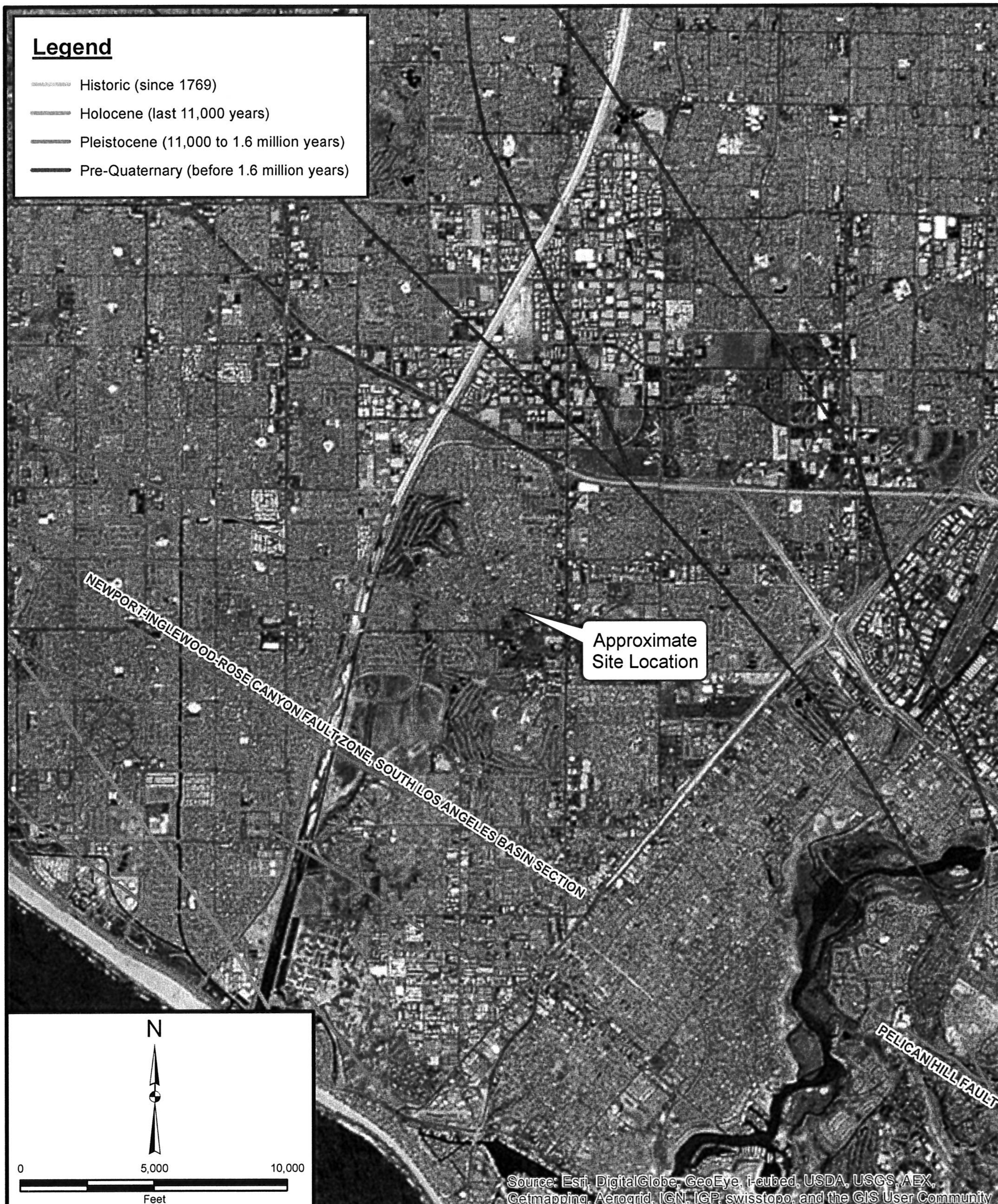

Project: 10646.001	Eng/Geol: CR/JAR
Scale:1 " = 5,000 '	Date: March 2014

Faults: CGS, 2010
Author: Leighton Geomatics (mmurphy)

REGIONAL FAULT MAP

2880 Mesa Verde Drive East
Costa Mesa, California

Figure 4

Leighton

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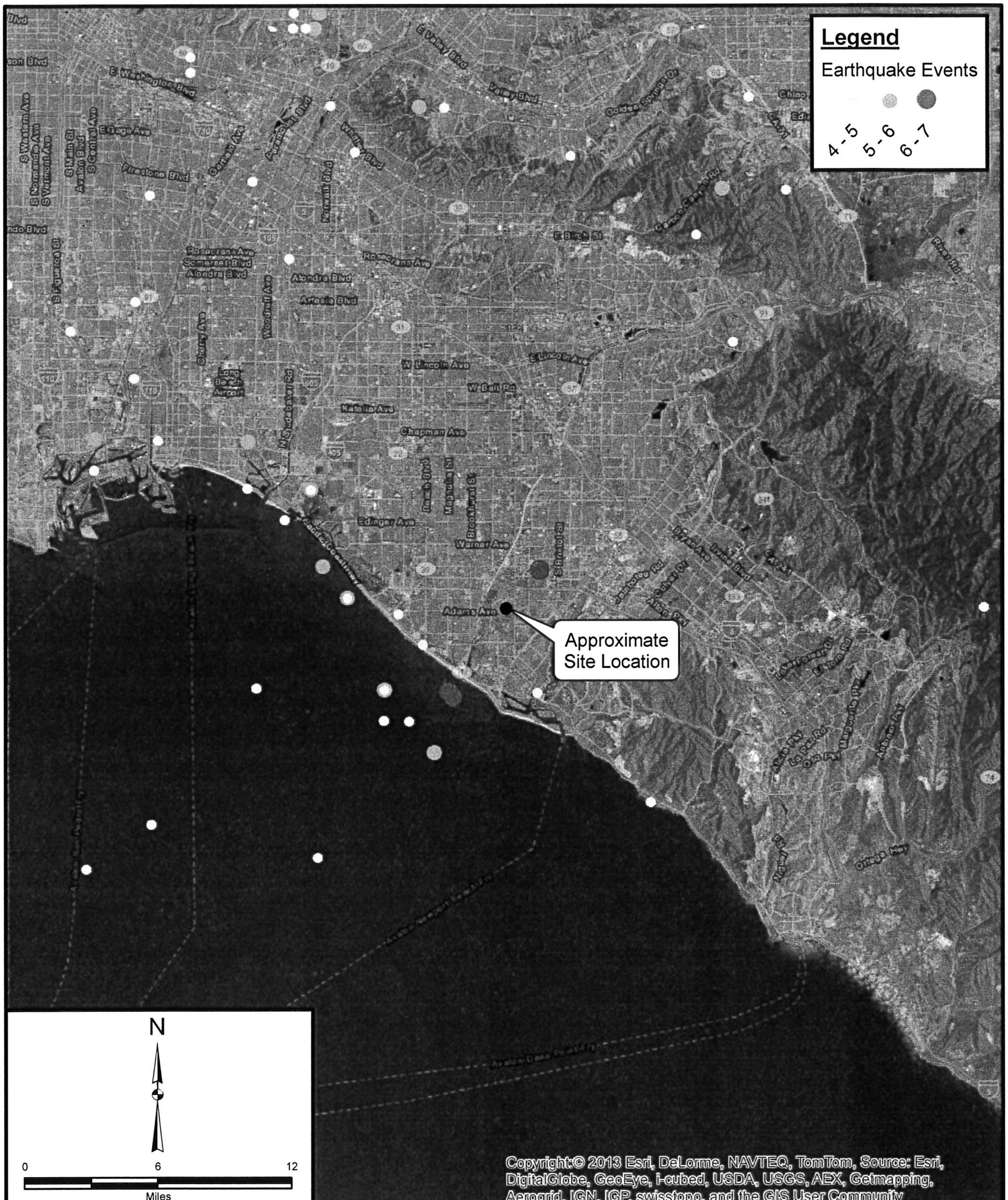

Project: 10646.001	Eng/Geol: CR/JAR
Scale:1 " = 6 miles	Date: March 2014

Base Map: ESRI ArcGIS Online 2014
Thematic Information: Leighton
Author: Leighton Geomatics (mmurphy)

HISTORICAL SEISMICITY MAP

2880 Mesa Verde Drive East
Costa Mesa, California

Figure 5

Leighton

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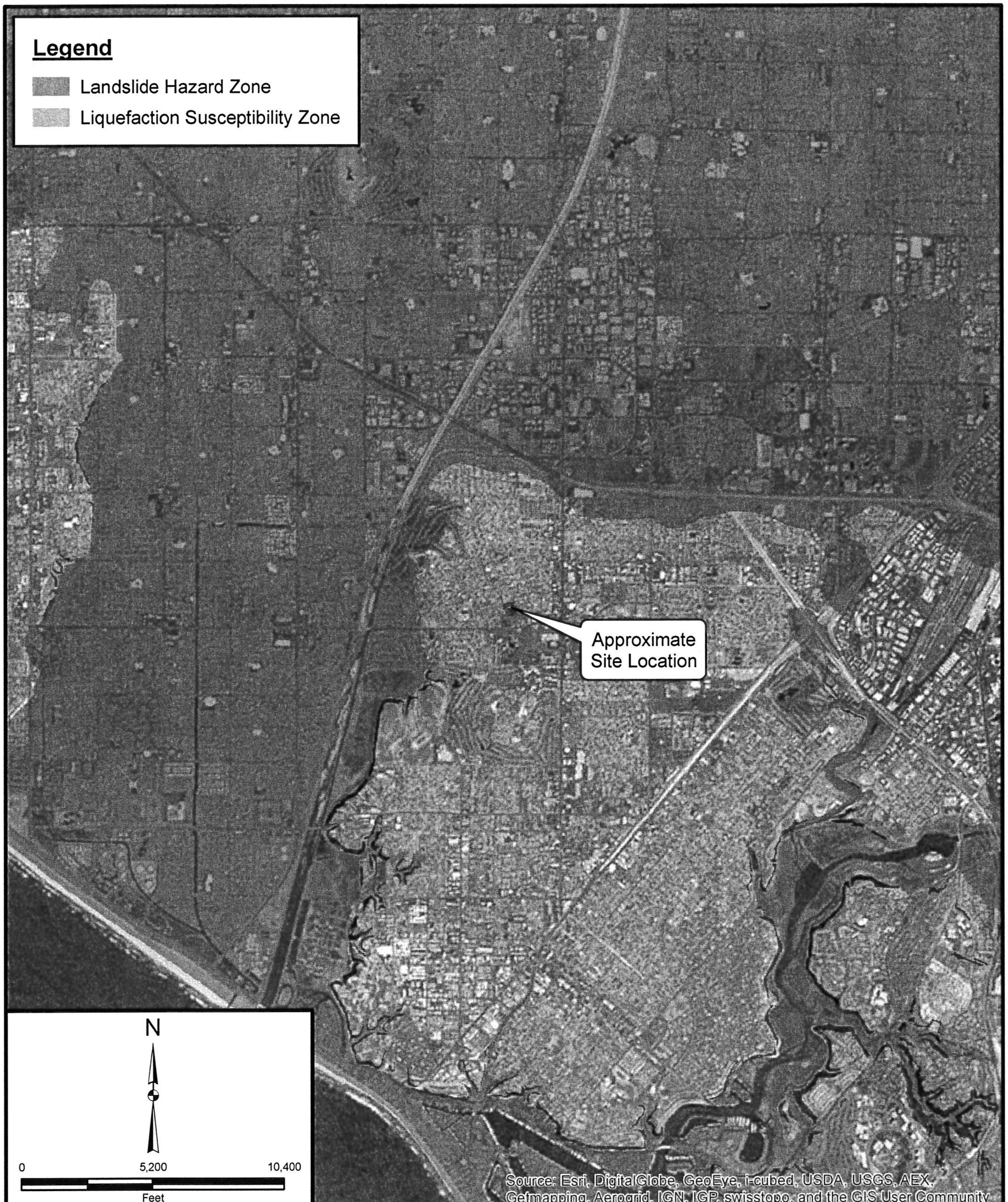


Project: 10646.001	Eng/Geol: CR/JAR
Scale: 1 " = 5,280 '	Date: March 2014

CGS, Seismic Hazards Zonation Program

Author: Leighton Geomatics (mmurphy)

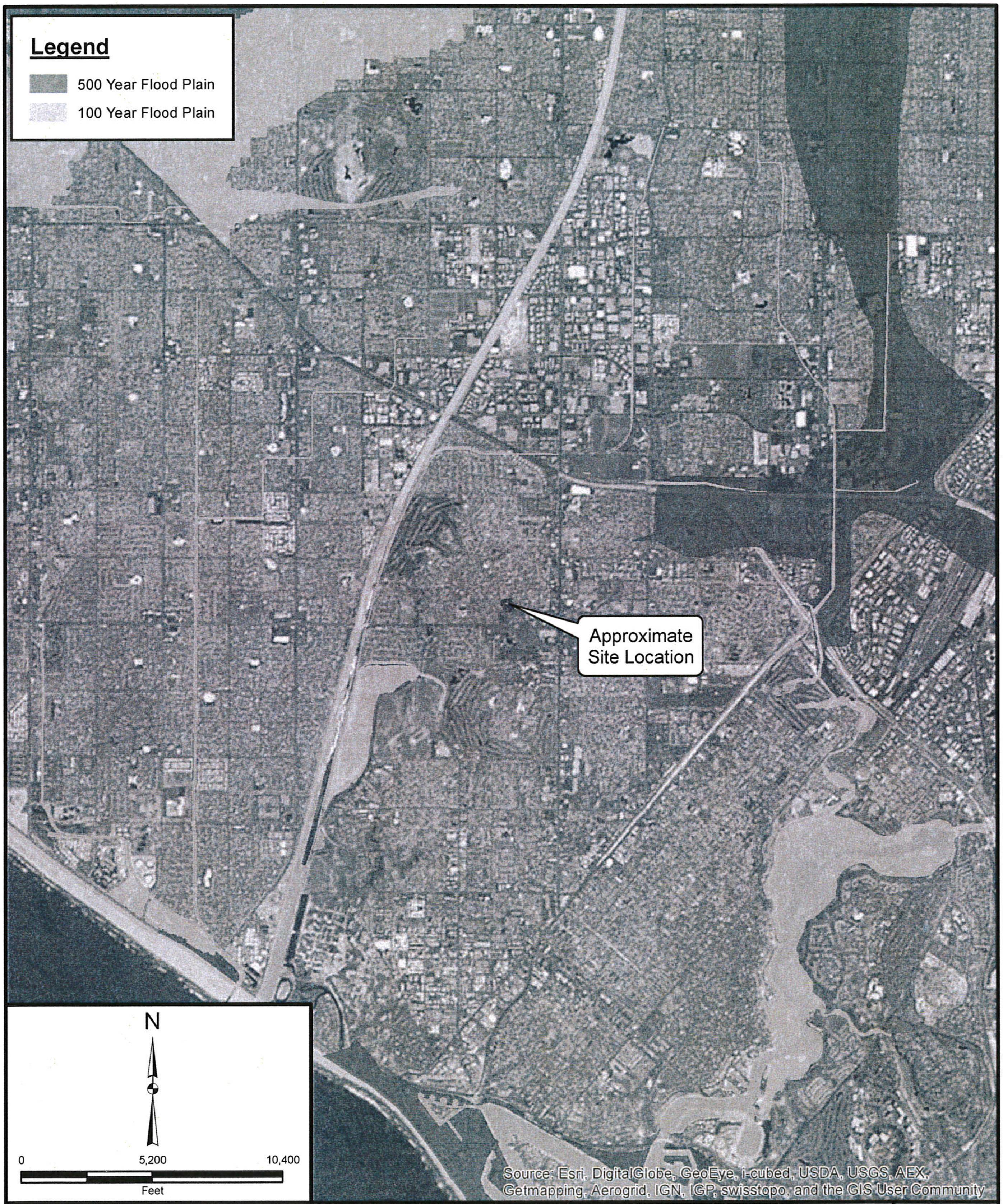
SEISMIC HAZARD MAP

2880 Mesa Verde Drive East
Costa Mesa, California

Figure 6

Leighton

Map Saved as V:\drafting\10646\001\GIS\of_2014-03-10\10646.001_figure6_SeisHazMap.mxd on 3/25/2014 11:05:53 AM



Project: 10646.001	Eng/Geol: CR/JAR
Scale:1 " = 5,280 '	Date: March 2014

CGS, FEMA Q3 Flood Data

Author: Leighton Geomatics (mmurphy)

FLOOD HAZARD ZONE MAP

2880 Mesa Verde Drive East
Costa Mesa, California

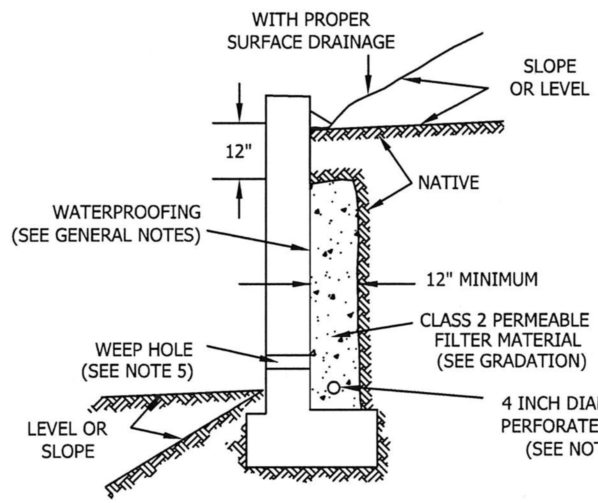
Figure 7

Leighton

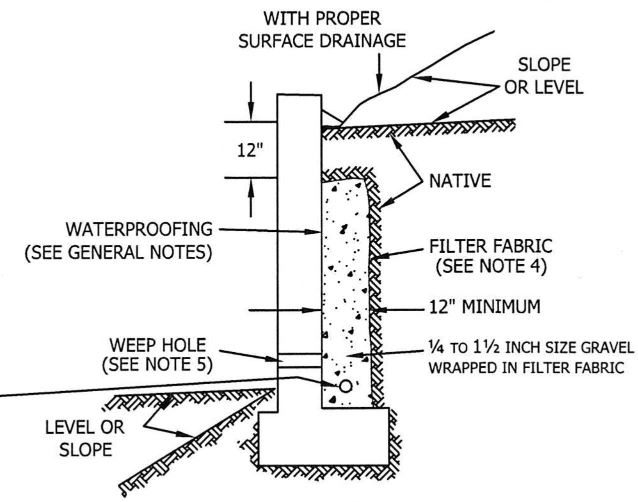
Map Saved as V:\drafting\10646\001\GIS\of_2014-03-10\10646.001_figure7_FloodHazMap.mxd on 3/25/2014 11:07:32 AM

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 8

APPENDIX A

GEOTECHNICAL BORING LOG B-1

Project No.	10646-001	**Date Drilled**	3-11-14
Project	2880 Mesa Verde Dr. East	**Logged By**	Joe Roe
Drilling Co.	Martini Drilling Corporation	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger, 140lb Autohammer, 30" Drop	**Ground Elevation**	59'
Location	See Figure 2 - Boring Location Map	**Sampled By**	Joe Roe

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							CL	@0': 4-inches Asphalt Concrete (AC) over 3.5-inches asphalt treated Aggregate Base (AB), non-woven geofabric @2.5-inches.	
				R-1	3 4 8	110.4	18	CL	**Artificial Fill, undocumented: (Afu)** @8-inches: Silty CLAY (CL), dark brown, very moist, with fine grained sand and asphalt pieces.	
55									**Quaternary Old Paralic Deposits: (Qopf)** @2': Sandy CLAY (CL), dark reddish brown, very moist, firm, medium grained sand.	
	5			R-2	5 16 28	114.3	17		@5': Hard, oxidized, reddish brown, coarse grained sand, minor gleying.	
				R-3	7 15 24	114.7	17		@7': Dark reddish brown, moist, very stiff, medium grained sand, well developed blocky structure.	
50										
	10			R-4	7 13 16	94.3	29		@10': CLAY (CL), dark reddish brown to reddish black, moist, very stiff, some silt and fine grained sand, heavy manganese development.	
45										
	15			R-5	8 15 18	95.4	27	ML-CL	@15': Clayey SILT to Silty CLAY (ML-CL), olive green, moist, very stiff, very fine grained sandy laminations.	
40										
	20			S-1	4 11 13			ML	@20': Sandy SILT (ML), olive brown, slightly moist, very stiff, very fine grained, trace clay.	
								CL	@21.3': CLAY (CL), dark reddish brown, slightly moist, very stiff.	
								SP-SM	@22': SAND with silt (SP-SM), yellow brown, moist, dense, very fine grained, poorly graded, oxidized.	
35										
	25			S-2	4 14 19					
									Total Depth of Boring: 26.5 feet bgs No Groundwater encountered during drilling Boring backfilled with soil cuttings and capped with cold patch asphalt upon completion of drilling. Excess soil cuttings spread on site.	
30	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 B-2

Project No.	10646-001	**Date Drilled**	3-11-14
Project	2880 Mesa Verde Dr. East	**Logged By**	Joe Roe
Drilling Co.	Martini Drilling Corporation	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger, 140lb Autohammer, 30" Drop	**Ground Elevation**	58'
Location	See Figure 2 - Boring Location Map	**Sampled By**	Joe Roe

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
	0			BB-1				SC	@0': 2-inches Asphalt Concrete (AC) over 3-inches Aggregate Base (AB), asphalt treated Aggregate Base (AB), non-woven geofabric @2-inches. **Artificial Fill, undocumented: (Afu)**	
55				R-1	8 16 34	124.4	10		@5": Clayey SAND (SC), orange brown to dark reddish brown, moist, dense, coarse grained sand.	EI
								CL	**Quaternary Old Paralic Deposits: (Qopf)** @3.5': Sandy CLAY (CL), dark reddish brown, slightly moist, very stiff, fine to coarse grained sand, moderate blocky structure, clay lined pedogenic faces, approximate channel incision filled with clayey sand (SC), spotty manganese development in sand channel.	
	5			R-2	5 12 16					DS
50				R-3	9 23 28	96.7	27		@7': CLAY (CL), olive grayish white, moist, hard, abundant $CaCO_3$ development with concretions.	
	10			R-4	9 14 19	93.8	36		@10': CLAY with silt, gray, very stiff.	
45										
	15			R-5	5 10 19	95.0	30		@15': Silty CLAY (CL), olive gray, moist, very stiff, abundant $CaCO_3$, moderate blocky structure.	
40										
	20			S-1	5 12 14			ML	@20': Sandy SILT (ML), yellow brown, dry, very stiff, very fine grained, micaceous.	
35										
	25			S-2	12 26 29			SP	@25': SAND (SP), light yellow brown, dry, very dense, fine to medium grained sand, poorly graded.	
30									Total Depth of Boring: 26.5 feet bgs No Groundwater encountered during drilling Boring backfilled with soil cuttings and capped with cold patch asphalt upon completion of drilling. Excess soil cuttings spread on site.	
	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	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 B-3

Project No.	10646-001	**Date Drilled**	3-11-14
Project	2880 Mesa Verde Dr. East	**Logged By**	Joe Roe
Drilling Co.	Martini Drilling Corporation	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger, 140lb Autohammer, 30" Drop	**Ground Elevation**	57'
Location	See Figure 2 - Boring Location Map	**Sampled By**	Joe Roe

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							CL	@0': 4-inches Asphalt Concrete over 3-inches asphalt treated Aggregate Base, non-woven geofabric @2-inches. **Artificial Fill, undocumented: (Afu)** @1': Silty CLAY with sand (CL), dark brown, wet, fine to coarse grained sand, asphalt pieces.	
55				R-1	3 6 13					
								CL	**Quaternary Old Paralic Deposits: (Qopf)** @3': Sandy CLAY (CL), dark reddish brown, moist, stiff, fine to medium grained sand.	
	5			R-2	5 12 14				@5': Very stiff, minor $CaCO_3$.	
50				R-3	3 7 12				@7': CLAY (CL), olive gray to grayish white, moist, stiff, abundant $CaCO_3$ development, hackly structure with concretions, blocky.	
	10			R-4	7 13 19				@10': Olive green, very stiff, heavy carbonate mineralization, paleosol, laminated.	
45										
	15			R-5	6 14 21			ML	@15': Clayey SILT (ML), olive green, very moist, very stiff, laminated.	
40										
	20			S-1	4 11 13				@20': Sandy SILT with trace clay (ML), olive brown to reddish brown, slightly moist, very stiff, very fine grained sand, some mica, minor spotty manganese development.	
35										
	25			S-2	4 11 23				@25': Very moist, hard, micaceous.	
30									Total Depth of Boring: 26.5 feet bgs No Groundwater encountered during drilling Boring backfilled with soil cuttings and capped with cold patch asphalt upon completion of drilling. Excess soil cuttings spread on site.	
	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

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GEOTECHNICAL BORING LOG B-4

Project No.	10646-001	**Date Drilled**	3-11-14
Project	2880 Mesa Verde Dr. East	**Logged By**	Joe Roe
Drilling Co.	Martini Drilling Corporation	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger, 140lb Autohammer, 30" Drop	**Ground Elevation**	58'
Location	See Figure 2 - Boring Location Map	**Sampled By**	Joe Roe

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
	0			BB-1				CL	**Artificial Fill, undocumented: (Afu)** @0': Grass area, Topsoil - 8-inches thick. @8-inches: CLAY with silt and some sand (CL), dark brown, very moist, some fine grained sand and rootlets.	
				R-1	3 6 10				@2': Silty CLAY (CL), dark brown, very moist, stiff, trace fine grained sand, porous, 1 to 3-mm unlined voids, minor gleying of root lined pores.	AL
55								CL	**Quaternary Old Paralic Deposits: (Qopf)** @3': Silty CLAY with sand (CL), medium brown, very moist, stiff, coarse grained sand, minor $CaCO_3$ on poorly developed soil pedogens, LL=38; PL=17; PI=21	
	5			R-2	3 6 10					
				R-3 BB-2	4 10 18				@7': Silty CLAY (CL), olive gray, moist, very stiff, laminated, abundant $CaCO_3$ development with concretions, poorly developed blocky structure.	
50										
	10			R-4	6 12 17				@10': Becomes grayish white to olive grey lean CLAY (CL), color change due to presence of caliche ($CaCO_3$)	CN
45										
	15			S-1	3 8 9			ML	@15': Clayey SILT (ML), olive green, very moist, very stiff, abundant $CaCO_3$, 86% passing No. 200 sieve	-200
40										
	20			R-5	3 14 17			CL	@20': CLAY (CL), olive gray, moist, very stiff, moderately oxidized, $CaCO_3$ development along poorly developed blocky structure, with $CaCO_3$ concretions.	
35										
	25			S-2	8 18 22			SP	@25.5': SAND (SP), light gray brown, dry, dense, very fine grained, poorly graded.	
30										
	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 B-4

Project No.	10646-001	**Date Drilled**	3-11-14
Project	2880 Mesa Verde Dr. East	**Logged By**	Joe Roe
Drilling Co.	Martini Drilling Corporation	**Hole Diameter**	8"
Drilling Method	Hollow Stem Auger, 140lb Autohammer, 30" Drop	**Ground Elevation**	58'
Location	See Figure 2 - Boring Location Map	**Sampled By**	Joe Roe

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
	30			R-6	17 50/6"			SW	@30': SAND (SW), light brown, dry, very dense, fine to coarse grained, well graded, unconsolidated.	
25										
	35			S-3	15 24 42				@35': SAND (SW), light brown, dry, very dense, fine to coarse grained, well graded, unconsolidated.	
20										
	40			R-7	17 50/5"			SW-GW	@40': SAND with gravel (SW-GW), orange brown to yellow brown, dry, very dense, fine to coarse grained sand, fine to coarse gravel, some gravel >3-inches in size and mechanically fractured, well graded.	
15										
	45			S-4	9 17 30			SP-SM	@45': SAND with silt (SP-SM), light gray, dry, dense, very fine grained, micaceous.	
10										
	50			R-8	14 40 50/3"			SP	@50': SAND (SP), light gray, dry, very dense, very fine grained, poorly graded. @51': Becomes coarse grained sand with gravel.	
									Total Depth of Boring: 51.3 feet bgs No Groundwater encountered during drilling Boring backfilled with soil cuttings upon completion of drilling. Excess soil cuttings spread on site.	
5										
	55									
0										
	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

APPENDIX B

Boring No.	B-4							
Sample No.	S-1							
Depth (ft.)	15.0							
Sample Type	SPT							
Soil Identification	Brown silt (ML)							
Moisture Correction								
Wet Weight of Soil + Container (g)	0.00							
Dry Weight of Soil + Container (g)	0.00							
Weight of Container (g)	1.00							
Moisture Content (%)	0.00							
Sample Dry Weight Determination								
Weight of Sample + Container (g)	300.00							
Weight of Container (g)	0.00							
Weight of Dry Sample (g)	300.00							
Container No.:								
After Wash								
Method (A or B)	B							
Dry Weight of Sample + Cont. (g)	41.10							
Weight of Container (g)	0.00							
Dry Weight of Sample (g)	41.10							
% Passing No. 200 Sieve	**86.3**							
% Retained No. 200 Sieve	13.7							

Leighton

PERCENT PASSING
No. 200 SIEVE
ASTM D 1140

Project Name: Pinnacle/Mesa Verde Drive East
Project No.: 10646.001
Client Name: Pinnacle Residential
Tested By: MVH/ACS Date: 03/20/14

-200 B-4, S-1 @ 15

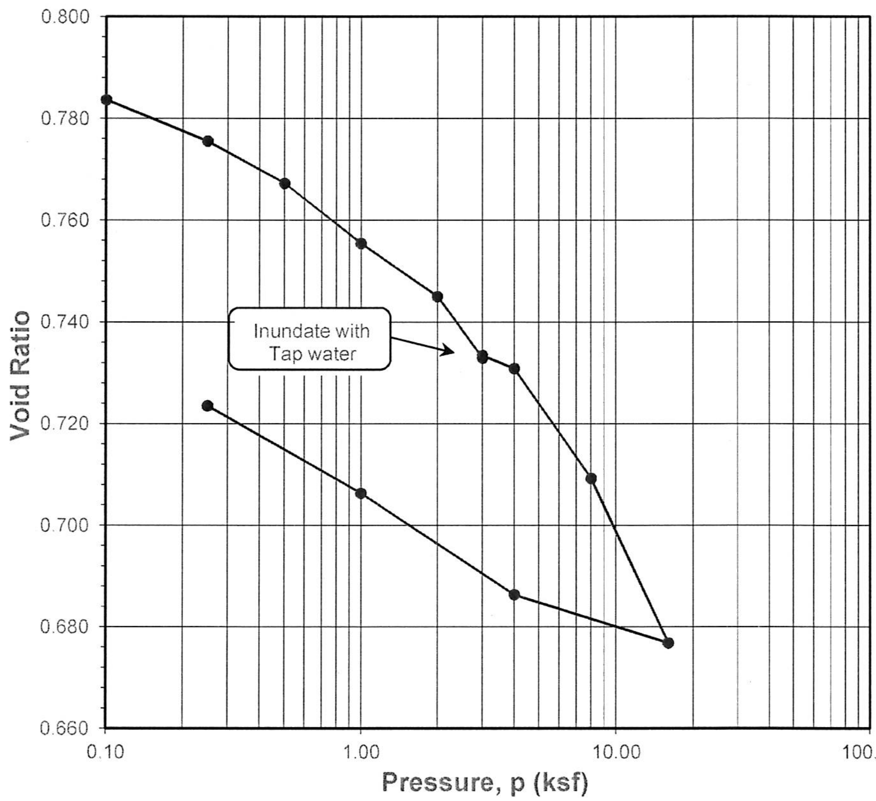
ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Leighton

Project Name:	Pinnacle/Mesa Verde Drive East	Tested By: G. Bathala	Date: 03/17/14
Project No.:	10646.001	Checked By: J. Ward	Date: 03/26/14
Boring No.:	B-4	Depth (ft.): 10.0	
Sample No.:	R4	Sample Type: Ring	
Soil Identification:	Light olive gray lean clay (CL), caliche noted		

Sample Diameter (in.)	2.416
Sample Thickness (in.)	1.000
Wt. of Sample + Ring (g)	188.03
Weight of Ring (g)	46.42
Height after consol. (in.)	0.9659
Before Test	
Wt.Wet Sample+Cont. (g)	229.11
Wt.of Dry Sample+Cont. (g)	191.37
Weight of Container (g)	37.75
Initial Moisture Content (%)	24.6
Initial Dry Density (pcf)	94.5
Initial Saturation (%)	85
Initial Vertical Reading (in.)	0.3345
After Test	
Wt.of Wet Sample+Cont. (g)	248.50
Wt. of Dry Sample+Cont. (g)	219.07
Weight of Container (g)	59.98
Final Moisture Content (%)	26.12
Final Dry Density (pcf)	96.9
Final Saturation (%)	95
Final Vertical Reading (in.)	0.2983
Specific Gravity (assumed)	2.70
Water Density (pcf)	62.43

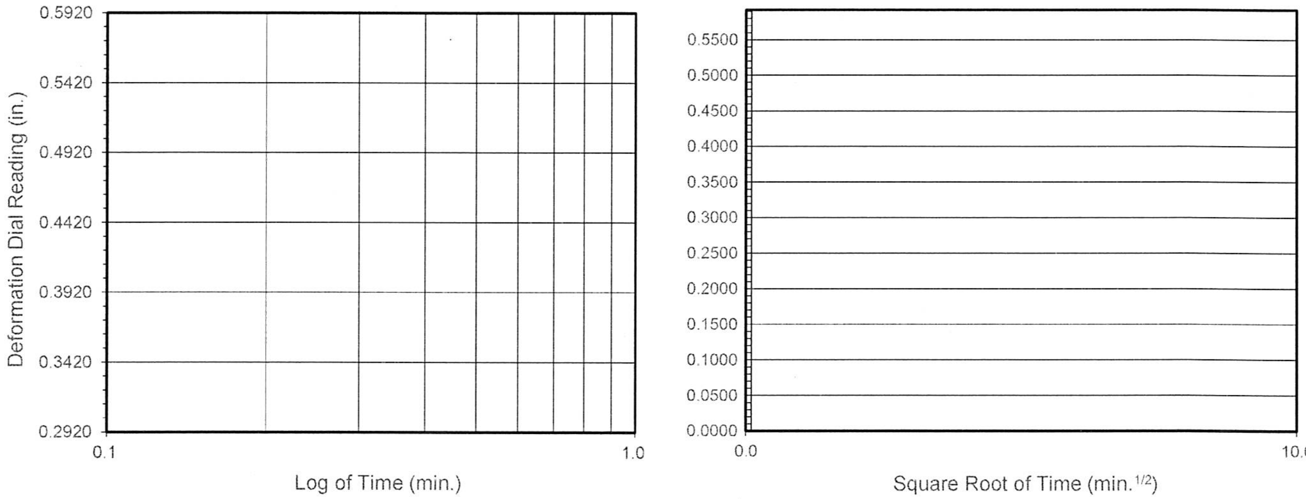


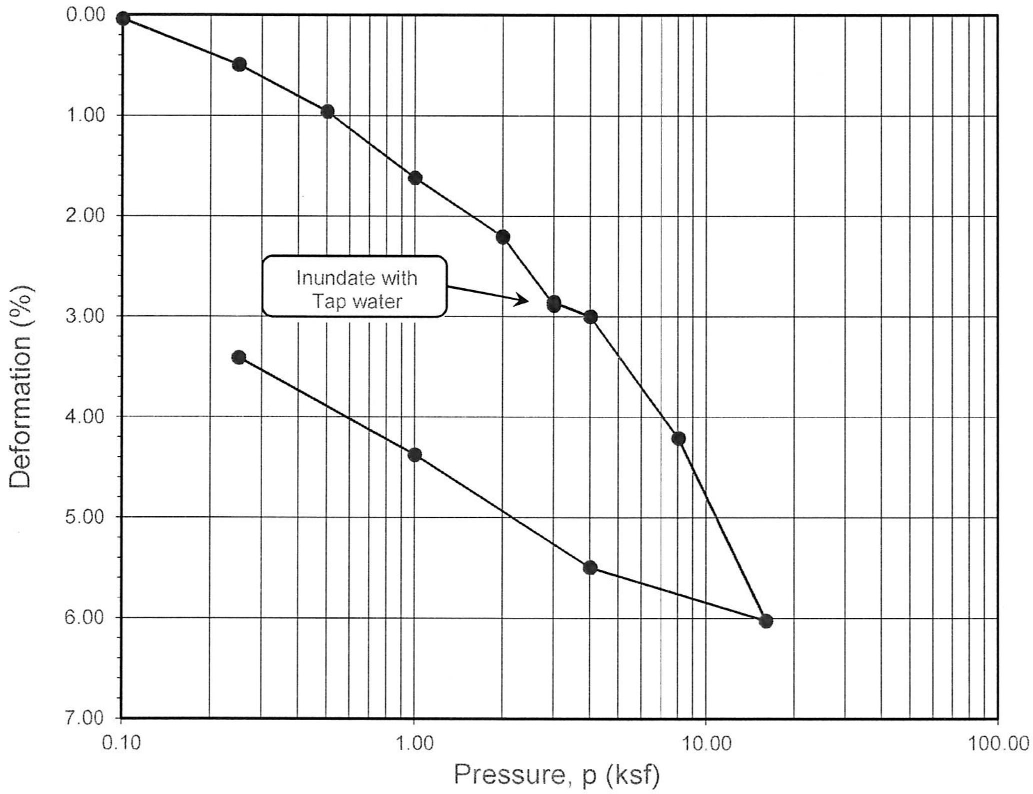
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deforma-tion (%)
0.10	0.3341	0.9996	0.00	0.04	0.784	0.04
0.25	0.3290	0.9945	0.06	0.55	0.776	0.49
0.50	0.3241	0.9896	0.08	1.04	0.767	0.96
1.00	0.3172	0.9827	0.11	1.73	0.755	1.62
2.00	0.3107	0.9762	0.18	2.39	0.745	2.21
3.00	0.3033	0.9688	0.24	3.13	0.733	2.89
3.00	0.3036	0.9691	0.24	3.10	0.733	2.86
4.00	0.3016	0.9671	0.29	3.29	0.731	3.00
8.00	0.2877	0.9532	0.47	4.68	0.709	4.21
16.00	0.2670	0.9325	0.73	6.76	0.677	6.03
4.00	0.2752	0.9407	0.44	5.94	0.686	5.50
1.00	0.2880	0.9535	0.28	4.66	0.706	4.38
0.25	0.2983	0.9638	0.21	3.62	0.724	3.41

No Time Readings

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)

No Time Readings





Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
B-4	**R4**	**10.0**	**24.6**	**26.1**	**94.5**	**96.9**	**0.784**	**0.724**	**85**	**95**

Soil Identification: Light olive gray lean clay (CL), caliche noted

Leighton

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 10646.001

Pinnacle/Mesa Verde Drive East

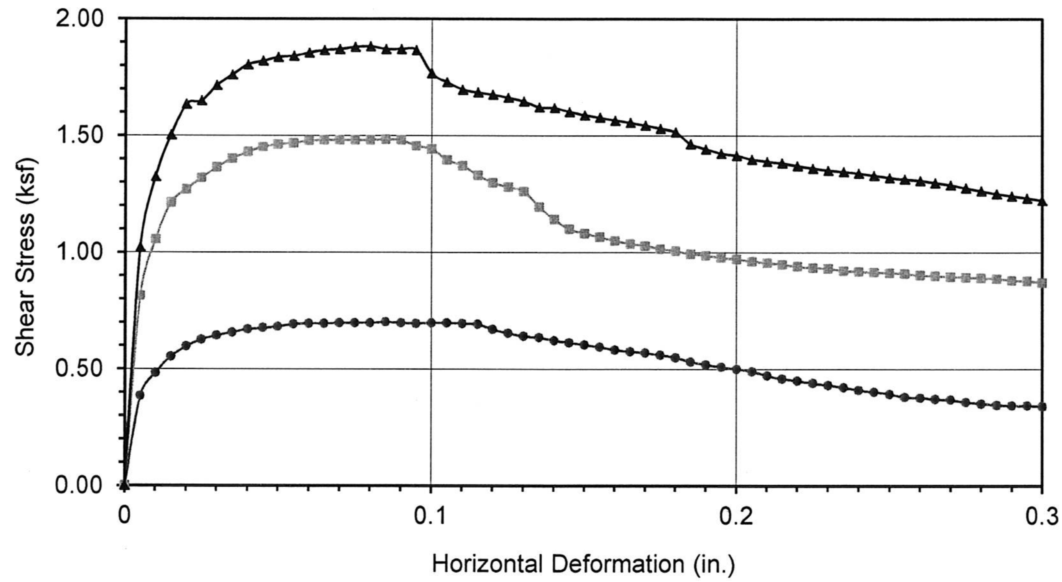
Leighton

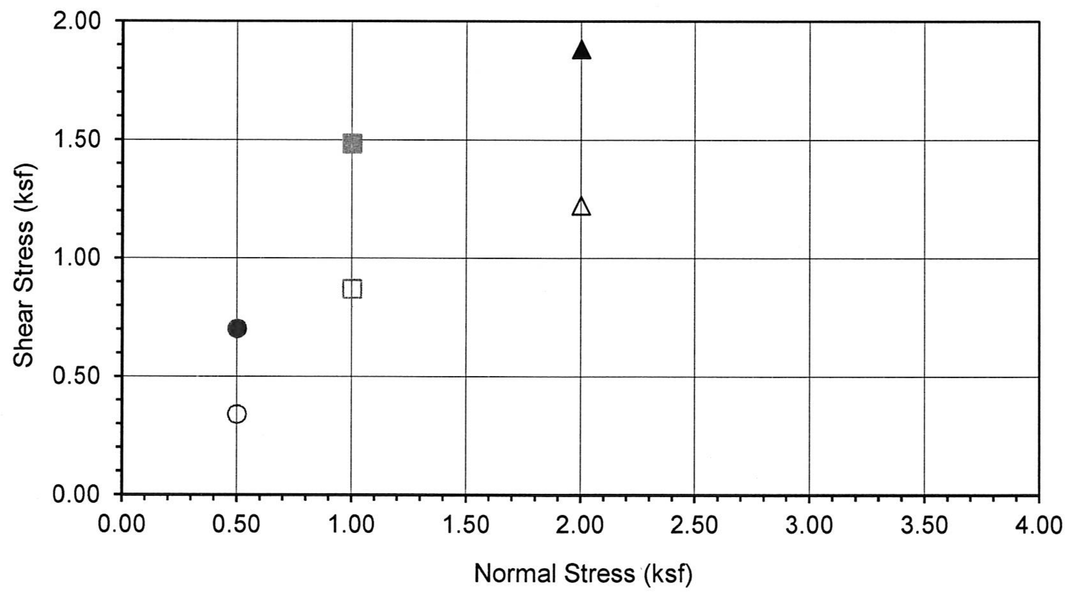
DIRECT SHEAR TEST

Consolidated Undrained

Project Name: Pinnacle/Mesa Verde Drive East   Tested By: G. Bathala   Date: 03/17/14
Project No.: 10646.001   Checked By: J. Ward
Boring No.: B-2   Sample Type: Ring
Sample No.: R2   Depth (ft.): 5.0
Soil Identification: Olive brown lean clay (CL)

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	187.26	187.36	187.69
Weight of Ring(gm):	42.17	43.01	43.98
Before Shearing			
Weight of Wet Sample+Cont.(gm):	111.52	111.52	111.52
Weight of Dry Sample+Cont.(gm):	100.61	100.61	100.61
Weight of Container(gm):	57.74	57.74	57.74
Vertical Rdg.(in): Initial	0.2587	0.2402	0.0000
Vertical Rdg.(in): Final	0.2399	0.2251	0.0005
After Shearing			
Weight of Wet Sample+Cont.(gm):	206.67	218.01	183.56
Weight of Dry Sample+Cont.(gm):	173.92	181.90	148.35
Weight of Container(gm):	58.00	70.37	37.96
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**B-2**
Sample No.	**R2**
Depth (ft)	**5**
Sample Type: Ring	
Soil Identification: Olive brown lean clay (CL)	

Normal Stress (kip/ft²)	0.500	1.000	2.000
Peak Shear Stress (kip/ft²)	● 0.701	■ 1.484	▲ 1.883
Shear Stress @ End of Test (ksf)	○ 0.340	□ 0.871	△ 1.223
Deformation Rate (in./min.)	0.0500	0.0500	0.0500
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	25.45	25.45	25.45
Dry Density (pcf)	96.2	95.7	95.3
Saturation (%)	91.3	90.2	89.3
Soil Height Before Shearing (in.)	1.0188	1.0151	1.0005
Final Moisture Content (%)	28.3	32.4	31.9

Leighton

DIRECT SHEAR TEST RESULTS
Consolidated Undrained

Project No.: 10646.001

Pinnacle/Mesa Verde Drive East

03-14

DS B-2, R2 @ 5

EXPANSION INDEX of SOILS

ASTM D 4829

Leighton

Project Name:	Pinnacle/Mesa Verde Drive East	Tested By:	M. Van Horn	Date:	03/18/14
Project No.:	10646.001	Checked By:	J. Ward	Date:	03/20/14
Boring No.:	B-2	Depth (ft.):	0-5		
Sample No.:	BB-1				
Soil Identification:	Brown clayey sand (SC)				

Dry Wt. of Soil + Cont. (g)	1000.00
Wt. of Container No. (g)	0.00
Dry Wt. of Soil (g)	1000.00
Weight Soil Retained on #4 Sieve	0.00
Percent Passing # 4	100.00

MOLDED SPECIMEN	Before Test	After Test
Specimen Diameter (in.)	4.01	4.01
Specimen Height (in.)	1.0000	1.0340
Wt. Comp. Soil + Mold (g)	587.20	457.70
Wt. of Mold (g)	163.50	0.00
Specific Gravity (Assumed)	2.70	2.70
Container No.	O	O
Wet Wt. of Soil + Cont. (g)	854.90	621.20
Dry Wt. of Soil + Cont. (g)	795.20	557.64
Wt. of Container (g)	0.00	163.50
Moisture Content (%)	7.51	16.13
Wet Density (pcf)	127.8	133.5
Dry Density (pcf)	118.9	115.0
Void Ratio	0.418	0.466
Total Porosity	0.295	0.318
Pore Volume (cc)	61.0	68.1
Degree of Saturation (%) [S_{meas}]	**48.5**	93.4

SPECIMEN INUNDATION in distilled water for the period of 24 h or expansion rate < 0.0002 in./h

Date	Time	Pressure (psi)	Elapsed Time (min.)	Dial Readings (in.)
03/18/14	12:05	1.0	0	0.1230
03/18/14	12:15	1.0	10	0.1220
Add Distilled Water to the Specimen				
03/18/14	13:31	1.0	76	0.1540
03/19/14	7:03	1.0	1128	0.1570
03/19/14	8:03	1.0	1188	0.1570

Expansion Index (EI_{meas}) = ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	**35**

ATTERBERG LIMITS

ASTM D 4318

Leighton

Project Name:	Pinnacle/Mesa Verde Drive East	Tested By:	M. Van Horn	Date:	03/20/14
Project No. :	10646.001	Input By:	J. Ward	Date:	03/21/14
Boring No.:	B-4	Checked By:	J. Ward		
Sample No.:	BB-1	Depth (ft.)	0-5		
Soil Identification:	Olive lean clay (CL)				

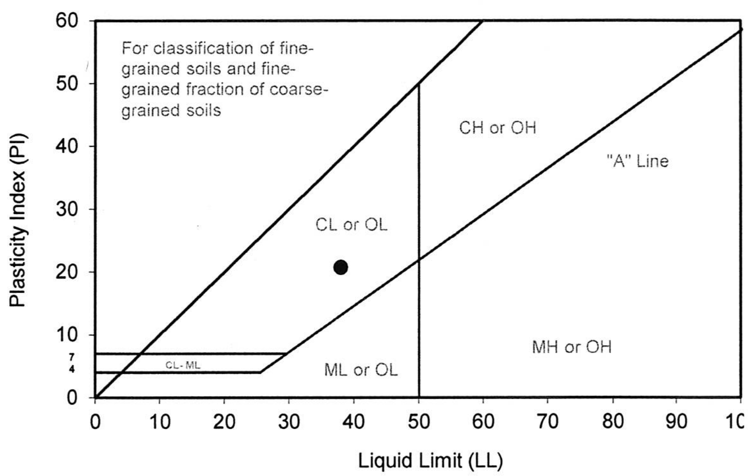
TEST	PLASTIC LIMIT		LIQUID LIMIT			
NO.	1	2	1	2	3	4
Number of Blows [N]			32	25	17	
Wet Wt. of Soil + Cont. (g)	6.52	8.37	13.64	12.86	19.34	
Dry Wt. of Soil + Cont. (g)	5.75	7.26	10.25	9.56	14.11	
Wt. of Container (g)	1.09	1.10	1.09	1.06	1.10	
Moisture Content (%) [Wn]	16.52	18.02	37.01	38.82	40.20	

Liquid Limit	**38**
Plastic Limit	**17**
Plasticity Index	**21**
Classification	**CL**

PI at "A" - Line = 0.73(LL-20) | 13.14

One - Point Liquid Limit Calculation

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



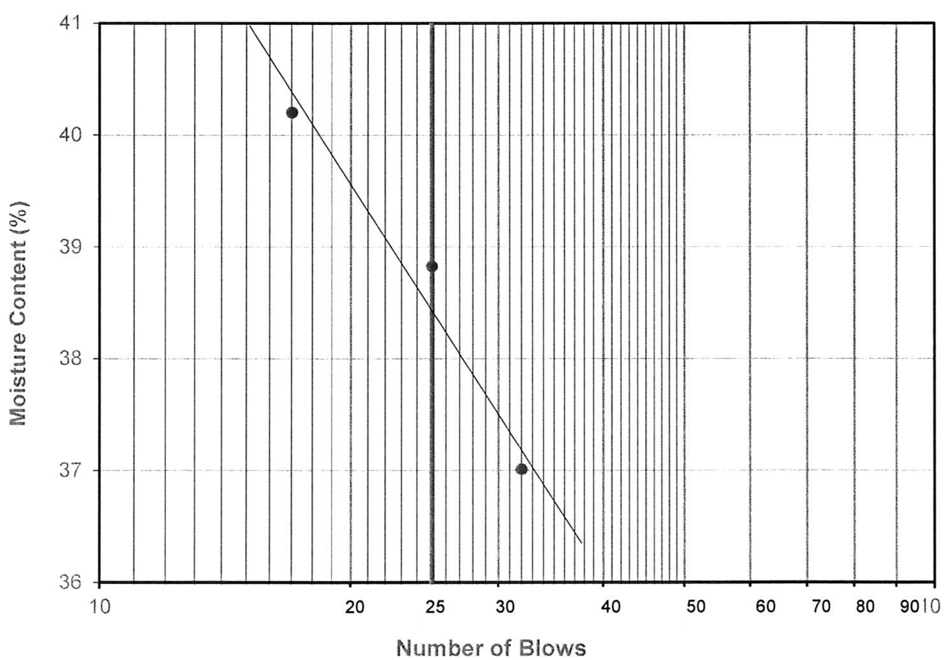
PROCEDURES USED

☐ Wet Preparation
Multipoint - Wet

☒ Dry Preparation
Multipoint - Dry

☒ Procedure A
Multipoint Test

☐ Procedure B
One-point Test



APPENDIX C

APPENDIX C

LEIGHTON AND ASSOCIATES, INC.
EARTHWORK AND GRADING GUIDE SPECIFICATIONS

TABLE OF CONTENTS

C-1.0 GENERAL

C-1.1 Intent

These Earthwork and Grading Guide Specifications are for grading and earthwork shown on the current, approved grading plan(s) and/or indicated in the Leighton and Associates, Inc. geotechnical report(s). These Guide Specifications are a part of the recommendations contained in the geotechnical report(s). In case of conflict, the project-specific recommendations in the geotechnical report shall supersede these Guide Specifications. Leighton and Associates, Inc. shall provide geotechnical observation and testing during earthwork and grading. Based on these observations and tests, Leighton and Associates, Inc. may provide new or revised recommendations that could supersede these specifications or the recommendations in the geotechnical report(s).

C-1.2 Role of Leighton and Associates, Inc.

Prior to commencement of earthwork and grading, Leighton and Associates, Inc. shall meet with the earthwork contractor to review the earthwork contractor's work plan, to schedule sufficient personnel to perform the appropriate level of observation, mapping and compaction testing. During earthwork and grading, Leighton and Associates, Inc. shall observe, map, and document subsurface exposures to verify geotechnical design assumptions. If observed conditions are found to be significantly different than the interpreted assumptions during the design phase, Leighton and Associates, Inc. shall inform the owner, recommend appropriate changes in design to accommodate these observed conditions, and notify the review agency where required. Subsurface areas to be geotechnically observed, mapped, elevations recorded, and/or tested include (1) natural ground after clearing to receiving fill but before fill is placed, (2) bottoms of all "remedial removal" areas, (3) all key bottoms, and (4) benches made on sloping ground to receive fill.

Leighton and Associates, Inc. shall observe moisture-conditioning and processing of the subgrade and fill materials, and perform relative compaction testing of fill to determine the attained relative compaction. Leighton and Associates, Inc. shall provide *Daily Field Reports* to the owner and the Contractor on a routine and frequent basis.

C-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 Guide Specifications prior to commencement of grading. The Contractor shall be solely responsible for performing grading and backfilling in accordance with the current, approved plans and specifications.

The Contractor shall inform the owner and Leighton and Associates, Inc. of changes in work schedules at least one working day in advance of such changes so that appropriate observations and tests can be planned and accomplished. The Contractor shall not assume that Leighton and Associates, Inc. is aware of all grading operations.

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

C-2.0 PREPARATION OF AREAS TO BE FILLED

C-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 Leighton and Associates, Inc. Care should be taken not to encroach upon or otherwise damage native and/or historic trees designated by the Owner or appropriate agencies to remain. Pavements, flatwork or other construction should not extend under the "drip line" of designated trees to remain.

Leighton and Associates, Inc. shall evaluate the extent of these removals depending on specific site conditions. Earth fill material shall not contain more than 3 percent of organic materials (by dry weight: ASTM D 2974-00). 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.

C-2.2 Processing

Existing ground that has been declared satisfactory for support of fill, by Leighton and Associates, Inc., shall be scarified to a minimum depth of 6 inches (15 cm). Existing ground that is not satisfactory shall be over-excavated as specified in the following Section C-2.3. 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.

C-2.3 Overexcavation

In addition to removals and over-excavations 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 over-excavated to competent ground as evaluated by Leighton and Associates, Inc. during grading. All undocumented fill soils under proposed structure footprints should be excavated

C-2.4 Benching

Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical units), (>20 percent grade) the ground shall be stepped or benched. The lowest bench or key shall be a minimum of 15 feet (4.5 m) wide and at least 2 feet (0.6 m) deep, into competent material as evaluated by Leighton and Associates, Inc. Other benches shall be excavated a minimum height of 4 feet (1.2 m) into competent material or as otherwise recommended by Leighton and Associates, Inc. Fill placed on ground sloping flatter than 5:1 (horizontal to vertical units), (<20 percent grade) shall also be benched or otherwise over-excavated to provide a flat subgrade for the fill.

C-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 Leighton and Associates, Inc. as suitable to receive fill. The Contractor shall obtain a written acceptance (*Daily Field Report*) from Leighton and Associates, Inc. prior to fill placement. A licensed surveyor shall provide the survey control for determining elevations of processed areas, keys and benches.

C-3.0 FILL MATERIAL

C-3.1 Fill Quality

Material to be used as fill shall be essentially free of organic matter and other deleterious substances evaluated and accepted by Leighton and Associates, Inc. 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 Leighton and Associates, Inc. or mixed with other soils to achieve satisfactory fill material.

C-3.2 Oversize

Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 6 inches (15 cm), shall not be buried or placed in fill unless location, materials and placement methods are specifically accepted by Leighton and Associates, Inc. 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 feet (3 m) measured vertically from finish grade, or within 2 feet (0.61 m) of future utilities or underground construction.

C-3.3 Import

If importing of fill material is required for grading, proposed import material shall meet the requirements of Section C-3.1, and be free of hazardous materials ("contaminants") and rock larger than 3-inches (8 cm) in largest dimension. All import soils shall have an Expansion Index (EI) of 20 or less and a sulfate content no greater than (≤) 500 parts-per-million (ppm). A representative sample of a potential import source shall be given to Leighton and Associates, Inc. at least four full working days before importing begins, so that suitability of this import material can be determined and appropriate tests performed.

C-4.0 FILL PLACEMENT AND COMPACTION

C-4.1 Fill Layers

Approved fill material shall be placed in areas prepared to receive fill, as described in Section C-2.0, above, in near-horizontal layers not exceeding 8 inches (20 cm) in loose thickness. Leighton and Associates, Inc. may accept thicker layers if testing indicates the grading procedures can adequately compact the thicker layers, and only if the building officials with the appropriate jurisdiction approve. Each layer shall be spread evenly and mixed thoroughly to attain relative uniformity of material and moisture throughout.

C-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 D 1557-09.

C-4.3 Compaction of Fill

After each layer has been moisture-conditioned, mixed, and evenly spread, each layer shall be uniformly compacted to not-less-than (≥) 90 percent of the maximum dry density as determined by ASTM Test Method D 1557-09. In some cases, structural fill may be specified (see project-specific geotechnical report) to be uniformly compacted to at-least (≥) 95 percent of the ASTM D 1557-09 modified Proctor laboratory maximum dry density. For fills thicker than (>) 15 feet (4.5 m), the portion of fill deeper than 15 feet below proposed finish grade shall be compacted to 95 percent of the ASTM D 1557-09 laboratory maximum density. 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.

C-4.4 Compaction of Fill Slopes

In addition to normal compaction procedures specified above, compaction of slopes shall be accomplished by back rolling of slopes with sheepsfoot rollers at increments of 3 to 4 feet (1 to 1.2 m) in fill elevation, or by other methods producing satisfactory results acceptable to Leighton and Associates, Inc.. Upon completion of grading, relative compaction of the fill, out to the slope face, shall be at least 90 percent of the ASTM D 1557-09 laboratory maximum density.

C-4.5 Compaction Testing

Field-tests for moisture content and relative compaction of the fill soils shall be performed by Leighton and Associates, Inc. Location and frequency of tests shall be at our field representative(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).

C-4.6 Compaction Test Locations

Leighton and Associates, Inc. shall document the approximate elevation and horizontal coordinates of each density test location. The Contractor shall coordinate with the project surveyor to assure that sufficient grade stakes are established so that Leighton and Associates, Inc. can determine the test locations with sufficient accuracy. Adequate grade stakes shall be provided.

C-5.0 EXCAVATION

Excavations, as well as over-excavation for remedial purposes, shall be evaluated by Leighton and Associates, Inc. during grading. Remedial removal depths shown on geotechnical plans are estimates only. The actual extent of removal shall be determined by Leighton and Associates, Inc. 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, then observed and reviewed by Leighton and Associates, Inc. prior to placement of materials for construction of the fill portion of the slope, unless otherwise recommended by Leighton and Associates, Inc.

C-6.0 TRENCH BACKFILLS

C-6.1 Safety

The Contractor shall follow all OSHA and Cal/OSHA requirements for safety of trench excavations. Work should be performed in accordance with Article 6 of the *California Construction Safety Orders*, 2003 Edition or more current (see also: http://www.dir.ca.gov/title8/sb4a6.html).

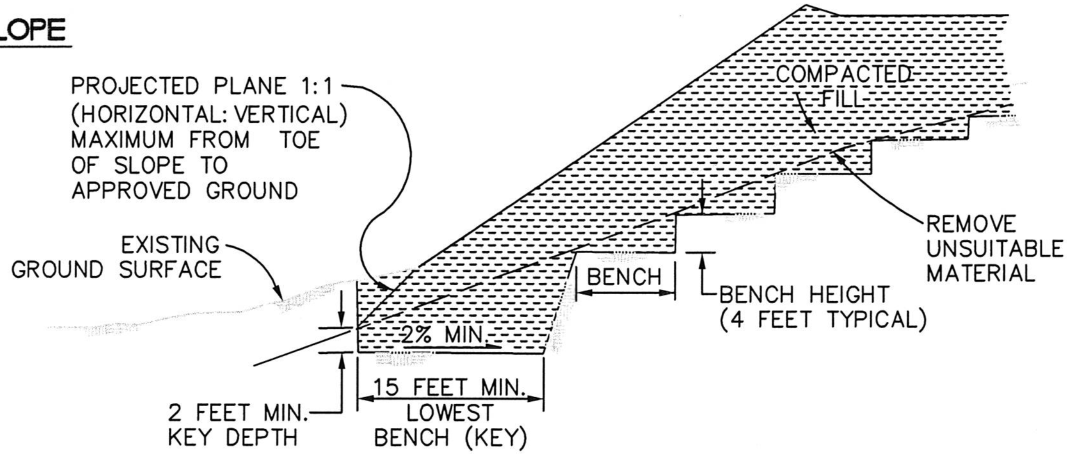
C-6.2 Bedding and Backfill

All utility trench bedding and backfill shall be performed in accordance with applicable provisions of the 2012 Edition of the *Standard Specifications for Public Works Construction* (Green Book). Bedding material shall have a Sand Equivalent greater than 30 (SE>30). Bedding shall be placed to 1-foot (0.3 m) over the top of the conduit, and densified by jetting in areas of granular soils, if allowed by the permitting agency. Otherwise, the pipe-bedding zone should be backfilled with Controlled Low Strength Material (CLSM) consisting of at least one sack of Portland cement per cubic-yard of sand, and conforming to Section 201-6 of the 2012 Edition of the *Standard Specifications for Public Works Construction* (Green Book). Backfill over the bedding zone shall be placed and densified mechanically to a minimum of 90 percent of relative compaction (ASTM D 1557-09) from 1 foot (0.3 m) above the top of the conduit to the surface. Backfill above the pipe zone shall **not** be jetted. Jetting of the bedding around the conduits shall be observed by Leighton and Associates, Inc. and backfill above the pipe zone (bedding) shall be observed and tested by Leighton and Associates, Inc.

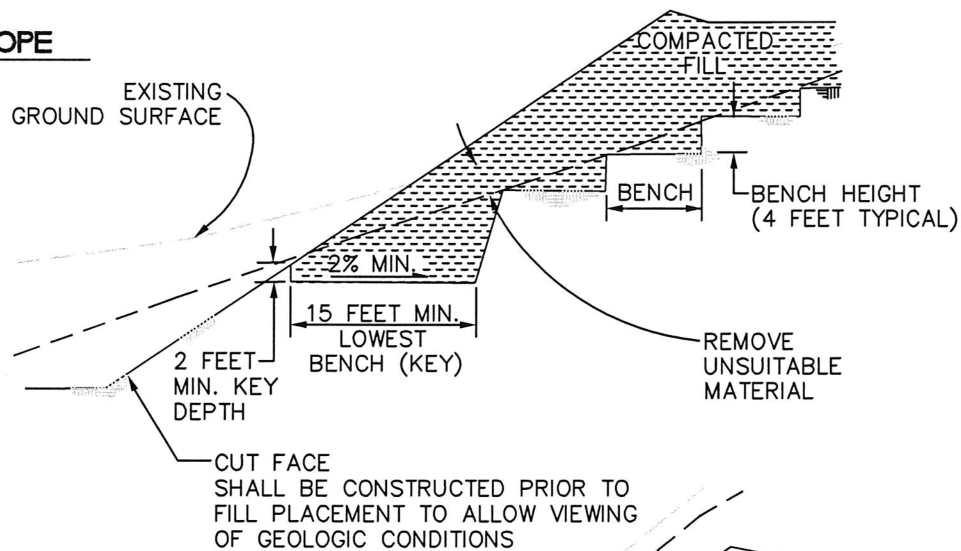
C-6.3 Lift Thickness

Lift thickness of trench backfill shall not exceed those allowed in the Standard Specifications of Public Works Construction unless the Contractor can demonstrate to Leighton and Associates, Inc. that the fill lift can be compacted to the minimum relative compaction by his alternative equipment and method, and only if the building officials with the appropriate jurisdiction approve.

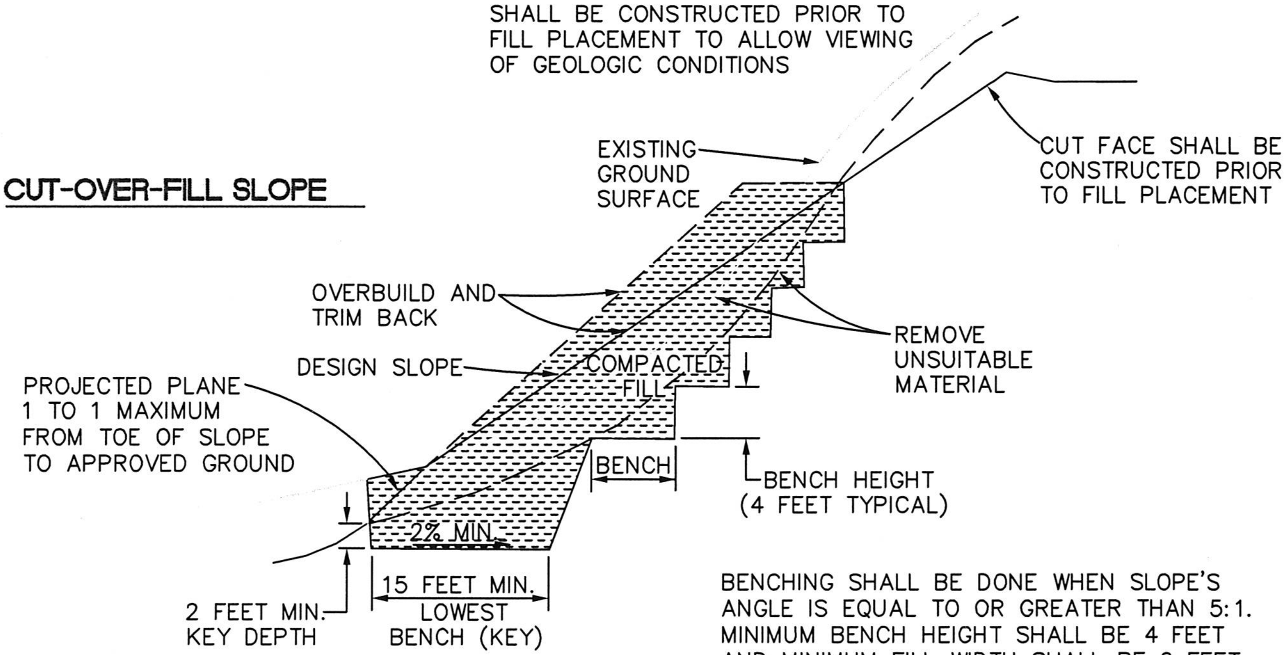
FILL SLOPE



FILL-OVER-CUT SLOPE

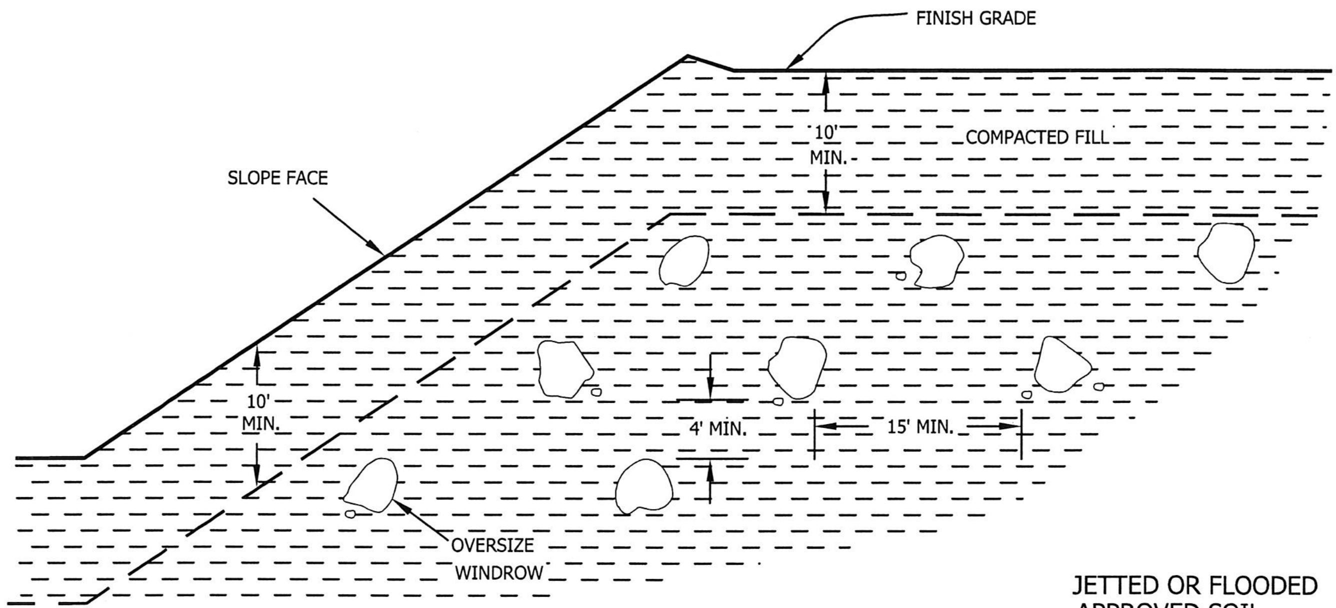


CUT-OVER-FILL SLOPE

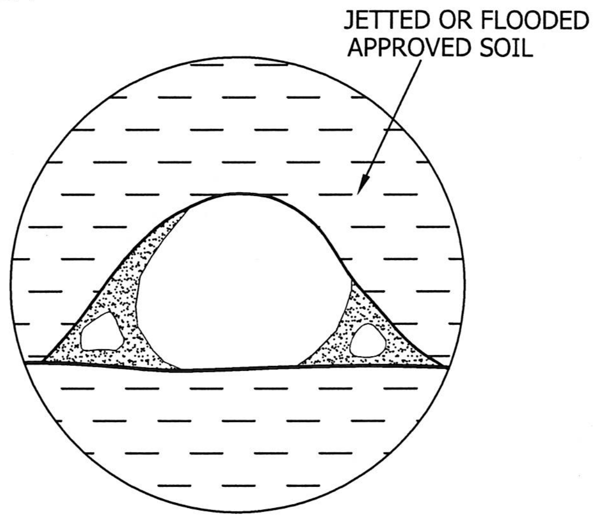


BENCHING SHALL BE DONE WHEN SLOPE'S ANGLE IS EQUAL TO OR GREATER THAN 5:1. MINIMUM BENCH HEIGHT SHALL BE 4 FEET AND MINIMUM FILL WIDTH SHALL BE 9 FEET.

KEYING AND BENCHING	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS A	Leighton

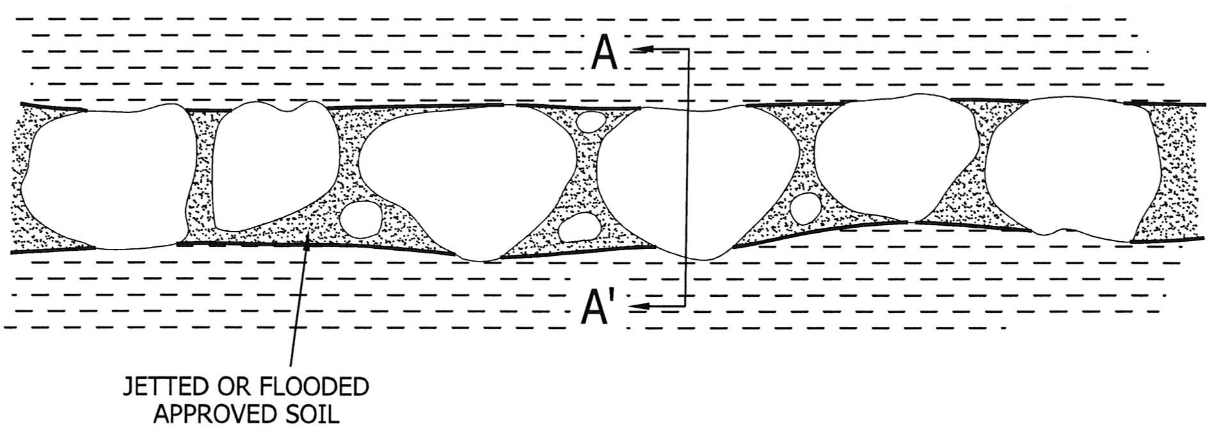


- Oversize rock is larger than 8 inches in largest dimension.
- Backfill with approved soil jetted or flooded in place to fill all the voids.
- Do not bury rock within 10 feet of finish grade.
- Windrow of buried rock shall be parallel to the finished slope face.

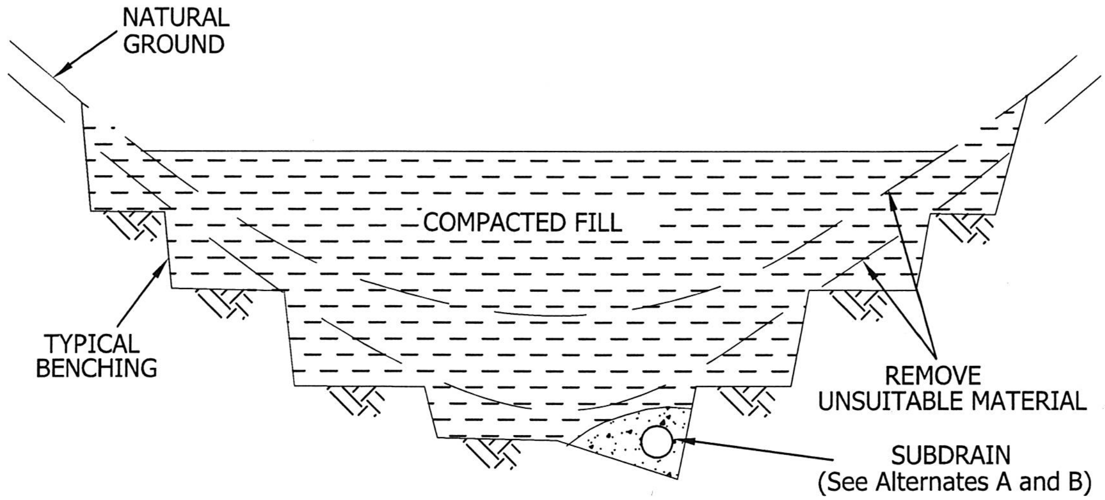


SECTION A-A'

PROFILE ALONG WINDROW

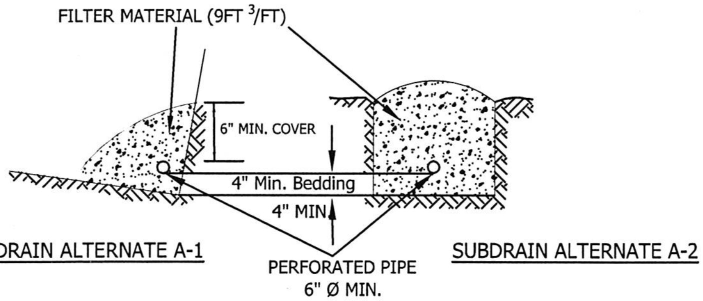


OVERSIZE ROCK DISPOSAL	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS B	Leighton



SUBDRAIN ALTERNATE A

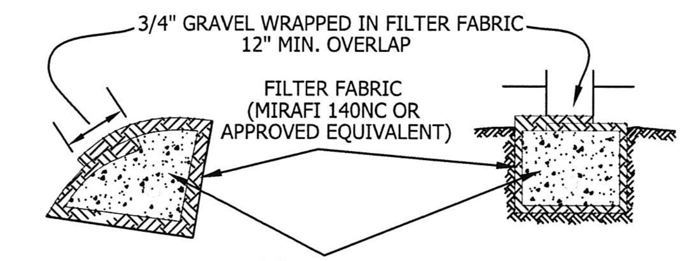
PERFORATED PIPE SURROUNDED
WITH FILTER MATERIAL



FILTER MATERIAL
FILTER MATERIAL SHALL BE CLASS 2 PERMEABLE MATERIAL PER STATE OF CALIFORNIA STANDARD SPECIFICATION, OR APPROVED ALTERNATE.
CLASS 2 GRADING AS FOLLOWS:

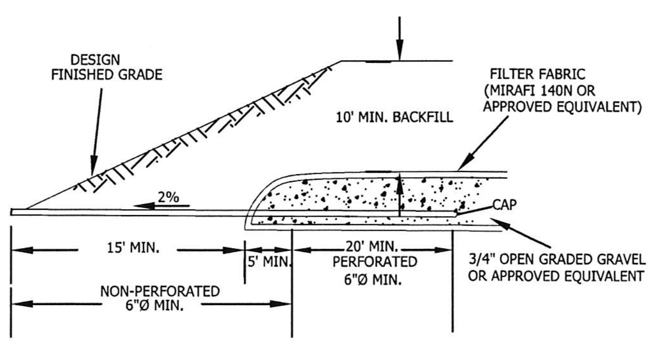
Sieve Size	Percent Passing
1"	100
3/4"	90-100
3/8"	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

SUBDRAIN ALTERNATE B



PERFORATED PIPE IS OPTIONAL PER GOVERNING AGENCY'S REQUIREMENTS

DETAIL OF CANYON SUBDRAIN TERMINAL



CANYON SUBDRAIN	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS C	Leighton

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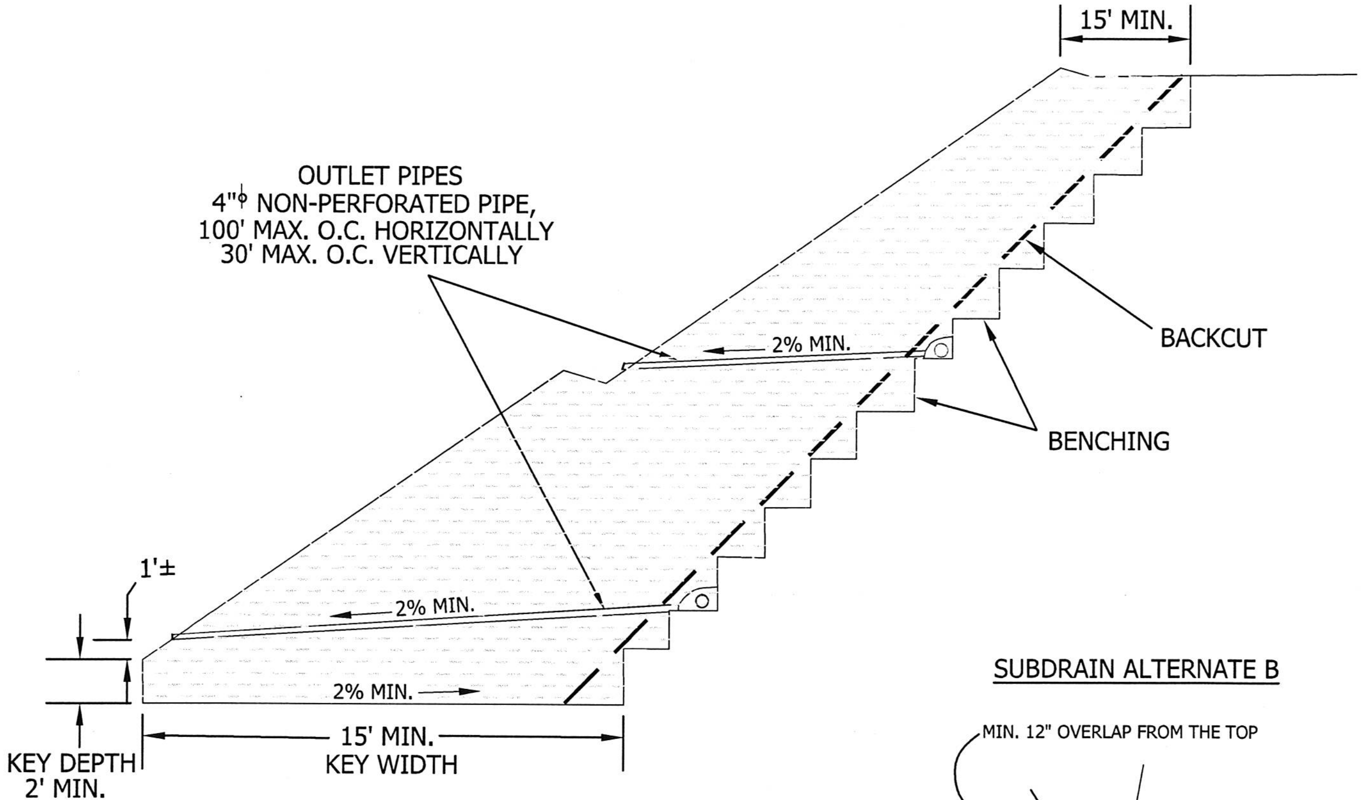
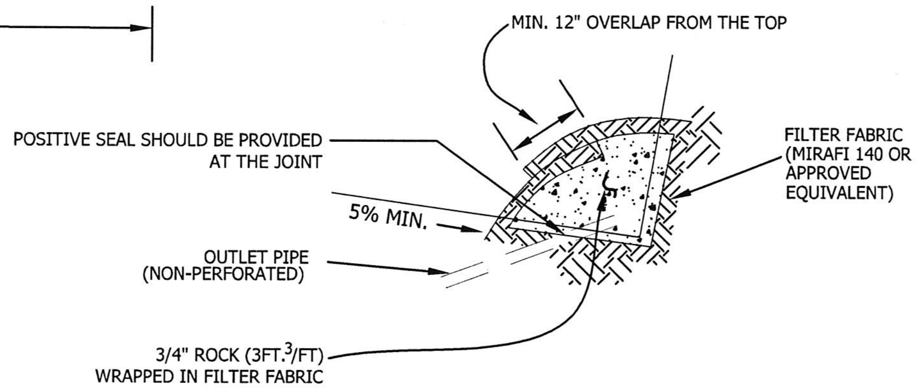
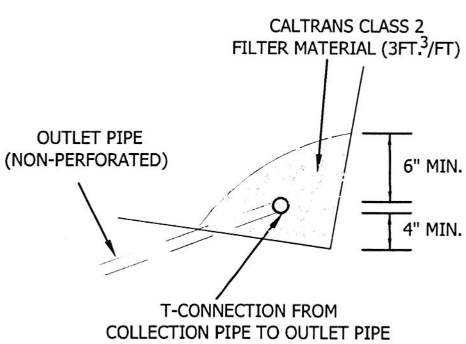


SUBDRAIN ALTERNATE B



SUBDRAIN ALTERNATE A



- SUBDRAIN INSTALLATION - Subdrain collector pipe shall be installed with perforations down or, unless otherwise designated by the geotechnical consultant. Outlet pipes shall be non-perforated pipe. The subdrain pipe shall have at least 8 perforations uniformly spaced per foot. Perforation shall be 1/4" to 1/2" if drilled holes are used. All subdrain pipes shall have a gradient at least 2% towards the outlet.
- SUBDRAIN PIPE - Subdrain pipe shall be ASTM D2751, ASTM D1527 (Schedule 40) or SDR 23.5 ABS pipe or ASTM D3034 (Schedule 40) or SDR 23.5 PVC pipe.
- All outlet pipe shall be placed in a trench and, after fill is placed above it, rodded to verify integrity.

BUTTRESS OR REPLACEMENT FILL SUBDRAINS	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS D	Leighton

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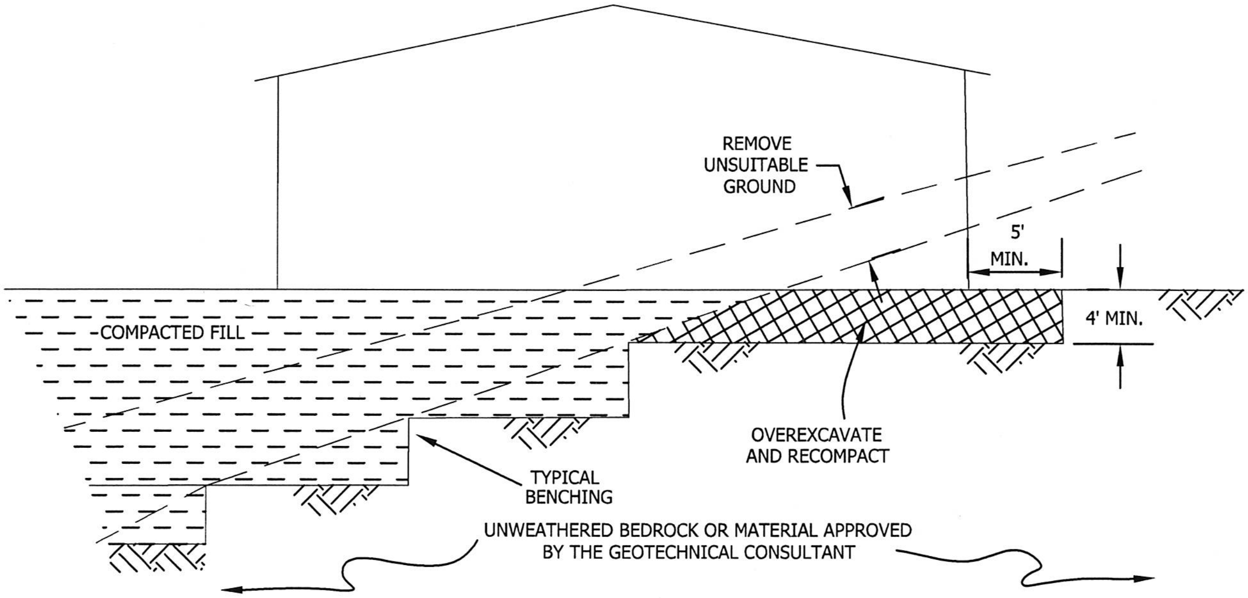
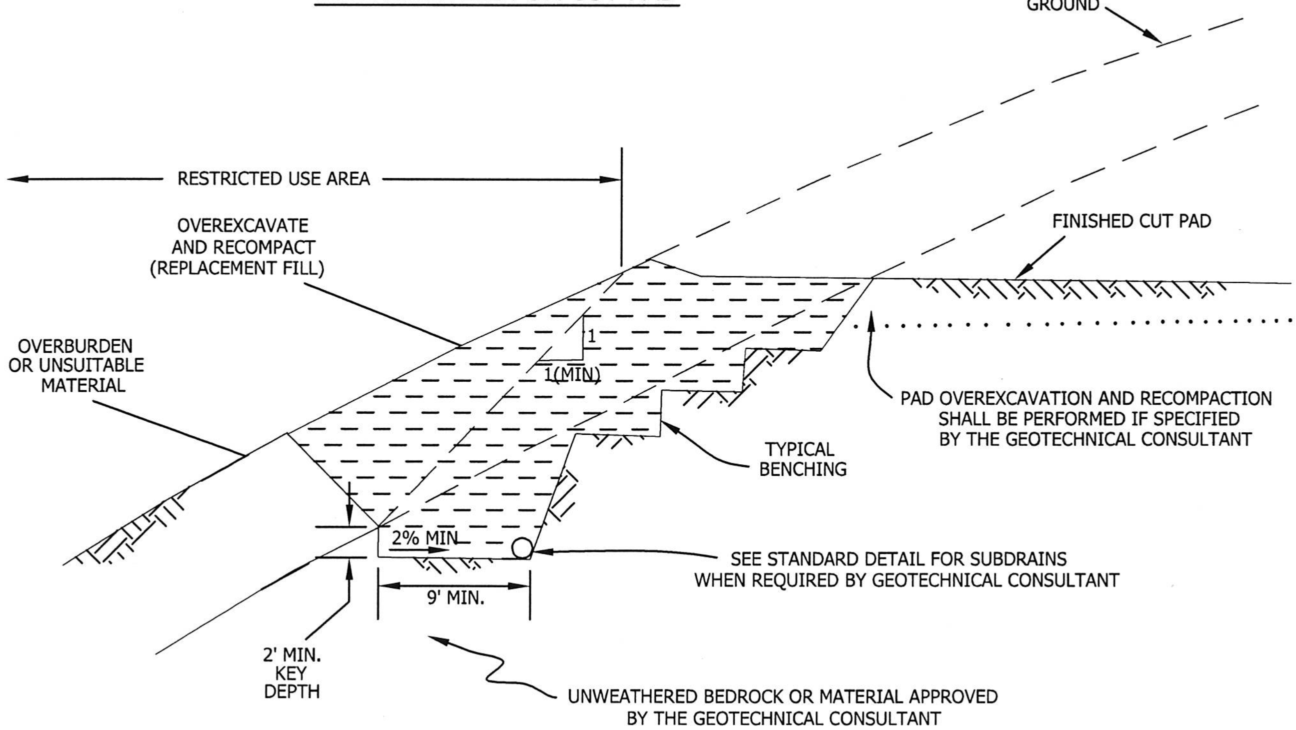
CUT-FILL TRANSITION LOT OVEREXCAVATION



SIDE HILL FILL FOR CUT PAD



TRANSITION LOT FILLS AND SIDE HILL FILLS	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS E	Leighton

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