Leighton and Associates, Inc.
A LEIGHTON GROUP COMPANY

TRANSMITTAL

To: Highland Fairview Properties, LLC
14225 Corporate Way
Moreno Valley, California 92553

Date: September 29, 2014

Project No. 111061-126

Attention: Mr. Wayne Peterson, Vice President of Community Planning

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Subject: Preliminary Geotechnical Evaluation for Environmental Impact Report, World Logistics Center Specific Plan, South of Highway 60 between Redlands Boulevard and Gilman Springs Road, City of Moreno Valley, California

LEIGHTON AND ASSOCIATES, INC.

By Simon Saiid/Robert F. Riha

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PRELIMINARY GEOTECHNICAL EVALUATION FOR ENVIRONMENTAL IMPACT REPORT WORLD LOGISTICS CENTER SPECIFIC PLAN SOUTH OF HIGHWAY 60 BETWEEN REDLANDS BOULEVARD AND GILMAN SPRINGS ROAD CITY OF MORENO VALLEY, CALIFORNIA

Prepared For:

Highland Fairview Properties, LLC
14225 Corporate Way
Moreno Valley, California 92553

Project No. 111061-126

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Attention: Mr. Wayne Peterson, Vice President of Community Planning

Subject: Preliminary Geotechnical Evaluation for Environmental Impact Report, "World Logistics Center Specific Plan", South of SR 60 between Redlands Boulevard and Gilman Springs Road, City of Moreno Valley, California

In accordance with your request and authorization, we are pleased to present this updated preliminary geotechnical evaluation for the "World Logistics Center" Specific Plan area located in the City of Moreno Valley, California. This revised report was prepared to incorporate review comments by City of Moreno Valley (See References), revised boundary and 2013 CBC parameters. It is our opinion that the subject property is considered suitable for proposed development provided our recommendations included in this report are implemented during the various phases of site development. However, it should be anticipated that additional geotechnical studies may be required based on future site development plans to determine limits of fault zones in areas not previously explored, confirm structural setbacks from known fault zones, and verify/provide specific remedial earthwork and foundation design recommendations for individual buildings/lots.

If you have any questions regarding this report, please do not hesitate to contact this office. We appreciate this opportunity to be of service.

Respectfully submitted,

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1.0 INTRODUCTION

1.1 Purpose and Scope

This preliminary geotechnical evaluation report presents the results of our review of the geotechnical constraints and opportunities associated with the World Logistics Center Specific Plan area generally located south of Highway 60 with Redlands Boulevard forming the western boundary and Gilman Springs the eastern boundary in the City of Moreno Valley, California (see Figure 1). The purpose of this report was to update our previous geotechnical and geologic findings and preliminary recommendations based on the revised specific plan area (Highland Fairview, 2014). Additional site specific geologic and geotechnical studies may be needed when site development plans are prepared. Our scope of services for this report included the following:

- Review of provided site development plans, existing soils reports and relevant geologic maps (see References).
- A site reconnaissance on June 6, 2011 and February 7, 2012.
- Preparation of this illustrated report (6 copies) summarizing our findings, conclusions and recommendations.

1.2 Previous Studies

Leighton previously conducted several geotechnical and site specific fault studies for the overall specific plan area and adjacent sites (see References). These studies included a preliminary geotechnical investigation (Leighton, 2007a), supplemental fault investigation (Leighton, 2007b, 2012) and updated geotechnical report (Leighton, 2008c). A supplemental fault study is currently ongoing within the property between Theodore Street and Gilman Springs Road, north of Alessandro Road. Pertinent information and findings provided in these studies are referred to as necessary or included herein for ease of reference. In case of conflict, this report should supersede the conclusions and recommendations of previous referenced reports.

1.3 Site Description

The site is located in the eastern portion of the City of Moreno Valley, with a small portion located within an unincorporated area of Riverside County. The property is generally bounded by Gilman Springs Road on the east, Redlands

Boulevard on the west, SR-60/Eucalyptus Avenue on the north, and the San Jacinto Wildlife preserve on the south as depicted on the *Site Location Map* (Figure 1). The majority of the property has been utilized for various agricultural and farming purposes. Most roads traversing the site are paved; however, a few "dirt" roads, residential buildings and a San Diego Gas and Electric natural gas compressor station were visible during our site reconnaissance.

Gas lines from three major gas companies, namely Southern California Gas (SCG), San Diego Gas & Electric and Questar Line 90 transect the subject site. The SCG gas line is parallel to Cottonwood Avenue near Redlands Boulevard in the west, and parallel to Alessandro Boulevard near Gilman Springs Road in the east. This same gas line is also parallel to Virginia Road from Alessandro Boulevard in the north to the San Jacinto Wildlife area in the south. The gas line from Questar Line 90 is parallel to and about 600 feet south of Alessandro Boulevard up to Theodore Street, and is then along Alessandro Boulevard over to Gilman Springs Road. A water line also traverses the site, west to east along an easterly projection of Dracaea Avenue.

Topographically, the site generally slopes in a southeasterly direction towards the offsite historic Mystic Lake. The prominent Mount Russell is located near the southwest corner of the property with a high point elevation onsite of about 2,500 feet (msl). The maximum relief of site elevations between north and south sides is estimated to be 350 feet. Based on review of available maps, site elevations range from approximately 1,430 feet above mean sea level (msl) at the southeastern corner to approximately 1,780 feet (msl) at several prominent ridges along SR-60 near the off ramp to Gilman Springs Road. Four (4) distinct unnamed drainage channels transect the site in a general north-south direction. The easterly slopes of Mount Russell lie adjacent and partially within the southwest corner of the property. Vegetation on site generally consists of a minimal growth of annual weeds and grasses as well as selected dry farming crops. A few trees and local moderately thick native scrub vegetation exist near and within the existing drainages and non-farmed properties.

Access towers and irrigation pipelines were observed throughout the site, which are possibly related to past agricultural activities. Overhead and underground utility lines are present along the perimeter boundaries. The property north of Eucalyptus Ave. and south of SR-60 contains the Metropolitan Water District Inland Feeder Pipeline. A portion of this property was used as a staging area during construction of the Inland Feeder Riverside Badlands Tunnel from

October 1998 through July 2001 (Arabashi et al, 2003). The tunnel entry point is referred to as the Gilman Portal and contains a below ground outlet structure, associated access roads and fencing. A considerable stockpile of various soils, gravels and rock fragments is located west of the existing northwest to southeast drainage and Theodore Street. These materials are assumed to be spoils from the Inland Feeder tunnel and pipeline alignment construction.

Although no other significant structures were observed, some isolated minor structures and grading activities associated with past agricultural use should be expected. Such structures may include buried remnants of foundations, agricultural debris, pavements, and septic systems. Stockpiles of demolition debris and concrete rubble were also locally observed. Based on field observations, the adjacent properties are primarily used for residential and agricultural purposes and have similar topography.

1.4 Proposed Development

Based on information provided, we understand that cut and fill grading up to 50 feet may be required to create pads for commercial and industrial developments. It is also our understanding that large buildings/distribution centers are expected to consist of concrete tilt-up walls, a steel truss roof structures founded on conventional isolated pad and continuous perimeter wall footings with a slab on grade floor systems. Although structural loads are not known to us at this time, typical column loads are expected to range up to 125 kips and perimeter bearing wall loads are to range up to 6 kips per lineal foot.

The preliminary grading and foundation design recommendations presented in this report are for planning purposes and additional site investigations should be performed once site grading and foundation plans become available for individual building sites.

2.0 FIELD EXPLORATIONS AND LABORATORY TESTING

2.1 Previous Field Explorations

Our previous field explorations conducted for this site are as follows:

- Forty nine (49) exploratory borings (B-6 through B-54) were completed on November 12, 2007 through January 23, 2008. Results of field and laboratory testing are presented in our referenced report (Leighton, 2008c).
- Eighty (80) exploratory test pits (TP-1 through TP-80) were excavated on November 2007. Results of field and laboratory testing are presented in our referenced report (Leighton, 2007b).
- Seven (7) cone penetration tests (CPT-1 through CPT-7) extending to a maximum depth of approximately 80.5 feet below the existing ground surface were conducted in 2006. Detailed results including CPT logs are presented in our referenced report (Leighton, 2006).
- Subsurface fault studies were also performed by Leighton (2007a, 2004, 2012) which included twelve (12) fault trenches with a total length of 16,118 lineal feet (3.05 miles) ranging in depth from 18 to 40 feet.

Approximate locations of previous geotechnical borings, test pits, CPT soundings, and fault trenches are depicted on the *Field Exploration Map*, Plate 1. Work performed by others are also referenced in this report and shown on Plate 1.

2.2 Previous Laboratory Testing

Laboratory testing performed during our previous studies on representative samples included collapse and/or swell potential, expansion index, direct shear, gradation, in-situ moisture and density, maximum dry density and optimum moisture content, R-value, and soluble sulfate concentration, pH, chloride concentration and resistivity. As a part of this geotechnical evaluation, we reviewed pertinent laboratory testing performed during these previous investigations.

3.0 SUMMARY OF GEOTECHNICAL CONDITIONS

3.1 Regional Geology

The World Logistics Center project is located within the Peninsular Ranges Province, which is characterized by northwest trending elongated mountain ranges and valleys. The Peninsular Ranges Province is divided into three major fault bounded tectonic blocks within the San Andreas Fault System, which consist of (from west to east): Santa Ana, Perris, and San Jacinto Blocks. The site is situated near the northeastern boundary of the relatively stable Perris Block.

The Perris Block, approximately 20 miles by 50 miles in extent, is bounded by the San Jacinto Fault Zone to the northeast, the Elsinore Fault Zone to the southwest, the Cucamonga Fault Zone to the northwest and to the southeast by the poorly defined boundary of the Temecula Basin. The Perris Block has had a complex tectonic history in response to movement on the Elsinore and San Jacinto Fault Zones. Thin sedimentary and volcanic materials locally mantle the crystalline bedrock. Alluvial and colluvial deposits fill the lower valley areas.

The study area is located within the northern portion of the San Jacinto Valley, a fault-bounded tectonic basin that has evolved from movement along the San Jacinto fault system resulting in a down-dropped northwest-trending trough. The elongate transtensional basin is believed to have formed as a result of a right step of the fault zone between the Casa Loma and Claremont strands of the fault zone (Morton and Matti, 1993).

As mapped by the USGS (2006), the natural geologic units on the subject property include; various young alluvial deposits, older alluvial-fan deposits, landslide deposits, San Timoteo formation, and granitic bedrock, see the *Regional Geologic Map*, (Figure 2).

3.2 Site Geologic Units

The earth materials encountered or observed on site consist of undocumented fill, topsoil, alluvium, landslide materials, San Timoteo Formation and granitic bedrock. These units are discussed in the following sections in order of increasing age.

3.2.1 Undocumented Fill

Undocumented fills are related to grading activities to construct onsite access roads, Inland Feeder Pipeline, stockpiled fill materials, perimeter unimproved dirt roads, site agricultural leveling (drainage ditch backfill) and exploratory fault trenches (Leighton, 2004). The approximate location of the fault trenches are depicted on Plate 1. All undocumented fills are considered unsuitable for support of additional fill or structures or other planned improvements in their present condition. In areas where the planned grading will not remove the undocumented fill soils, they should be removed and recompacted in accordance with Section 6.1 of this report. Undocumented fill materials cleared of debris and significant organic materials are suitable for use as compacted fills. It should be noted that the most recent previous fault trenches were backfilled with compacted fill (Leighton, 2007a).

3.2.2 Topsoil

Topsoil is mantling the majority of the site (Leighton, 2007a). The thickness of the topsoil was generally uniform, and limited to the upper 12 to 18-inches of the existing ground surface. Topsoil generally consisted of yellow brown, silty sand, silty clay to sandy clay with scattered gravel sized clasts. There are abundant in-filled animal burrows (Krotovina) containing finer silt and sands. Topsoil includes recently tilled soil. Topsoil cleared of debris and significant organic materials is suitable for use as compacted fill.

3.2.3 Alluvium

Alluvial soil was encountered in all exploratory borings, fault trenches, and test pits excavated at the site. The alluvial soils were deposited as part of a complex depositional environment and generally include interbedded fine sands and silts with varying amounts of clay. The yellow-brown to medium gray recent alluvial soils (younger alluvium) are found in drainages and believed to constitute the upper surficial materials (upper 3 to 10 feet). The deeper materials (older alluvium and older fan-deposits) are generally dark yellow-brown to dark gray and consist of silty fine sand to sandy silt with interbedded lenses of silt clay. The alluvium along the southeastern side of the site is significantly denser and contains considerable amounts of coarser sands and gravel. Pertinent engineering characteristics of the encountered alluvium are summarized below:

- ***Compressibility Characteristics***: The alluvium is generally loose in the upper 10 to 15 feet in most areas with an average N-value on the order of 4 to 10. At depths greater than 15 feet, the alluvium is generally medium dense with an average N-value on the order of 10 to 30. Based on the results of our laboratory testing on representative samples, the coefficient of compressibility (Cc) is expected to vary significantly depending on depth and location. Typical Cc value ranging from 0.01 to 0.10 is calculated based on applied loads of 1 to 2 kips. However, higher Cc (0.15 to 0.25) can be encountered in localized areas such as in the vicinity of Boring B-14 at a depth of 25 feet. The results of our testing also indicate a high rebound potential during unloading for some of the tested alluvium. This rebound affect may cause some elevation rise in areas of significant excavation.
- ***Expansion Potential***: The results of the laboratory testing on representative samples indicate that the Expansion Index (EI) of the onsite soils is expected to vary from one location to another. The results indicate the presence of very low to high expansive soils (0<EI<131) in accordance with the classification of the California Building Code (CBC). However, the majority of the site materials are expected to have an EI of less than 51. The more expansive soils (EI>51) are expected to be localized and associated with interbedded silt and clay layers.
- ***Collapse Potential:*** The 'hydro-collapse' potential was evaluated in the laboratory on representative soil samples in accordance with the consolidation test procedure per ASTM D4546. The test specimens were inundated with water at confining pressures ranging from 1 or 2 kips per square-foot, approximating those pressures that may be exerted after completion of grading. The test results indicate that the potential for 'hydro-collapse' is generally lower than 5 percent (moderate trouble). However, higher collapse potential should be expected in localized areas and may range up to 9 percent as encountered in Borings B-8 and B-10.

3.2.4 Landslide Materials

A large older landslide has been mapped primarily offsite on the north easterly flanks of Mount Russell, near the southwest portion of the property (See Figure 2). The landslide appears to have originated on the

higher slopes (offsite) and moved northeast, partially onto the subject property. The majority of the landslide material is offsite and not affecting the planned development areas. If development is proposed in this area, additional study for determination of landslide stability or remedial measures such as buttressing or removal of slide debris may be required.

3.2.5 San Timoteo Formation

The Pleistocene aged San Timoteo Formation is observed in surface exposures in the northeastern portion of the property and encountered at depth in some of our exploratory borings B-6, B-7 and B-17 (Leighton, 2006). The San Timoteo Formation locally consists of poorly to moderately consolidated sands, silts, sandy gravels and gravel conglomerates. The non-porous, medium dense to dense San Timoteo formation materials are considered suitable to support additional fill or foundation loads.

3.2.6 Granitic Bedrock

The southwest elevated corner of the site (Mount Russell) is comprised of heterogeneous granitic bedrock. Several large boulders are visible on the easterly facing slopes that may be concern for rock fall hazard to development located west of Davis Road/Theodore Street. The location of these boulders relative to site development should be reviewed when development plans become available.

3.3 Surface Water and Groundwater

Surface water and groundwater was not observed or encountered up to the maximum explored depth of 81 feet. According to the Riverside County General Plan Safety Element (2003), depths of groundwater within the subject site are reported to be in excess of 100 feet below existing ground surface. Two deep test wells were constructed southeast of the subject site in August 1980 (Mann, 1991). These wells were drilled to depths of 1110 feet and 1350 feet below existing ground surface. Groundwater was reported at 106 feet in one of the wells. Groundwater is not anticipated to be a constraint during grading or future development. However, localized perched groundwater conditions may occur due to heavy rainfall or irrigation.

4.0 FAULTING AND SEISMICITY

4.1 Regional Faulting and Fault Activity

The subject site, like the rest of Southern California, is located within a seismically active region as a result of being located near the active margin between the North American and Pacific tectonic plates. The principal source of seismic activity is movement along the northwest-trending regional fault systems such as the San Andreas, San Jacinto, and Elsinore Fault Zones. Currently, these fault systems accommodate up to approximately 55 millimeters per year (mm/yr) of slip between the plates. The onsite San Jacinto Fault Zone is estimated to accommodate slip of approximately 12 mm/yr (WGCEP, 1995). However, geodetic measurements between 1973 and 1981 show that the San Jacinto and San Andreas Faults currently have comparable strain rates. King (1983) and Morton (1993) have estimated an average slip rate of as much as 20 mm/yr for the San Jacinto Fault. The San Jacinto Fault zone presents an appreciable seismic hazard in Southern California.

By definition of the California Geological Survey, an active fault is a fault, which has had surface displacement within Holocene time (about the last 11,000 years). This definition is used in delineating Earthquake Fault Zones as mandated by the Alquist-Priolo Geologic Hazards Zones Act of 1972 and as most recently revised in 2007 (Bryant, 2007) as the Alquist-Priolo Earthquake Fault Zoning Act and Earthquake Fault Zones. The intent of this act is to require fault investigations on sites located within Earthquake Fault Zones to assure that certain inhabited structures are not constructed across the traces of active faults.

The nearest Alquist-Priolo zoned “active faults” is the onsite Claremont Segment of the San Jacinto Fault Zone, (see Figure 3). The nearest offsite fault zones include: Casa Loma Segment of the San Jacinto Fault Zone, located 1.6 miles (2.5 km) to the south, the San Andreas Fault Zone, located 12.7 miles (20.5 km) northeast, and the Glen Ivy Segment of the Elsinore Fault is located approximately 22.7 miles (36.6 km) to the southwest of the site (Blake, 2000b and Bryant, 2007). Additionally, an un-named fault splay is mapped at the northwest property boundary, near the SR-60/Theodore Street interchange (see Plate 1).

4.2 Local Faulting and Fault Activity

Historically, the San Jacinto fault zone has produced earthquakes in the magnitude range of 6.2Mw to 7.2Mw ('Mw' is the Moment Magnitude as defined by the U.S.G.S). Of all the fault systems in California, the San Jacinto Fault and San Andreas Fault are among the most active. Since the recording of seismic events in the mid-19th century, at least 6 major earthquakes have occurred along the San Jacinto Fault Zone (Blake, 2000a). Each of these major quakes have produced moderate to severe damage to buildings and roads, and have resulted in several fatalities over this time-period (Von Hake, 1971). Hundreds of minor earthquakes (magnitude 1.0 to 2.9) occur annually in the San Jacinto Valley. The majority of these earthquakes occur in the bedrock underlying the alluvium unit typically at depths of 3 to 5 miles (5-8 km).

A table of major quakes (□6.0Mw) within 20 miles of the study area in the last 200 years, (Blake, 2000a) is presented in the following table.

Date	Moment Magnitude	Intensity	Distance from Study Area
1880	6.0 Mw	X	0.8 miles (1.3 km)
1923	6.25 Mw	IX	6.7 miles (10.7 km)
1899	6.4 Mw	VIII	14.2 miles (22.8 km)
1918	6.8 Mw	VIII	16.3 miles (26.2 km)

Evidence for recent (latest-prehistoric and possibly historic) fault ruptures have been identified within many trenches crossing the fault zone in other areas of the San Jacinto Valley (Kahle, 1987; Lohr, 1980; Lohr, 1997; etc.). Observations of Holocene faulting exposed in exploratory trenches in the area indicate that evidence of active faulting extends upward to within roughly the upper eight feet of the modern ground surface. Given the relatively short recurrence interval of earthquakes and fault rupture for the San Jacinto Valley area fault zones, a reasonable conclusion is that frequent faulting may have resulted in a higher number of recent surface rupture events in this area. Faults and related geologic structures resulting from these events should therefore be more easily recognizable in younger near surface sedimentary deposits.

The site is located along the Claremont Segment and near the Casa Loma Segment of the San Jacinto Fault Zone. Data published by the Southern California Earthquake Center (SCEC, 2007); the Working Group of California Earthquake Probabilities, (WGCEP, 1995); the United States Geological Survey

(USGS, 2007); Katherine Kendrick, (Kendrick, 2005); and Dr. Tom Rockwell, (Rockwell, 2003); all suggest a 60 to 200 year recurrence interval between surface ruptures for fault segments within this portion of the San Jacinto Fault Zone. Earthquake activity within this fault zone occurs on a more frequent interval than many other active fault zones in Southern California.

For example, assuming a characteristic recurrence model and a conservative value of 200 years for the return time between rupture events, the onsite faults could experience approximately 10 faulting events during the Late Holocene time period (~2000 y.b.p.). Assuming not every earthquake event is large enough to generate surface ground rupture for the entire length of the fault segment, and that multiple fault splays may exist with each zone, we can further hypothesize that even if only every third of the events are sufficiently large and generates ruptures within the zone being addressed, one should still see faulting readily apparent in middle to late Holocene deposits, if they are within the active fault zone.

Based upon historical accounts of ground rupture during earthquakes in the San Jacinto/Moreno Valley area of 1899, 1916, and 1918, ground rupture is believed to occur at normal recurrence intervals which may not provide adequate time for erosion to conceal fault features. Carl A. Von Hake noted in, *Earthquake Information Bulletin, Volume 3, Number 2, March - April 1971*, that severe damage, and ground rupture/subsidence has occurred in association with the previously mentioned earthquakes on the San Jacinto Fault Zone.

The onsite Claremont Segment of the San Jacinto Fault Zone is considered to be an **active** fault which has evidence of displacement of Holocene soils. The eastern portion of the site is transected by an Earthquake Fault Zone (EFZ) as created by the Alquist-Priolo Earthquake Fault Zoning Act (Bryant, 2007). The western portion of the site is crossed by the City of Moreno Valley Seismic Zone and the postulated trace of the Casa Loma Fault (Dibblee, 1981, Park, 1995). A detailed fault investigation was performed by Leighton for the postulated extension of the Casa Loma fault and the associated City of Moreno Valley Seismic Zone (Leighton, 2004, 2007). The postulated Farm Road Strand (Park, 1995) was not encountered in our exploratory fault trench T-7 (Leighton, 2007a). During this fault study it was determined by Leighton and the City of Moreno Valley's consulting geologist that the postulated Farm Road Strand did not exist as previously interpreted (Leighton, 2007, Eberhart, 2007). This 30 to 40 foot deep exploratory trench was located south of and adjacent to Alessandro Boulevard and east of Theodore Street at the postulated location of the Farm Road Strand (See Plate 1).

Although no active faulting was observed at the projection of the Casa Loma fault or postulated Farm Road Strand, some local discontinuous fracturing was observed and documented (Leighton 2004, 2007a, and 2012). It is Leighton's opinion that a "special foundation or grading remediation zone" for the areas supporting structures intended for human occupancy be created where co-seismic deformation is observed. This area should be determined after the conclusion of recommended subsurface evaluations and when building locations are being considered for planning purposes. California Code of Regulations, Section 3601 states, "A structure for human occupancy is any structure used or intended for supporting or sheltering any use of occupancy, which is expected to have a human occupancy rate of more than 2,000 person-hours per year". Additional fault study and structural setback determination should be anticipated. Specific remedial earthwork and foundation recommendations will need to be evaluated when development plans become available. Structures intended for human occupancy should **not** be located within any structural setback zone as determined by those studies.

4.3 Seismic Design Parameters

The nearest known active fault and source of the design earthquake is the San Jacinto Valley Segment (Claremont Segment) of the San Jacinto Fault located along the eastern boundary of the site. The effect of strong seismic shaking may be reduced to prevent structure failure by adhering to the 2013 California Building Code (CBC) and seismic design parameters suggested by the Structural Engineers Association of California.

Due to the large size of this project, we are providing below two sets of parameters: Area 1 generally represents the western portion of the site and Area 2 represents the eastern portion of the site. These parameters should give a general range for the seismic design coefficients expected on this site, which should be confirmed for each lot/building once site development plans become available.

The site-specific seismic coefficients provided in this section are based on an interactive tool/program currently available on USGS website. Based on ASCE 7-10 as the Design Code Reference Document and site Class D, the seismic coefficients for this site are as listed in the following table:

Table 1. 2013 CBC Seismic Coefficients per USGS General Procedure

CBC Categorization/Coefficient		Design Value (g)	
		Area 1 Lat: 33.9211 Long: -117.1556	**Area 2** Lat: 33.9107 Long: -117.0936
Site Class Definition	D		
Mapped Spectral Response Acceleration at 0.2s Period, S_s		**2.17**	**2.56**
Mapped Spectral Response Acceleration at 1s Period, S_1		**0.99**	**1.18**
Short Period Site Coefficient at 0.2s Period, F_a		**1.00**	**1.00**
Long Period Site Coefficient at 1s Period, F_v		**1.50**	**1.50**
Adjusted Spectral Response Acceleration at 0.2s Period, S_{MS}		**2.17**	**2.56**
Adjusted Spectral Response Acceleration at 1s Period, S_{M1}		**1.48**	**1.76**
Design Spectral Response Acceleration at 0.2s Period, S_{DS}		***1.45***	***1.71***
Design Spectral Response Acceleration at 1s Period, S_{D1}		***0.99***	***1.18***

* g- Gravity acceleration

The results of the analysis also indicate that the adjusted Peak Ground Acceleration (PGA_M) for Area 1 is 0.84g and for Area 2 is 0.99g.

4.4 Secondary Seismic Hazards

Secondary hazards generally associated with severe ground shaking during an earthquake are ground rupture, tsunamis and seiches, landsliding, rockfalls, ground fissuring and subsidence, liquefaction, and seismic densification. These hazards are discussed in the following sections.

4.4.1 Ground Rupture

Ground rupture is generally considered most likely to occur along pre-existing active (Holocene) faults. Our review of previous investigations and data gathered during the previous fault trenching investigation has identified through-going recent (Holocene) faults near the eastern boundary. As such, the potential for site ground rupture along this fault zone is considered high and structural setbacks are recommended and lifeline utilities that cross the fault zones should be evaluated for the effects caused by possible ground rupture.

4.4.2 Tsunamis and Seiches

Due to the distance to large bodies of water (inland seas, large rivers, and oceans) from the site, the possibility of tsunamis is considered nil. An

examination of ridge top elevations surrounding Perris Lake compared to the current elevation of the lake reveals that there is little likelihood that seiches from Perris Lake could overcome these observed ridge top elevations. The ephemeral Mystic Lake is projected to be approximately 70 feet below the south project boundary (low point). Seiches on Mystic Lake to affect this site are considered unlikely.

4.4.3 Landsliding

Landslides have been previously mapped by others and identified by us during our field investigation within the property. This large bedrock landslide exists, primarily offsite on the north eastern flank of Mount Russel (see Figure 2). This landslide complex is not anticipated to impact this development as currently planned, however, additional review should be performed if development is planned for this vicinity. Land sliding within the low lying portions of the property due to seismic activity is not anticipated within the site due to the flat-lying nature of the site and the anticipated design grades.

4.4.4 Rock Falls

The steep sloping hillsides in the area located generally southeast of the intersection of Theodore Street and Alessandro Boulevard, southwest corner of the World Logistics Center project contains many potentially loose boulders. The potential for rock fall due to either erosion or seismic ground shaking is considered high in this area. Remedial measures such as rock removal, anchoring, catchment areas, rock fences, or setbacks may be required if development is planned along these slopes. The potential hazard from individual rocks should be assessed as site development plans are provided.

4.4.5 Ground Fissuring and Subsidence

No evidence of recent ground fissuring was observed or been reported within the project boundary, or projecting into the property from immediately adjacent properties. Regional subsidence was reported in various areas of San Jacinto Valley as a result of ground water withdrawal and regional tectonics (Morton, 1994). Subsidence and resulting ground fissuring is documented approximately 6,600 feet south of the site (Figure 3). Localized discontinuous subsurface features likely related to past

historic ground shaking or liquefaction events were observed (Leighton, 2004, 2007a and 2012) and discussed in Section 4.2 of this report.

4.4.6 Liquefaction

Liquefaction of cohesionless soils or soils of low plasticity can be caused by strong vibratory motion due to earthquakes. Research and historical data indicate that loose granular soils or soils of low plasticity below a near surface groundwater table are most susceptible to liquefaction. Liquefaction is characterized by a loss of shear strength in the affected soil layers, thereby causing the soil to flow as a viscous fluid. This effect may be manifested at the ground surface by settlement and/or sand boils. In order for the potential effects of liquefaction to be manifested at the ground surface, the soils generally have to be granular or of low plasticity, loose to medium dense, saturated relatively near the ground surface and must be subjected to a sufficient magnitude and duration of ground shaking.

Groundwater was not encountered during this or our past subsurface explorations on this site to a maximum depth of 81 feet. The subject site is not within a liquefaction hazard zone (Riverside County, 2003). Groundwater within the subject site is reported to be in excess of 100 feet below existing ground surface (see Section 3.3). Due to deep groundwater, relatively dense alluvial soils and interbedded clay layers underlying the site, it is our opinion that potential for liquefaction at the subject site is very low.

4.4.7 Seismic Densification

Ground accelerations generated from a seismic event can produce settlements in dry or moist sands (granular earth materials) with relative low density. We anticipate that the near-surface loose soil deposits susceptible to such seismically induced settlement will be removed and recompacted during grading. The potential total seismic densification is anticipated to be on the order of 1 inch. Due to relatively homogenous subsurface conditions (alluvial deposits in the upper 35 to 50 feet) the differential settlement is expected to be less than ½ inch over 40 lateral feet, provided that the recommendations for remedial grading (see Section 6.1.1) are adhered to. Additional evaluation of seismic densification during future site development plan reviews are recommended.

5.0 CONCLUSIONS

Based on the results of this preliminary geotechnical evaluation, it is our opinion that the planned site development is feasible from a geotechnical standpoint. The following is a summary of the major geotechnical constraints or opportunities that may affect site development:

- The existing onsite soils are generally suitable for reuse as fill during proposed grading provided they are free of organic material and debris. If encountered, the medium to high expansive soils (51<EI<131) should not generally be placed within the upper 5 feet of finished grades.
- The results of our testing indicate a high rebound potential during unloading for some of the tested alluvium. This rebound affect may cause some elevation rise in areas of significant excavation.
- Undocumented fills and the upper low density alluvium are potentially compressible and should be removed and recompacted prior to construction of settlement-sensitive structures. Depth of removal will generally depend on type of structure and/or footing size and loads. Where high collapse potential exist (>5%) percent, deeper removal and/or special ground improvements techniques may be required.
- All loosely placed exploratory fault trench backfill should be removed and recompacted in accordance with the recommendations in Section 6.1. The approximate locations of the trenches are depicted on the *Field Exploration Map* (Plate 1). Trenches excavated as part of Leighton's work (Leighton 2007a, 2012) have been backfilled with properly compacted fill and will not require re-excavation and compaction. Previously excavated trenches will require removal and recompaction.
- Due to the variable nature of site alluvial soils, cut slopes in excess of 5 feet in vertical height should be constructed as replacement fill slopes.
- The onsite earth materials can be generally excavated with heavy-duty conventional grading equipment in good working condition.
- Evidence of active faulting has been identified along the eastern boundary and structural setbacks have been recommended. The limit of recommended preliminary fault setback zone based on recent fault trenching is presented on the accompanying *Field Exploration Map* (Plate 1). This structural setback may vary pending future studies and cut or fill thickness near the fault zone.

- Structural setbacks from the edges of the unnamed drainages that are to remain in their natural state should also be considered. When the project plans are further developed, the design and natural slopes adjacent to structural improvements should be evaluated by Leighton.
- Where co-seismic deformation (fractures) as observed in the previous exploratory trenches (Leighton, 2004, 2007a) specific remedial grading and foundation recommendations will need to be evaluated when development plans become available.
- The onsite soils possess a very low to high expansion potential and negligible sulfate exposure to concrete. Limited laboratory test data also show that the onsite soils may have a mild to moderate corrosion potential for buried metal improvements.
- Strong ground shaking and settlement (seismic densification) may occur at this site due to local earthquake activity. Additional evaluations of seismically induced settlement should be performed when grading and development plans become available.
- Groundwater was not encountered up to the maximum explored depth of 81 feet during our site investigations. Shallow groundwater is not expected to be a factor during site grading.
- Perched water may develop in areas of soils with contrasting permeability's, possibly resulting in saturated fills or seepage from slopes. This condition is often a result of site water use and irrigation practices.
- Fill slopes are anticipated for the proposed development. Unprotected slope faces will be susceptible to erosion. This risk may be reduced by planting the slopes as soon as possible after grading, and by maintaining proper erosion control measures.

6.0 PRELIMINARY RECOMMENDATIONS

6.1 General Discussion/Remedial Grading

The proposed development of this site appears feasible from a geotechnical viewpoint. The presence of relatively compressible and locally expansive alluvium is a major geotechnical concern on this site. One of the most practical and cost effective methods to mitigate the loose/compressible alluvium is the removal and recompaction of the compressible materials prior to construction of structural improvements. For planning purposes, we anticipate overexcavation and recompaction of the upper 5 to 10 feet of alluvium beneath all settlement-sensitive structures/heavy foundations to reduce the potential for excessive differential settlement. The overexcavation may extend to a depth of 15 to 20 feet in areas where high collapse potential (>5%) or coseismic deformation is encountered. Moreover, due to the presence of potentially expansive soils on this site, selective grading procedures should be anticipated in certain areas to ensure that expansive soil is not placed within 5 feet of finish grades. The depth of removal/over-excavation is expected to be shallower (2 to 5 feet) below pavement and in hardscape areas.

Due to the limited nature of our investigations and the variable soils conditions encountered, the recommendations included below are considered preliminary. Building specific investigations will be needed to further delineate the extent of the compressible and expansive alluvium on individual pads so that differential settlement can be further verified or evaluated in view of the actual foundation plan and imposed fill or structural loads.

Active faulting has been identified onsite and preliminary fault setback zones have been provided (see Plate 1). Additional fault studies may be needed if structures are planned adjacent to the “preliminary setback zones” or other areas within the State Earthquake Fault Zone or other mapped faults.

6.1.1 Fault Area Restrictions

The location of the surface projection of the onsite active faults or limits of significant fault zones exposed in our exploratory trenches were surveyed by the project civil engineer at the time of our investigation (Leighton 2007a). Based on the distance between our exploratory trenches and understanding of onsite faulting, we preliminarily recommend a 100-foot setback on either side of the encountered fault zone at each trench, as

projected between trenches T-8 through T-10 and as shown on the accompanying *Field Exploration Map* (Plate 1).

The faults encountered within the fault setback zone have a dipping component and cut or fill grading which significantly changes the present grades will shift the fault setback zones. The preliminary fault setback zone presented on the *Field Exploration Map (*Plate 1) is simply plotted as a setback from the mapped and surveyed fault zone without adjustment due to overlying cuts or fills. The preliminary fault setback zones should be refined as the grading concept is developed and the scale of plan is increased (i.e. 100-scale minimum).

6.2 General Earthwork Considerations

For planning purposes, earthwork should be performed in accordance with the General Earthwork and Grading Specifications in Appendix B and the following preliminary recommendations below. The recommendations contained in Appendix B are general grading specifications provided for typical grading projects and some of the recommendations may not be strictly applicable to this project. We anticipate that the findings of the recommended additional subsurface investigations may require modification to the recommendations presented herein.

6.2.1 Structural Fills

The onsite soils are generally suitable for reuse as compacted fill, provided they are free of organic materials, debris and oversize materials (greater than 8 inches in greatest dimension). The optimum lift thickness to produce a uniformly compacted fill will depend on the type and size of compaction equipment used. In general, fill should be placed in uniform lifts not exceeding 8 inches in thickness. Fill soils should be placed and compacted to a minimum 90 percent relative compaction (as determined by ASTM Test Method D1557). The compacted soil should be at or near optimum moisture content (±2 percent or depending on the prevailing soil material). If required per project requirements, the upper 12 inches of subgrade for floor slab should be compacted to a minimum 95 percent relative compaction.

The outer portion of fill slopes should be either overbuilt by 2 feet (minimum) and trimmed back to the finished slope configuration or

compacted in vertical increments of 5 feet (maximum) by a sheepsfoot roller as the fill is placed. The slope face should then be trackwalked by dozers of appropriate weight to achieve the final configuration and compaction out to the slope face.

Placement and compaction of fill should be performed in accordance with the City of Moreno Valley Grading Ordinance under the fulltime observation and testing of the geotechnical consultant.

6.2.2 Shrinkage and Subsidence

The volume change of excavated onsite materials upon recompaction is expected to vary with materials, density, insitu moisture content, location and compaction effort. The in-place and compacted densities of soil materials vary and accurate overall determination of shrinkage is difficult to estimate. Therefore, we recommend that site grading include, if possible, a balance area or ability to adjust import or export quantities to accommodate some variation.

For planning purposes, we recommend that the following shrinkage factors be applied:

Depth	Shrinkage (%)
0 to 1 feet	25 to 35
1 to 5 feet	18 to 22
5 to 10 feet	14 to 18
10 to 30 feet	8 to 14

In addition, due to the past and recent agricultural discing of the surface soils, a subsidence value of 0.2 to 0.4 feet should be applied to the existing topographic surface elevations and 0.1 to 0.2 feet to other non-disked areas. In addition, a subsidence factor of 0.10 feet should be applied to the soils below the removal bottoms due to recompaction and surcharging.

6.2.3 Import Soils

If import soils are needed to establish the site design elevations, it should be granular in nature, relatively free of organic material, have an expansion index less than 51 (per ASTM Test Method D4829), and have a low corrosion impact to the proposed improvements. Import soils, if

needed, and potential borrow sites should be evaluated by the geotechnical consultant prior to being imported to the site.

6.2.4 Trench Excavation and Backfill

Utility trenches should be backfilled with compacted fill in accordance with the *Standard Specifications for Public Works Construction*, ("Greenbook"), 2012 Edition. Fill material should be placed in lifts not exceeding 8 inches in uncompacted thickness and should be compacted to at least 90 percent relative compaction (ASTM D 1557). The upper 6 inches of backfill in all pavement areas should be compacted to at least 95 percent relative compaction.

Where granular backfill is used in utility trenches adjacent to moisture sensitive subgrades and foundation soils, we recommend that a cut-off "plug" of impermeable material be placed in these trenches at the perimeter of buildings, and at pavement edges adjacent to irrigated landscaped areas. A "plug" can consist of a 5-foot long section of clayey soils with more than 35-percent passing the No. 200 sieve, or a Controlled Low Strength Material (CLSM) consisting of one sack of Portland-cement plus one sack of bentonite per cubic-yard of sand. CLSM should generally conform to Section 201-6 of the Greenbook. The CLSM plug is intended to reduce the likelihood of water migrating from landscaped areas along permeable trench backfill into the building and pavement subgrades, resulting in wetting of moisture sensitive subgrade earth materials under buildings and pavements.

Excavation of utility trenches should be performed in accordance with the project plans, specifications and the *California Construction Safety Orders* (2010 Edition or more current). The contractor must be responsible for providing a "competent person" as defined in Article 6 of the *California Construction Safety Orders*. Contractors should be advised that sandy soils (such as fills generated from the onsite alluvium) could make excavations particularly unsafe. All safety precautions should be properly implemented at all times. In addition, excavations at or near the toe of slopes and/or parallel to slopes may be highly unstable due to the increased driving force and load on the trench wall. Spoil piles from the excavation(s) and construction equipment should be kept away from the

sides of the trenches. Leighton does not consult in the area of safety engineering.

6.3 Preliminary Foundation and Slab Design

6.3.1 Minimum Footing Dimensions and Embedment

Based on the results of our previous investigations, it is our opinion that the proposed commercial/industrial buildings may be supported on conventional (continuous) footings or isolated spread/pad footings with slab on grade. Foundations and slabs should be designed in accordance with structural considerations and the soils design recommendations presented below.

Footings should be embedded at least 18-inches below lowest adjacent grades. Footing embedments are measured from lowest adjacent finished grade, considered as the top of interior slabs-on-grade or the finished exterior grade, excluding landscape, whichever is lower. Footings located adjacent to utility trenches or vaults should be embedded below an imaginary 1:1 (horizontal to vertical) plane projected upward and outward from the bottom edge of the trench or vault towards the footing. Continuous/strip footings should have a minimum width of 18-inches, while column footings should have a minimum width of 24-inches. Upon the completion of the grading and structural plans, the geotechnical engineer should review the foundation loads and embedment in order to confirm the implementation of the recommendations herein. All footing excavations should be observed by geotechnical engineer before reinforcing steel is placed.

6.3.2 Allowable Vertical Bearing

The shallow foundation recommendations herein assume low expansive soil (EI <51). Foundations designed on soils of Expansion Index (EI) of 51 or greater shall require special design consideration. In accordance with the above footing dimensions and embedment, isolated pad footings (for column loads) and continuous/strip footings may be designed using an allowable bearing capacity of 2,000 psf. The bearing pressure value may be increased by 250 psf for each additional foot of embedment and each additional foot of width to a maximum vertical bearing value of 2,500 psf. These bearing values may be increased by one-third when considering

short-term seismic or wind loads. A sliding coefficient friction of 0.35 should be applied to the design of foundation elements.

Conventional footings/slab may be enhanced by structurally tying the slabs-on-grade to the perimeter and interior footings as designed by the project Structural Engineer. Continuous footings should be designed in accordance with the structural engineer requirements and should have a minimum reinforcement of four No. 5 reinforcing bars (two top and two bottom). Reinforcement of isolated footings should be per the structural engineer's design.

6.3.3 Interior Floor Slab Design

Slab-on-grade floors utilized with conventional foundations should be designed with a minimum thickness as indicated by the project structural engineer consistent with a modulus of subgrade reaction of 150 pounds-per-square-inch per inch (pci) and reinforced in accordance with the structural engineer's recommendations. A slip-sheet or equivalent should be used if crack-sensitive floor coverings (such as ceramic tiles, etc.) are to be placed directly on the concrete slab-on-grade.

It has been a standard of care in this locality to install a moisture retarder (2 inches of sand over an impermeable membrane over an additional 2 inches of sand) underneath all slabs where moisture condensation is undesirable. Moisture vapor retarders may retard but not totally eliminate moisture vapor movement from the underlying soils up through the slabs. Moisture vapor transmission may be additionally reduced by use of concrete additives. It is our opinion that the proposed vapor retarder system be designed or approved by the architect or entire design team including concrete subcontractors and manufacturers of floor coverings.

6.4 Settlement

Settlement of onsite fill materials is expected to occur during and within 90 days following fill placement. However, following the placement of fill and construction of structures, additional settlement may occur due to: (a) new footing/foundation loads, (b) compression within the fill due to the effects of landscaping irrigation, (c) compression of the left-in-place alluvial soils, and (d) dynamic settlements below the removal depths. The settlements below are general estimates/guidelines and

should be further verified based on actual structural loads/footing size and additional subsurface investigation performed for individual buildings.

6.4.1 Settlement from Building Loads

Settlement resulting from buildings located on compacted fill soils (minimum 90 percent relative compaction) is not expected to be significant and is estimated to be less than 2 inches (total) and ½ inch (differential) in 30 feet or the least dimension of the building, whichever is a greater distortion. The majority of settlement associated with building loads is anticipated to occur during construction as the load is applied. These settlements and angular distortions are for imposed building loads and do not include compression within the fill itself, underlying alluvium nor dynamic settlements.

6.4.2 Dynamic Settlement

The earth materials onsite may undergo significant earthquake-induced settlement during the design seismic event. To reduce the effects and magnitude of the earthquake-induced settlements, remedial grading is recommended (see Section 6.1). Following remedial grading, differential dynamic settlements are anticipated be on the order of ½ inch over 40 lateral feet as a result of the design basis earthquake.

6.5 Footing Setback

We recommend a minimum horizontal setback distance from the face of slopes for all structural footings (retaining and decorative walls, building footings, etc.). This distance is measured from the outside bottom edge of the footing horizontally to the slope face (or to the face of a retaining wall) and should be a minimum of H/2, where H is the slope height (in feet). The setback should not be less than 7 feet and need not be greater than 10 feet.

Note that the soils within the structural setback area possess poor lateral stability and improvements (such as retaining walls, sidewalks, fences, pavements, etc.) constructed within this setback area may be subject to lateral movement and/or differential settlement. Potential distress to such improvements may be mitigated by providing a deepened footing or a pier and grade-beam foundation system to support the improvement. The deepened footing should meet the setback as described above.

6.6 Lateral Resistance and Earth Pressures for Retaining Wall Design

For preliminary design purposes, the lateral earth pressure values for level or sloping backfill are recommended for retaining walls (loading dock walls) backfilled with onsite and/or import soils of low expansion potential (expansion potential less than 50 per ASTM Test Method D4829). Where applicable, basement walls or cantilever retaining walls (less than 12 feet in height) should be designed for lateral earth pressures as described in this section. The magnitude of these pressures depends on the amount that the wall can yield horizontally under load. If the wall can yield enough to mobilize full shear strength of backfill soils, then the wall can be designed for "active" pressure. If the wall cannot yield under the applied load, the shear strength of the soil cannot be mobilized and the earth pressure will be higher. Such walls should be designed for "at rest" conditions. If a structure moves toward the soils, the resulting resistance developed by the soil is the "passive" resistance. Retaining walls should be designed using the following equivalent fluid pressures:

Table 2. Wall Design Earth Pressures (Static, Drained Conditions)

Loading Conditions	Equivalent Fluid Density (pcf) For Level Backfill*	Equivalent Fluid Density (pcf) For 2H:1V Backfill*
Active*	35	50
At-Rest*	50	75
Passive**	300	300

*For low-expansive backfill, only.
** Maximum passive pressure not to exceed 3,000 psf at depth.

Foundations placed in properly compacted fill soils may be designed using a coefficient of friction of 0.35 (total frictional resistance equals coefficient of friction [concrete on soil] times the dead load). A design passive resistance value of 250 psf per foot of depth (with a maximum lateral passive value of 2,500 psf) may be used. The allowable lateral resistance can be taken as the sum of the frictional resistance and the passive resistance provided the passive resistance does not exceed two-thirds of the total allowable resistance. The total lateral resistance value may be increased by one-third when considering loads of short duration such as wind or seismic forces.

Unrestrained (yielding) cantilever walls should be designed for the active equivalent fluid weight value provided above for very low to low expansive soils

that are free draining. In the design of walls restrained from movement at the top (non-yielding) such as basement or elevator pit/utility vaults, the at-rest equivalent fluid weight value should be used. These equivalent fluid pressures do not include the effect of traffic or earthquake surcharges in the backfill. Accordingly, an increment of lateral earth pressure equal to $18H^2$ where H is the height of the wall, should be applied at a distance of 0.5 H above the toe of the wall. Under the combined effects of static and earthquake loads on the wall, a factor of safety between 1.1 and 1.2 is acceptable when evaluating the stability (sliding, overturning) of the wall (NAVFAC DM 7.2).

Should a sloping backfill other than a 2 to 1 (horizontal to vertical) be constructed above the wall (or a backfill is loaded by an adjacent surcharge load), the equivalent fluid weight values provided above should be evaluated on an individual case basis by the geotechnical engineer. All retaining wall structures should be provided with appropriate drainage and waterproofing. The outlet pipe should be sloped to drain to a suitable outlet. Typical wall drainage design is illustrated in Appendix D. In light of expansive wall backfill that might be generated during wall installation on this site, Leighton recommends that a 3-foot select backfill zone (rectangular distribution) be used. This select backfill should be very low to low expansive soil. The drainage layer may be used as part of this select backfill zone.

Alternatively, revised equivalent fluid pressure values for medium or highly expansive soil may be provided upon request by the structural consultant and/or developer for this project.

Foundations for retaining walls in competent formational soils or properly compacted fill should be embedded at least 18 inches below lowest adjacent grade with proper footing setback (Section 6.4). At this depth, an allowable vertical bearing capacity of 2,500 psf may be assumed. This vertical bearing value may be increased 500 psf per foot of additional depth to a maximum bearing of 3000 psf for retaining walls.

It is recommended that loading dock walls be designed for restrained conditions (indicated in the previous section of this report) where adjacent to the proposed building. Loading dock walls should be designed to resist lateral earth pressure and any additional lateral pressure caused by surcharge loads on the adjoining slab surface (stock piled goods). Approximately ¼ to ½ of the surcharge loads on the adjoining truck loading dock slab may be added as a uniform surcharge

load in back of the loading dock wall. For example, a stockpiled load of 100 psf may exert 25 to 50 psf on the back of the adjacent loading dock wall. The structural consultant should review the slab/wall connection(s) and the likely wall condition, i.e. restrained or active wall condition.

6.7 Site Soil Corrosivity

6.7.1 Ferrous Corrosivity

Many factors can affect the corrosion potential of soil including soil moisture content, resistivity, permeability and pH, as well as chloride and sulfate concentration. In general, soil resistivity, which is a measure of how easily electrical current flows through soils is the most influential factor. Based on the findings of studies presented in ASTM STP 1013 titled "Effects of Soil Characteristics on Corrosion" (February, 1989), the approximate relationship between soil resistivity and soil corrosiveness was developed as shown in Table 3.

Table 3. Relationship between Soil Resistivity and Soil Corrosivity

Soil Resistivity (ohm-cm)	Classification of Soil Corrosiveness
0 to 900	Very Severely Corrosive
900 to 2,300	Severely Corrosive
2,300 to 5,000	Moderately Corrosive
5,000 to 10,000	Mildly Corrosive
10,000 to >100,000	Very Mildly Corrosive

Acidity is an important factor of soil corrosivity. The lower the pH (the more acidic the environment), the higher the soil corrosivity will be with respect to buried metallic structures and utilities. As soil pH increases above 7 (the neutral value), the soil is increasingly more alkaline and less corrosive to buried steel structures, due to protective surface films, which form on steel in high pH environments. A pH between 5 and 8.5 is generally considered relatively passive from a corrosion standpoint. Chloride and sulfate ion concentrations, and pH appear to play secondary roles in affecting corrosion potential. High chloride levels tend to reduce soil resistivity and break down otherwise protective surface deposits, which can result in corrosion of buried steel or reinforced concrete structures.

6.7.2 Sulfate Attack

Sulfate ions in the soil can lower the soil resistivity and can be highly aggressive to Portland Cement concrete by combining chemically with certain constituents of the concrete, principally tricalcium aluminate. This reaction is accompanied by expansion and eventual disruption of the concrete matrix. Potentially high sulfate content could also cause corrosion of the structural concrete. In accordance with American Concrete Institute (ACI 318-08), Table 4 below provides exposure categories based on water-soluble sulfate in soil.

Table 4. Relationship between Sulfate Concentration and Exposure Category

Sulfate In Water (parts-per-million)	Water-Soluble Sulfate (SO_4) in soil (percentage by weight)	Exposure
0-150	0.00 - 0.10	Negligible
150-1,500	0.10 - 0.20	Moderate[1]
1,500-10,000	0.20 - 2.00	Severe
>10,000	Over 2.00	Very Severe

[1]= seawater

6.7.3 Corrosivity Test Results

To evaluate the corrosion potential of the subsurface soils, we tested a soil sample collected during our subsurface investigation for soluble sulfate and chloride content, pH and Resistivity testing. Results of these tests are summarized in Table 5 below.

Table 5. Results of Corrosivity Testing

Boring Number	Sample Depth (feet)	Sulfate (mg/kg)	Chloride (mg/kg)	PH	Minimum Resistivity (ohm-cm)
B-17	15 to 20	<150	460	7.98	14,385

Note: mg/kg = milligrams per kilogram, or parts-per-million

As shown in Table 5, minimum soil resistivity of 14,385 ohm-centimeters was measured in our laboratory test. Based on resistivity correlation presented in Table 3, it appears that the corrosion potential to buried steel may be characterized as "very mildly corrosive" for the tested soils. Ferrous pipe buried in moist to wet site earth materials should be avoided by using high-density polyethylene (HDPE) or other non-ferrous pipe when possible. Ferrous pipe can be protected by polyethylene bags, tap or

coatings, di-electric fittings or other means to separate the pipe from on-site soils. If buried metal improvements are planned at the site, specific recommendations for corrosion protection will need to be provided by a qualified corrosion engineer. Based on our limited laboratory test, it is our opinion that sulfate exposure may be considered "negligible" for the native subsurface materials sampled at the site.

6.8 Slope Stability

Based on our review of the conceptual grading plans and proposed remedial grading, cut slopes and fill slopes with 2:1 (H:V) slope ratios with maximum heights of 50 to 65 feet are proposed.

Based on the results of our laboratory testing and experience with similar soil types in nearby areas, the proposed 2:1 (horizontal to vertical) cut slopes will be generally grossly stable and have calculated factors of safety in excess of 1.5 under static conditions and 1.1 for seismic conditions for a maximum height of 50 feet. However, for planning purposes, due to the highly variable nature of the onsite alluvium, all cut slopes greater than 5 feet and less than 30 feet in height should be constructed as replacement fills as depicted in Appendix B. Cut slopes greater than 30 feet in height will require additional analysis. Due to proposed onsite stormwater basins and interbedded gravelly sand layers, the installation of a subdrain system and/or interceptor ditches may be needed during replacement fill construction. The subdrain system can be outletted into the site storm drain system or discharged properly into lower elevations. Additional slope stability evaluation should be performed when development plans become available.

In addition, slope faces are inherently subject to erosion, particularly if exposed to rainfall and irrigation. Landscaping and slope maintenance should be conducted as soon as possible in order to increase long-term surficial stability.

6.8.1 Natural Drainage Slopes

Based on our geotechnical evaluation, the existing major drainage courses on the east portion of the World Logistics Center property are over-steepened and subject to slope failure (toppling, slumping) and erosion. If the existing drainage courses are to remain in their natural state (i.e. undisturbed and not filed in), setbacks should be considered based on adjacent planned land use. For planning purposes, a preliminary setback

distance for structures equal to 2X the drainage channel slope height may be used. For landscaping and other non-structural uses, a minimal setback such as ½ the slope height or 7 foot minimum may be considered for improvements such as trails, fences etc. This minimal setback area is subject to effects of possible slop failure.

6.9 Drainage

All drainage should be directed away from structures and pavements by means of approved permanent/temporary drainage devices. Adequate storm drainage of any proposed pad should be provided to avoid wetting of foundation and pavement subgrade soils. Irrigation adjacent to buildings should be avoided wherever possible. As an option, sealed-bottom planter boxes and/or drought resistant vegetation should be used within 5-feet of buildings. Pavements should be separated from irrigated areas by embedded concrete curbs extending below pavement base. Below grade stormwater percolation or retention facilities (if planned) should be located a minimum distance of 15 feet from structural footings. Provisions to prevent the lateral migration of subsurface saturation below adjacent structures or into descending slopes may be needed.

6.10 Preliminary Pavement Design Parameters

For planning and estimating purposes we have made some assumptions based on the anticipated vehicle traffic usage. The appropriate pavement section will depend on the type of subgrade soil, shear strength, traffic load and planned pavement life. Since an evaluation of the actual subgrade soils cannot be made at this time, we have assumed an R-value of 30 based on limited site soil laboratory testing and our professional experience in the site vicinity. The pavement sections are calculated based on Traffic Indexes (TI) as indicated in Table 6 below:

Table 6. Preliminary Asphalt Pavement Section Thickness

General Traffic Condition	Design Traffic Index (TI)	Asphalt Concrete (inches)	Aggregate Base* (inches)
Automobile Parking Stalls	5.0	3.0	6.0
Automobile Drive Lanes	6.0	3.5	7.0
Truck Access & Parking Areas	7.0	4.0	9.0
Site Roadways	9.0	5.5	12.5
	10.0	6.0	14.5

Appropriate Traffic Index (TI) data should be selected by the project civil engineer or traffic engineering consultant and appropriate R-value of the subgrade soils will need to be determined after completion of rough grading to finalize the pavement design. Final pavement sections should be in general accordance with local, county and industry standards. The Caltrans pavement section design calculations were based on a pavement life of approximately 20 years with a normal amount of flexible pavement maintenance. Portland cement concrete should be used, rather than asphalt, in point and impact load areas such as loading docks and trash truck bin loading areas.

Subgrade soils in the upper 6 inches should be properly compacted to at least 95 percent relative compaction (ASTM D1557) and should be moisture-conditioned to optimum or slightly above optimum, and kept in this condition until the pavement section is constructed. Minimum relative compaction requirements for aggregate base should be 95 percent of the maximum laboratory density as determined by ASTM D1557.

Asphalt concrete and aggregate base should conform to *Caltrans Standard Specifications* (Latest Edition) Sections 39 and 26-1.02A, respectively. As an alternative, asphalt concrete can conform to Section 203-6 of the *Standard Specifications for Public Works Construction* (Green Book). Crushed aggregate base or crushed miscellaneous base can conform to Sections 200-2.2 and 200-2.4 of the *Standard Specifications for Public Works Construction* (Green Book), respectively. Pavement subgrades should be compacted to 90 percent and pavement base should be compacted to 95 percent of the ASTM D1557 laboratory maximum density for these materials.

For preliminary planning purposes, fire lanes and truck loading areas may be constructed of Portland Cement Concrete (PCC) with a minimum thickness of 6.0 inches assuming light axle loads and an average daily truck traffic (ADTT) of less than 500. For medium/heavy axle loads and an ADT of 500 or more, a minimum PCC thickness of 7 inches should be used, such as for trash corrals and trash truck aprons, loading docks, etc. All PCC pavement should have a minimum 28-day concrete compressive strength of 3,250 psi and have appropriate joints and saw cuts in accordance with either Portland Cement Association (PCA) or American Concrete Institute (ACI) guidelines. PCC subgrade should be compacted to 95 percent relative compaction in the upper 6 inches. For truck lanes and ramps, a 4-inch (minimum) layer of Class 2 aggregate base at 95 percent relative compaction should be considered beneath the PCC paving. This

4-inch layer of Class 2 aggregate may be used beneath other areas of PCC pavement to improve performance. The upper 6 inches of the underlying subgrade soils should also be compacted to at least 95 percent relative compaction (ASTM D1557).

The above PCC sections should be re-evaluated following the provision of the precise grading plans. We recommend that the ADT be confirmed by the project civil designer or traffic consultant prior to completion of the project.

Additional details should be added to the plans indicating the pavement thickness transitions, pavement joint dowels, expansion joints and sawcut joints. Use of concrete cutoff or edge barriers should be considered at the perimeter of the common parking or driveway areas when they are adjacent to either open (unfinished) or landscaped areas.

7.0 GEOTECHNICAL REVIEW

Geotechnical review is of paramount importance in engineering practice. The poor performance of many foundation and earthwork projects have attributed to inadequate construction review. We recommend that Leighton be provided the opportunity to review the following items.

7.1 Plans and Specifications

Leighton should review the project rough-grading plans as well as foundation and specifications prior to release for bidding and construction. Such review is necessary to evaluate whether the geotechnical recommendations have been effectively incorporated in plans and other construction documents. Review findings should be reported in writing by the geotechnical engineer. Additional subsurface evaluation may be warranted dependent upon the results of a detailed plan review.

7.2 Future Investigations

It is our opinion that the existing studies completed are sufficient for the current Specific Plan development. Based on our future review of Tentative Tract Map or specific plan, additional clarification studies may be recommended. If structural development is planned near the "preliminary structural setback zone" or other area of the State Earthquake Fault Hazard Zone or other mapped faults, coseismic fracture areas, additional geologic investigation in those areas is recommended. In order to refine the alignment and size of the setbacks, further trenching is necessary to further constrain the faulting as it crosses the property.

Additionally, upon review of grading and development plans, additional subsurface geotechnical exploration may be locally needed to further evaluate the subsurface conditions relative to the planned development for further evaluation of slope stability remedial earthwork and foundation design recommendations.

7.3 Construction Review

Observation and testing should be performed by Leighton during grading and construction. It should be anticipated that the subsurface conditions exposed during construction may vary from those encountered in the borings and other explorations. Reasonably continuous construction observation and review during

site grading and foundation installation allows for evaluation of the actual soil conditions and the ability to provide appropriate revisions during grading and construction, if required.

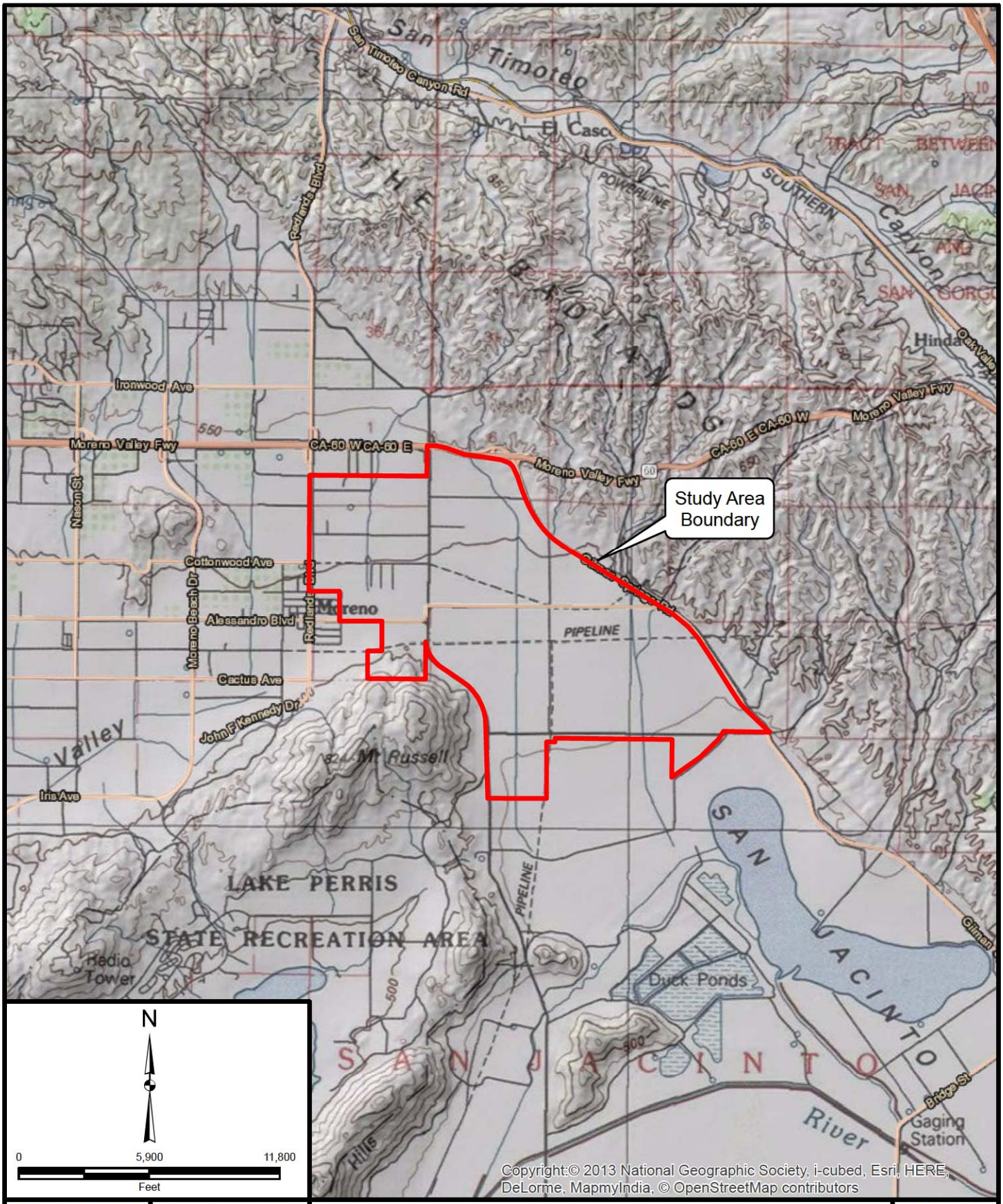
Site preparation, removal of unsuitable soils, geologic observation and mapping of all earthwork excavations, approval of imported earth materials, fill placement, foundation installation and other site geotechnically-related operations should be observed and tested by Leighton. Additional laboratory tests of subsurface materials to confirm compacted dry density and moisture content, consolidation potential corrosion potential, expansion potential, and resistance value (R-value) should be performed during or prior to grading.

8.0 LIMITATIONS

This preliminary report was necessarily based in part upon data obtained from a limited number of observances, site visits, soil samples, tests, analyses, histories of occurrences, spaced subsurface explorations and limited information on historical events and observations. Such information is necessarily incomplete. The nature of many sites is such that differing characteristics can be experienced within small distances and under various climatic conditions. Changes in subsurface conditions can and do occur over time.

This report was prepared for Highland Fairview Properties, LLC needs, directions, and requirements. This report is not authorized for use by, and is not to be relied upon by any party except Highland Fairview Properties, LLC, and its successors and assigns as owner of the property, with whom Leighton and Associates, Inc. has contracted for the work. Use of or reliance on this report by any other party is at that party's risk. Unauthorized use of or reliance on this report constitutes an agreement to defend and indemnify Leighton and Associates, Inc. from and against any liability which may arise as a result of such use or reliance, regardless of any fault, negligence, or strict liability of Leighton and Associates, Inc.

The client is referred to Appendix C regarding important information provided by the Associated Soil and Foundation Engineers (ASFE) on geotechnical engineering studies and reports and their applicability.



Project: 111061-126	Eng/Geol: SIS/RFR
Scale:1 " = 6,000 '	Date: September 2014

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

SITE LOCATION MAP

World Logistics Center Specific Plan
Moreno Valley, California

Figure 1

Leighton

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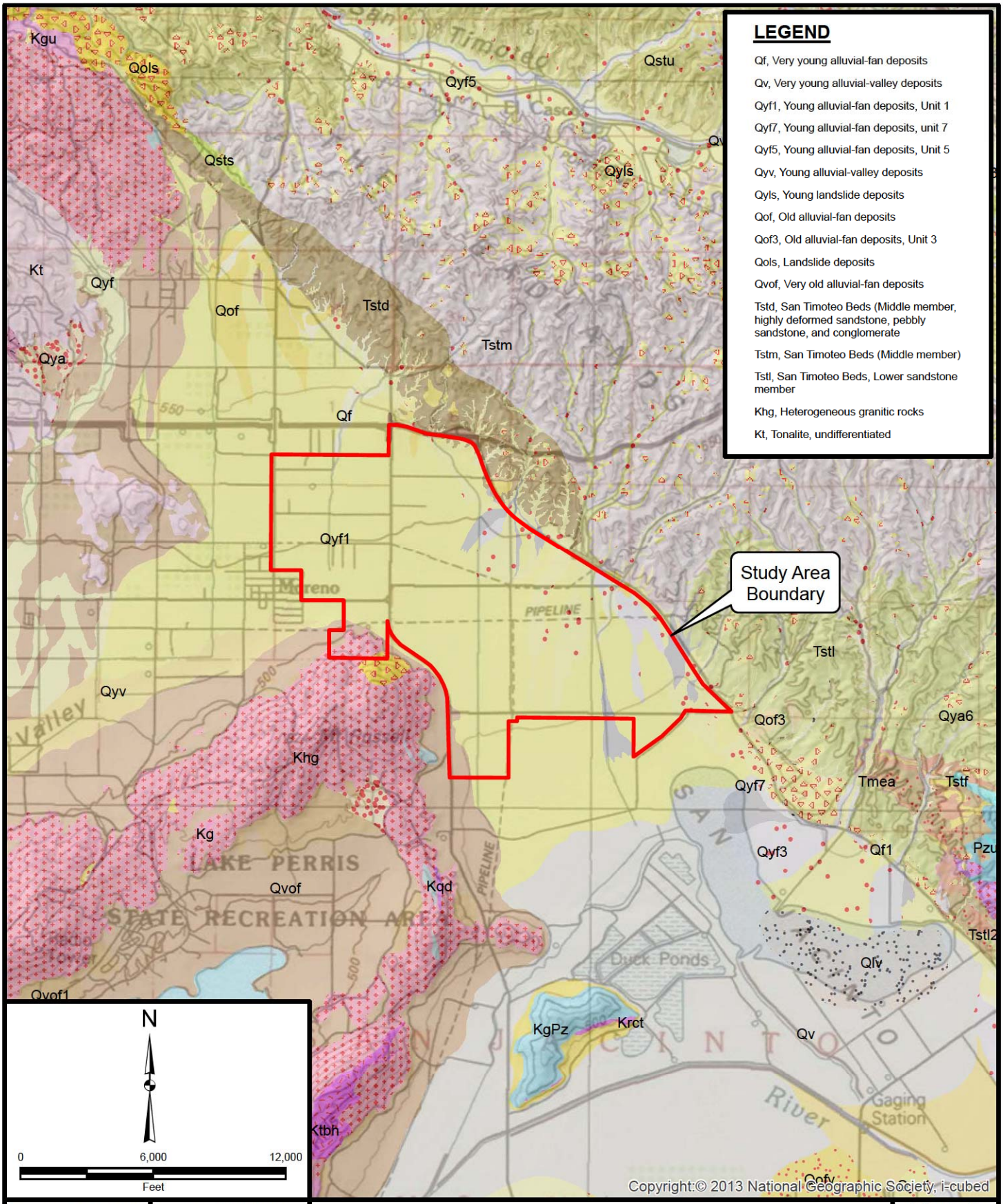

Project: 111061-126	Eng/Geol: SIS/RFR
Scale: 1 " = 6,000 '	Date: September 2014

Reference: USGS, 2006, Geologic map of San Bernardino and Santa Ana 30'x60' quadrangles, California, Version 1.0, Open File Report 206-1217
Author: (mmurphy)

REGIONAL GEOLOGY MAP

World Logistics Center Specific Plan
Moreno Valley, California

Figure 2

Leighton

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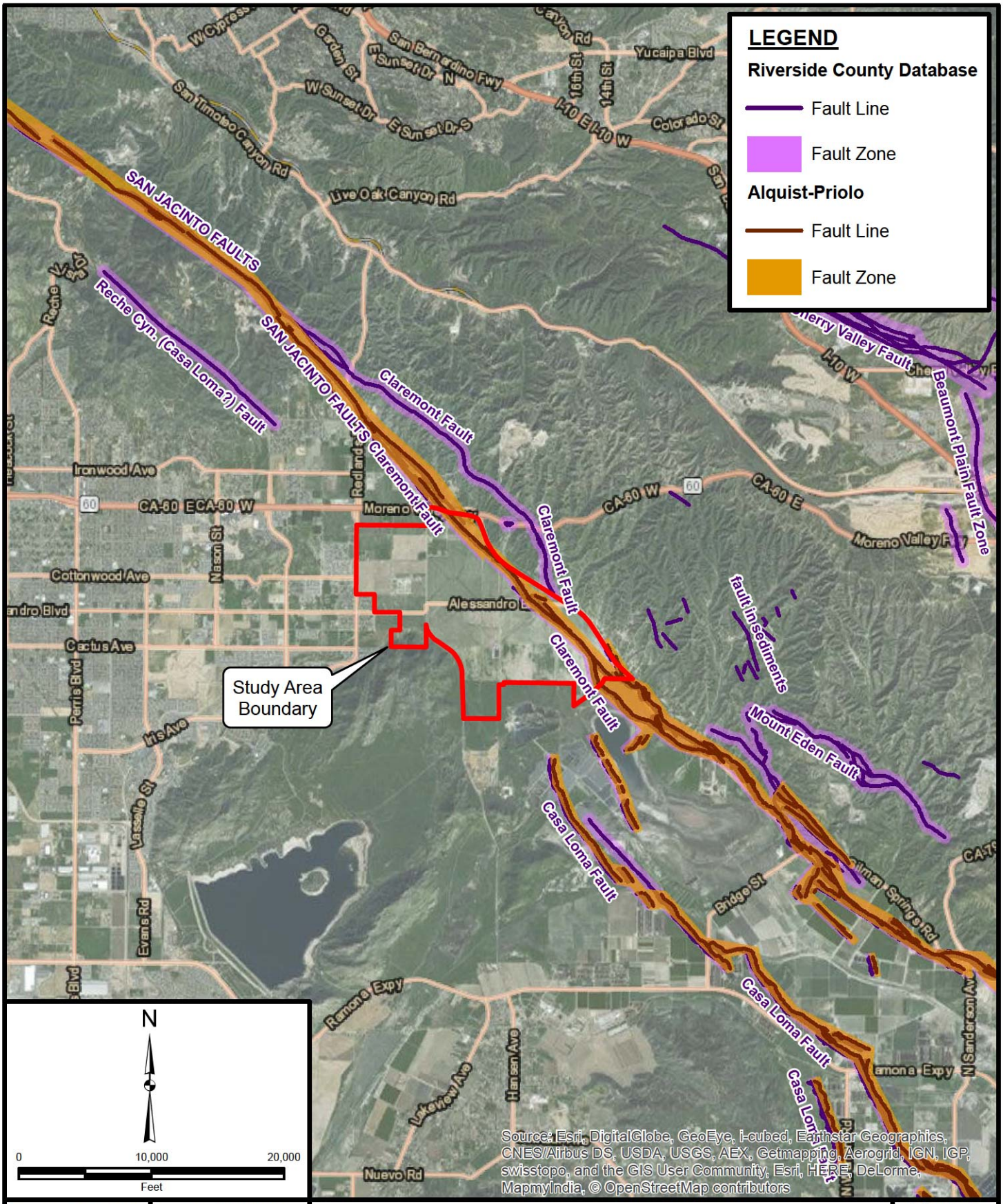


Project: 111061-126	Eng/Geol: SIS/RFR
Scale: 1 " = 10,000 '	Date: September 2014

Base Map: ESRI ArcGIS Online 2014
Faults: Riverside County, Alquist-Priolo

Author: (mmurphy)

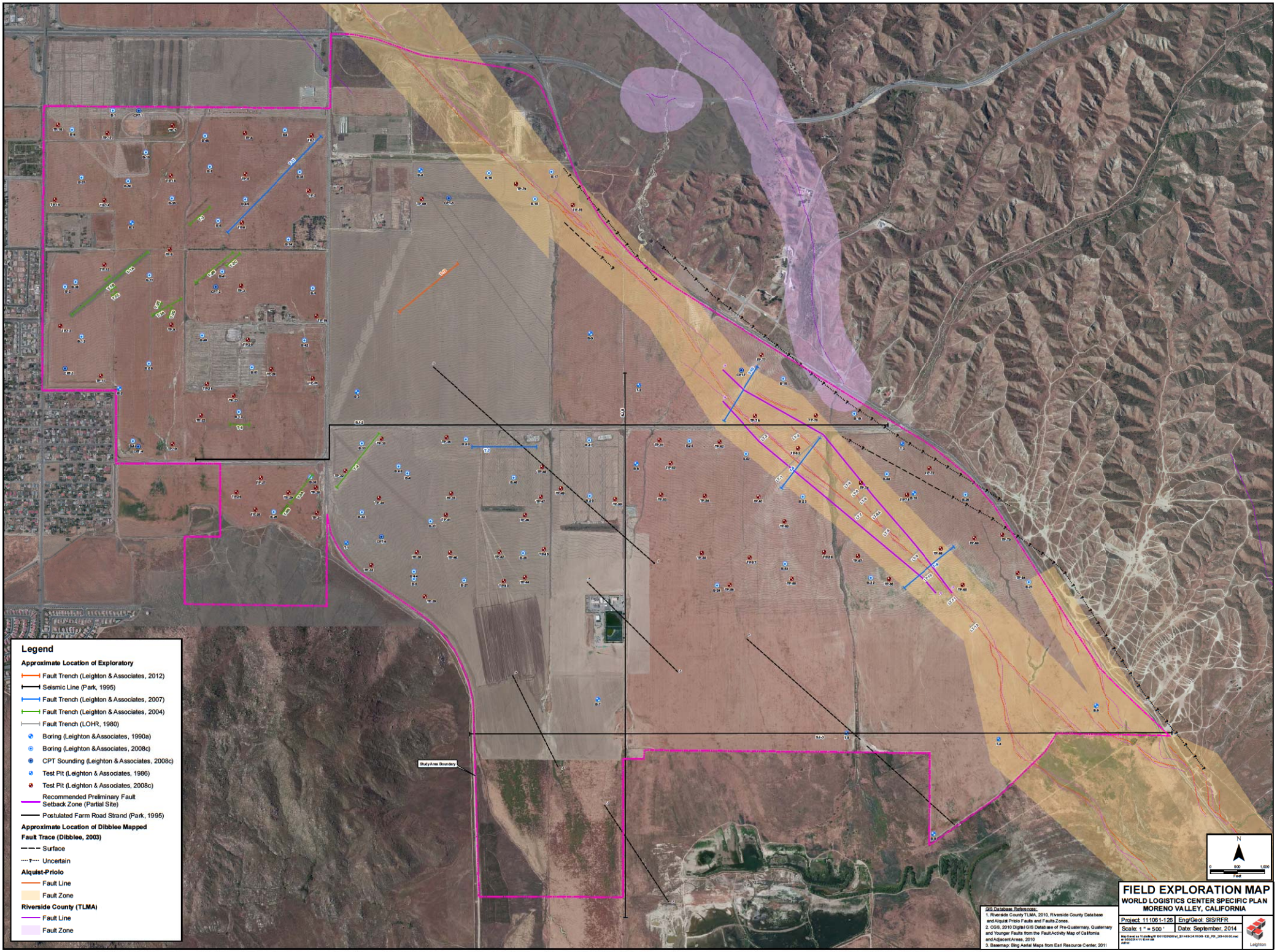
REGIONAL FAULT MAP

World Logistics Center Specific Plan
Moreno Valley, California

Figure 3

Leighton

Map Saved as P:\drafting\111061\126\Maps\111061-126_F03_RFM_2014-09-30.mxd on 9/30/2014 11:49:26 AM


Legend
Approximate Location of Exploratory
Fault Trench (Leighton & Associates, 2012)
Seismic Line (Park, 1995)
Fault Trench (Leighton & Associates, 2007)
Fault Trench (Leighton & Associates, 2004)
Fault Trench (LOHR, 1980)
Boring (Leighton & Associates, 1990a)
Boring (Leighton & Associates, 2008c)
CPT Sounding (Leighton & Associates, 2008c)
Test Pit (Leighton & Associates, 1986)
Test Pit (Leighton & Associates, 2008c)
Recommended Preliminary Fault Setback Zone (Partial Site)
Postulated Farm Road Strand (Park, 1995)
Approximate Location of Dibblee Mapped Fault Trace (Dibblee, 2003)
Surface
Uncertain
Alquist-Priolo
Fault Line
Fault Zone
Riverside County (TLMA)
Fault Line
Fault Zone
Study Area Boundary
GIS Database References:
1. Riverside County TLMA, 2010, Riverside County Database and Alquist Priolo Faults and Faults Zones.
2. CGS, 2010 Digital GIS Database of Pre-Quaternary, Quaternary and Younger Faults from the Fault Activity Map of California and Adjacent Areas, 2010
3. Basemap: Bing Aerial Maps from Esri Resource Center, 2011
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0 500 1,000
Feet
FIELD EXPLORATION MAP
WORLD LOGISTICS CENTER SPECIFIC PLAN
MORENO VALLEY, CALIFORNIA
Project: 11106-1-126
Eng/Geol: SIS/RFR
Scale: 1" = 500'
Date: September, 2014
Leighton

APPENDIX A

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APPENDIX B

General Earthwork and Grading Specifications

EARTHWORK AND GRADING SPECIFICATIONS

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STANDARD DETAILS

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B - Oversize Rock Disposal	Rear of Text
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E - Transition Lot Fills and Side Hill Fills	Rear of Text
Retaining Wall	Rear of Text

1.0 General

1.1 Intent

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

1.2 The Geotechnical Consultant of Record

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

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

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

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

1.3 The Earthwork Contractor

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

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

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

2.0 Preparation of Areas to be Filled

2.1 Clearing and Grubbing

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

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

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

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

2.2 Processing

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

2.3 Overexcavation

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

2.4 Benching

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

2.5 Evaluation/Acceptance of Fill Areas

All areas to receive fill, including removal and processed areas, key bottoms, and benches, shall be observed, mapped, elevations recorded, and/or tested prior to being accepted by the Geotechnical Consultant as suitable to receive fill. The Contractor shall obtain a written acceptance from the Geotechnical Consultant

prior to fill placement. A licensed surveyor shall provide the survey control for determining elevations of processed areas, keys, and benches.

3.0 Fill Material

3.1 General

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

3.2 Oversize

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

3.3 Import

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

4.0 Fill Placement and Compaction

4.1 Fill Layers

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

4.2 Fill Moisture Conditioning

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

4.3 Compaction of Fill

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

4.4 Compaction of Fill Slopes

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

4.5 Compaction Testing

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

4.6 Frequency of Compaction Testing

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

4.7 Compaction Test Locations

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

5.0 Subdrain Installation

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

6.0 Excavation

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

7.0 Trench Backfills

7.1 Safety

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

7.2 Bedding and Backfill

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

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

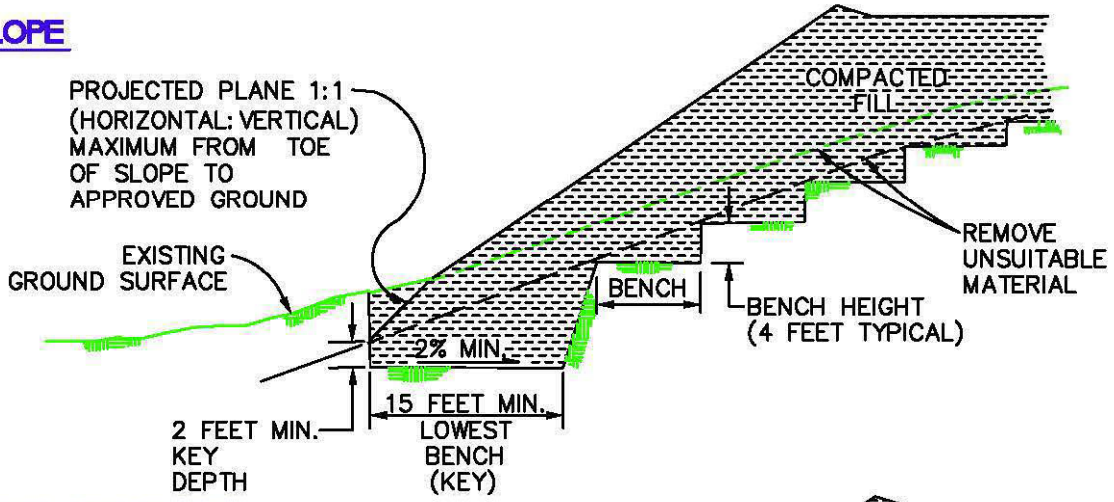
7.3 Lift Thickness

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

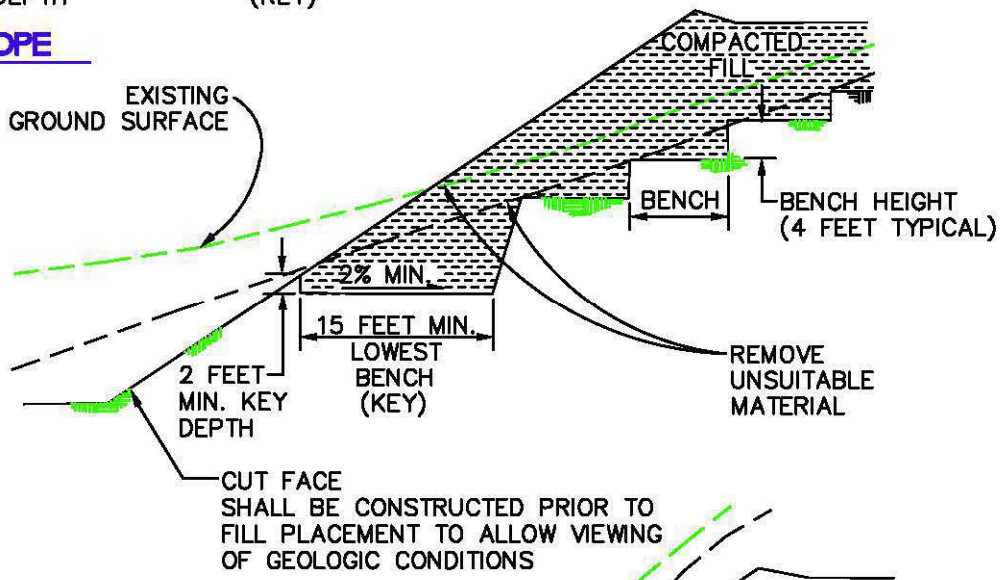
7.4 Observation and Testing

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

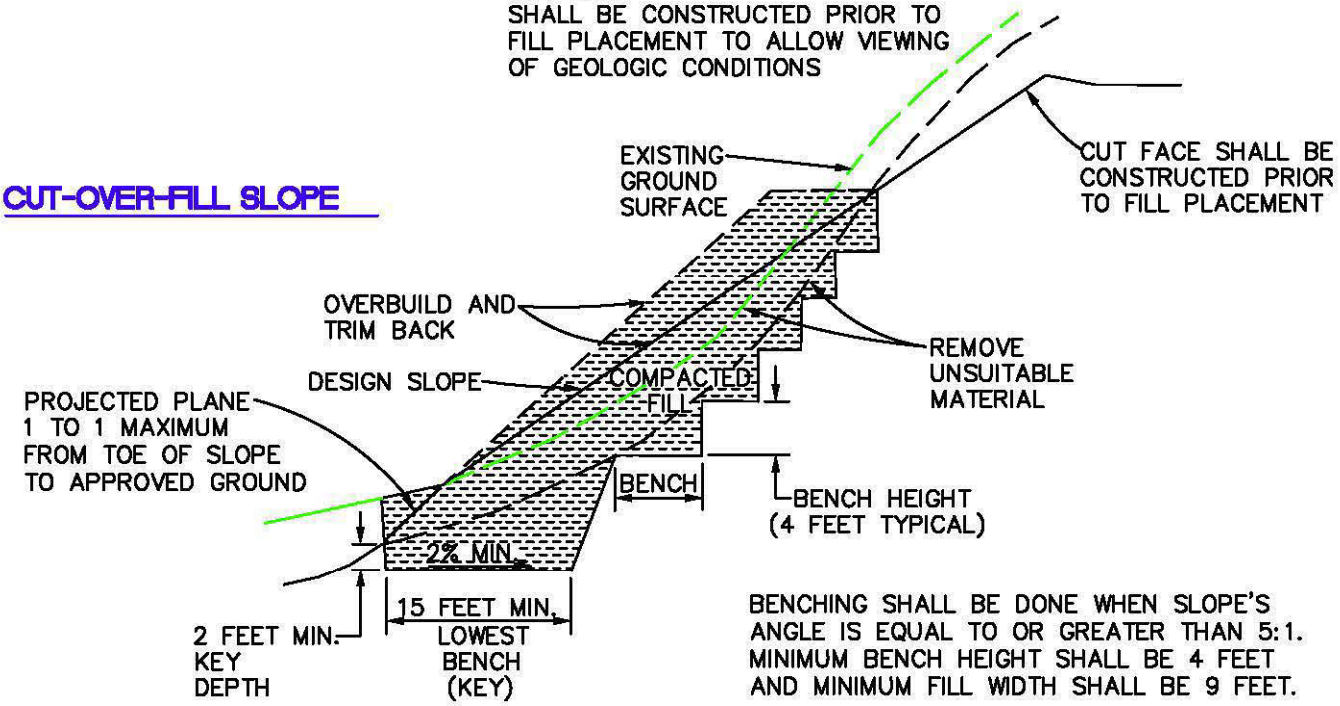
FILL SLOPE



FILL-OVER-CUT SLOPE

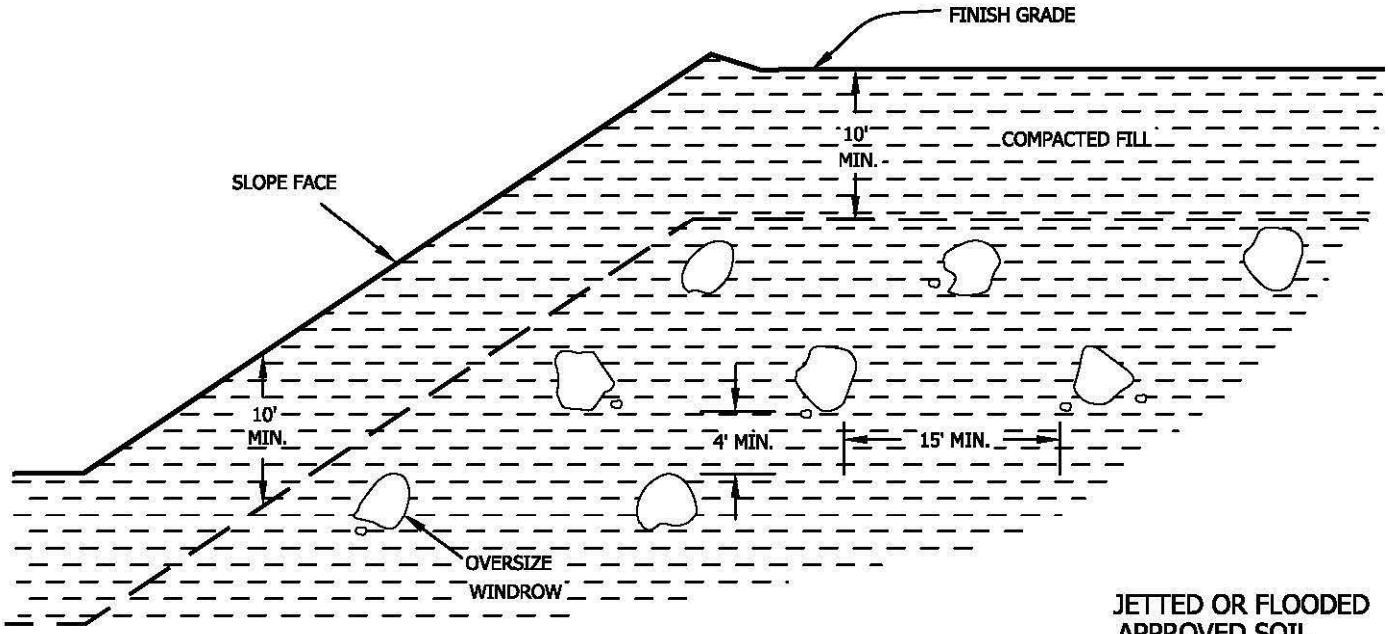


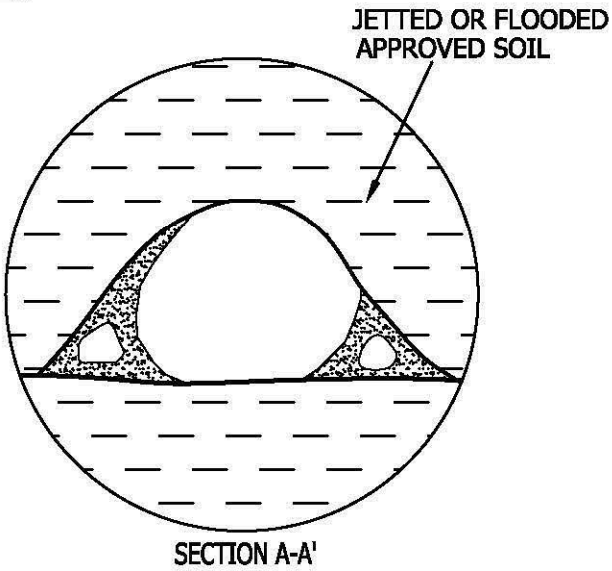
CUT-OVER-FILL SLOPE



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.

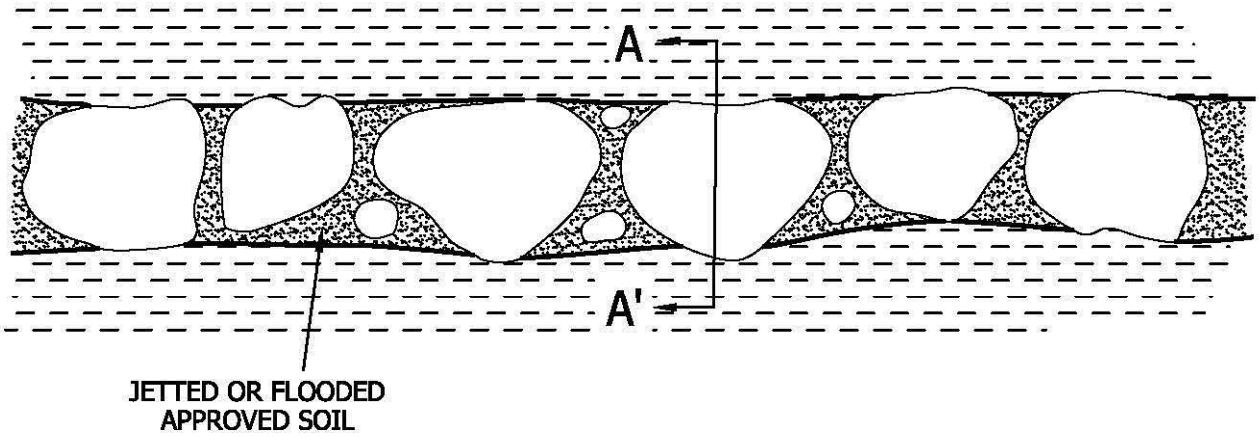
KEYING AND BENCHING	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAILS A	Leighton





- 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.

PROFILE ALONG WINDROW



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

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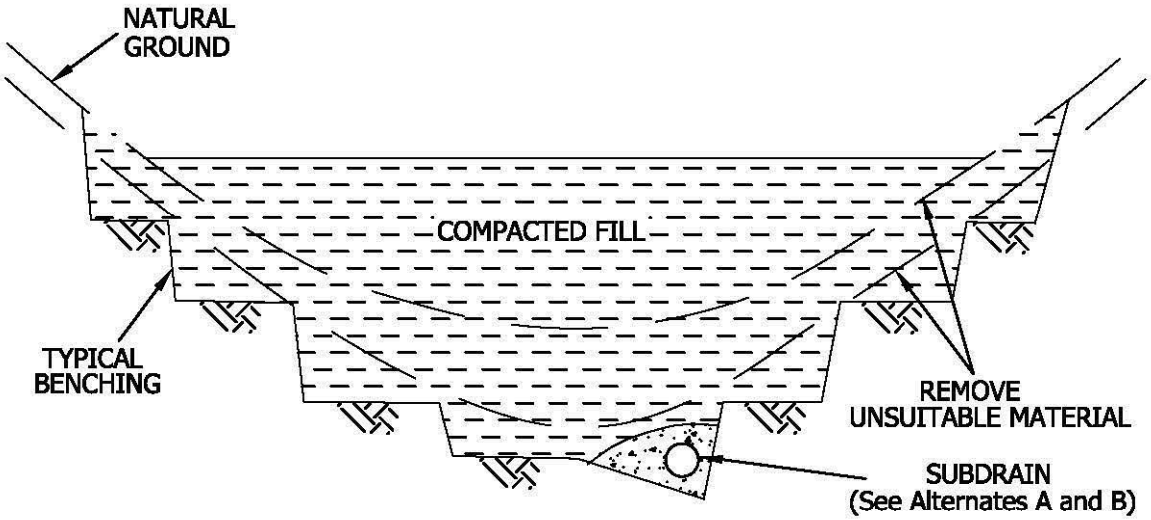
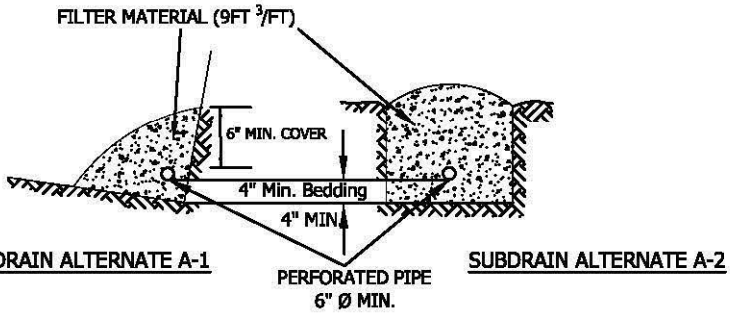


SUBDRAIN ALTERNATE A

PERFORATED PIPE SURROUNDED WITH FILTER MATERIAL

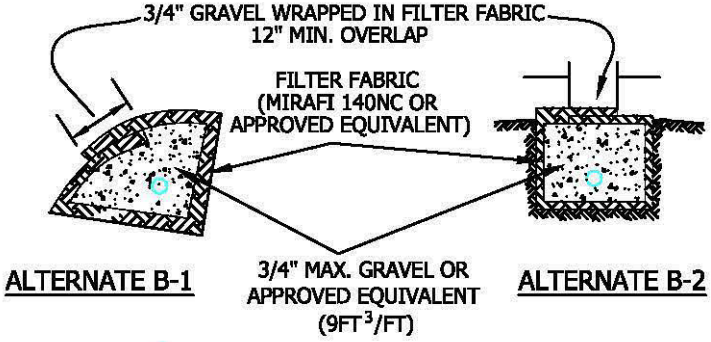


FILTER MATERIAL

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

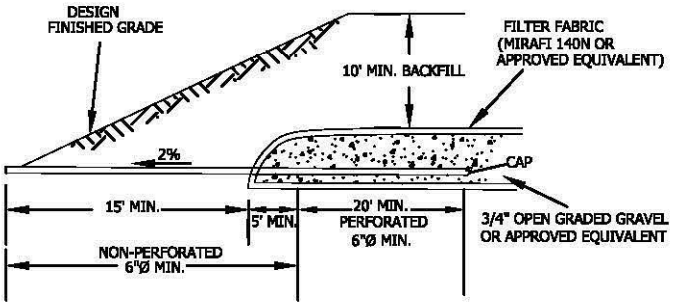
Sieve Size	Percent Passing
1"	100
3/4"	90-100
3/8"	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

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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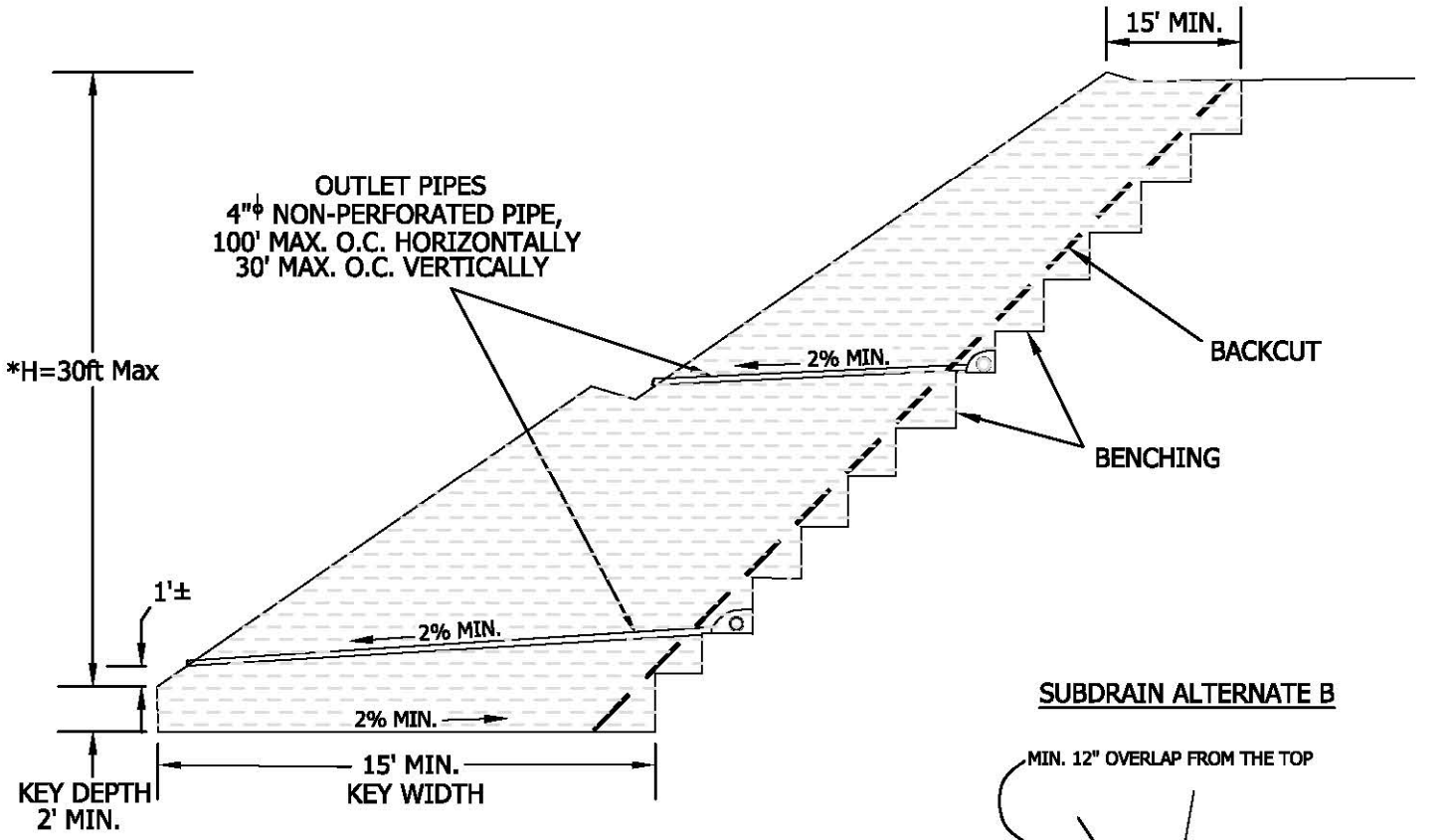

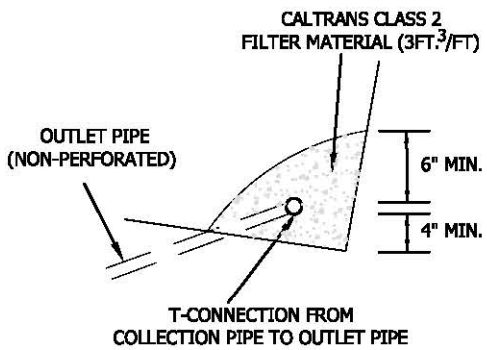

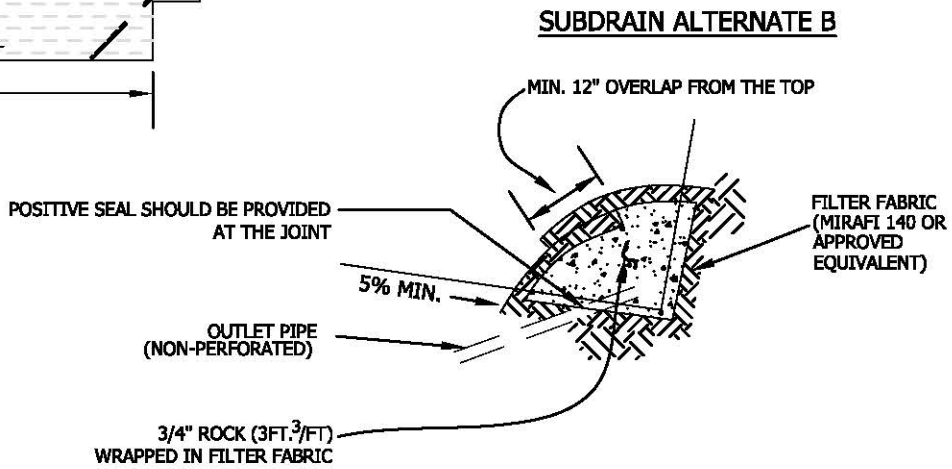

- SUBDRAIN INSTALLATION - Subdrain collector pipe shall be installed with perforations down or, unless otherwise designated by the geotechnical consultant. Outlet pipes shall be non-perforated pipe. The subdrain pipe shall have at least 8 perforations uniformly spaced per foot. Perforation shall be 1/4" 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.

* Slopes >30ft will require additional analysis/design

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

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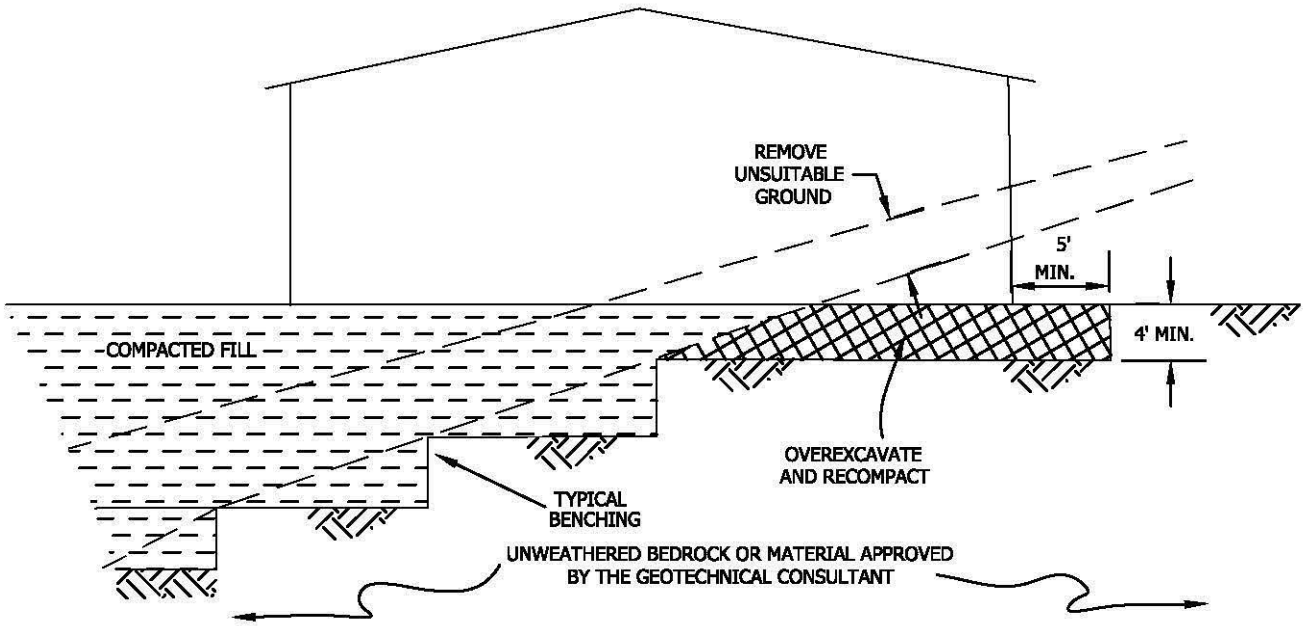
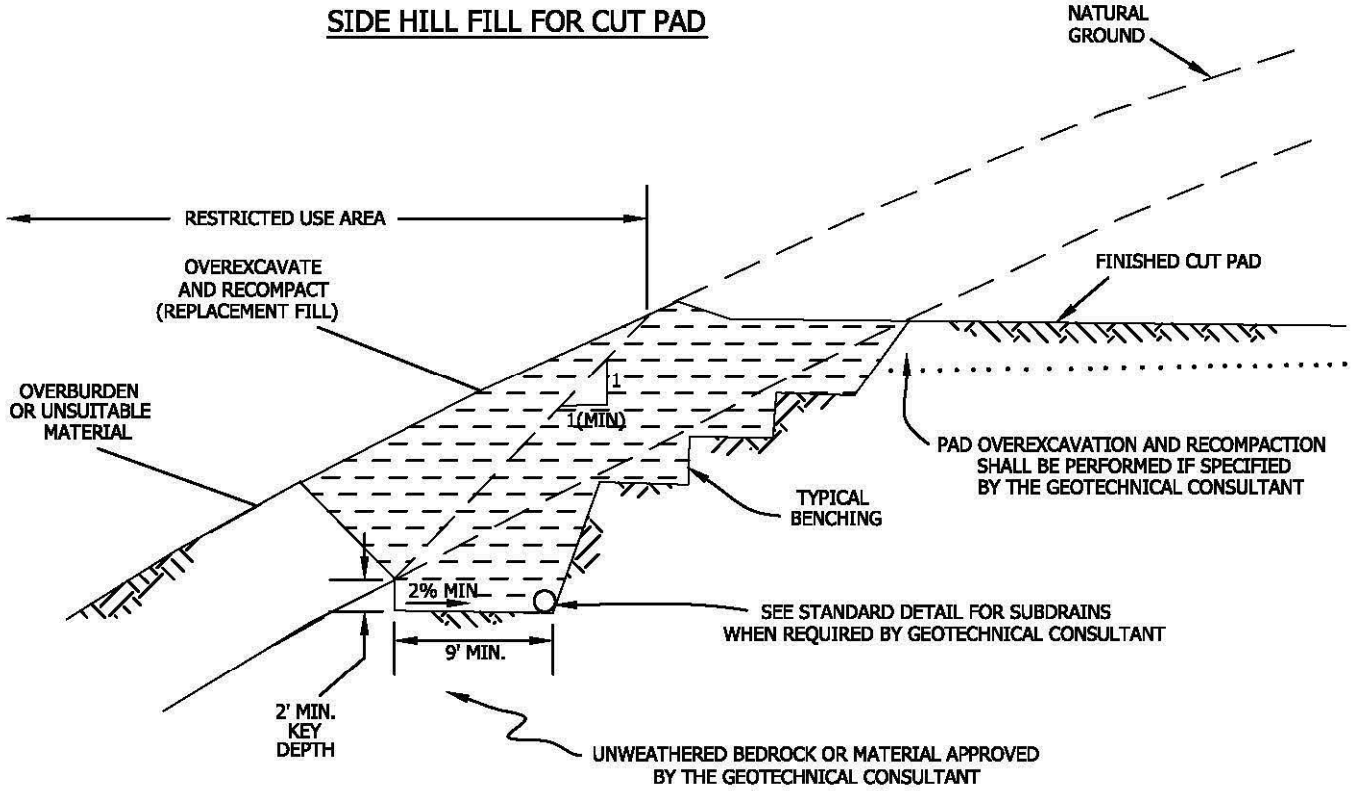
CUT-FILL TRANSITION LOT OVEREXCAVATION



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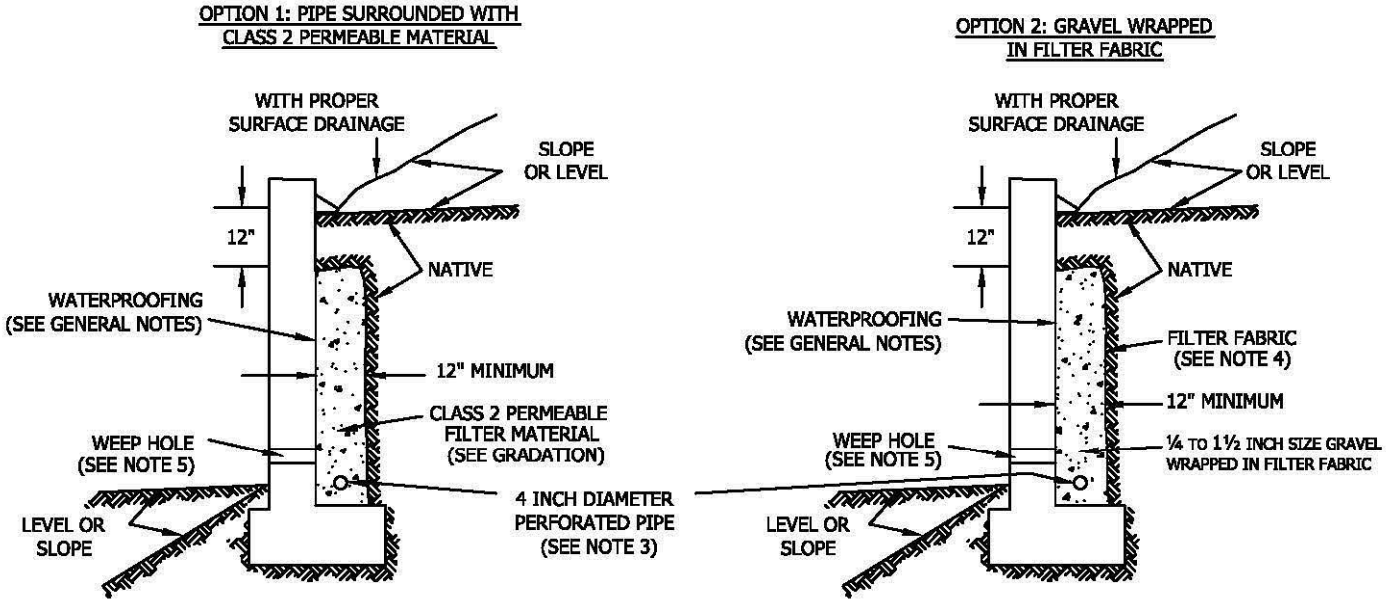
SUBDRAIN OPTIONS AND BACKFILL WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF ≤50



Class 2 Filter Permeable Material Gradation
Per Caltrans Specifications

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

GENERAL NOTES:

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

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

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

WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF ≤50

Leighton

Figure

APPENDIX C

ASFE, Important Information Regarding Geotechnical Engineering Studies

Important Information About Your

Geotechnical Engineering Report

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

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

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

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

Read the Full Report

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

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

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

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

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

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from 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 you ASFE-member geotechnical engineer for more information.

ASFE
THE BEST PEOPLE ON EARTH

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