11.7 Geology and Soils Reports

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Geology, Soils and Seismicity EIR Report

Lincoln Specific Plan, Whittier, California

February 28, 2014

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GEOLOGY, SOILS AND SEISMICITY ENVIRONMENTAL IMPACT REPORT
LINCOLN SPECIFIC PLAN EIR
Whittier, California

1.0 INTRODUCTION

The scope of work performed as part of the geology, soils and seismicity portion of the Lincoln (Site), Specific Plan Environmental Impact Report (EIR) included compilation and review of published geologic and seismic hazards maps, recent and historic black-and white and color aerial photographs, a Grading Exhibit prepared by FUSCO Engineering (dated November 22, 2013), and a geotechnical report for the Site by LGC Geotechnical, Inc. (LGC, dated April 26, 2013). A list of the reports, maps, and other relevant data reviewed for this study are presented in the References section at the end of this report.

As with all new construction, requirements for geotechnical and geologic/ seismic hazard studies are provided in Title 24 of the California Code of Regulations.

The results of the EIR-level evaluation for this study as well as pertinent impacts and mitigating measures are provided in the following report.

2.0 EXISTING CONDITIONS

2.1 GEOLOGIC SETTING

Regionally, the Site is located within the northeastern edge of the Los Angeles Coastal Plain on the La Habra Piedmont slope, an alluvial slope located near the foot of Puente Hills. This alluviated basin lies within an area west and northwest of the Peninsular Ranges and southeast of the Transverse Ranges geomorphic provinces. The region surrounding the Site is characterized by a broad, southerly sloping basin with intervening, low-lying hills (i.e. Coyote Hills), and several major rivers, namely, the San Gabriel and Santa Ana, that drain from the San Gabriel and San Bernardino Mountains, respectively.

The greater Los Angeles Basin began forming about 18 million years ago, during which time was represented by a number of intervening basins, which over time, had become infilled with

1,000's of feet of marine and non marine sedimentary and volcanic rocks. Beginning about 7 to 8 million years ago, the Los Angeles basin and surrounding environs were subjected to north-south compression associated with the onset of San Andreas transform fault system. The resulting San Andreas tectonic framework created most of the currently seismically active, northwest-southeast trending right-lateral strike slip faults of the Peninsular Ranges, as well as east-west-tending left-lateral and oblique slip faults of the western Transverse Ranges.

The nearby Whittier fault, which is located approximately one mile northerly of the Site displays faulting characteristics of both the Peninsular and Transverse Ranges provinces. Although there are no documented active or potentially active faults within or projecting towards the Site, a seismically active blind thrust fault known as the Puente Hills Blind Thrust fault underlies the Site at a depth of about 4.5 miles.

This portion of the Los Angeles Basin is mantled by approximately 3000 feet of Quaternary age (0 to 1.6 million years old) alluvial and fluvial sediment derived from eroded bedrock materials from the nearby Puente Hills and deposits laid down as flood plain deposits. These Holocene and late Pleistocene age sediments comprise the main groundwater-bearing strata in this portion of the eastern Los Angeles Basin. Beneath these non marine deposits are Pliocene to late Miocene age (1.6 to about 12 million years old) marine deposits, which include the Pico, Repetto, Modelo and Topanga formations attain a combined thickness in excess of 4,000 feet.

2.2 SITE CONDITIONS

According to the latest U.S. Geological Survey (USGS) topographic map for the Whittier 7.5 ' Quadrangle, the Site lies near the confluence of the eastern edge of the San Gabriel River flood plain and the distal portions of a broad alluvial fan emanating from nearby Puente Hills. Based on the latest grading plans for the project, the topography of the Site has an overall southwesterly-sloping surface with elevations ranging from about 230 feet above mean sea level (msl) along the northern margin to about 185 feet msl along the southern margin. The overall ground surface gradient is approximately 0.025. It appears that the majority of surface water runoff occurs primarily as sheet flow.

2.3 GEOLOGIC MATERIALS

The latest (i.e. 2010) geological mapping by the California Geological Survey (CGS) characterizes the native soils underlying the Site as young alluvial fan deposits. Prior mapping by the USGS in cooperation with the CGS also mapped these deposits as Holocene age alluvial fan and valley fill (i.e., flood plain) sediments. Subsurface data collected by LGC (2013) from their exploratory borings and test pits indicate the geologic materials that underlie the Site to the depths explored (i.e., 3 to about 52 feet below ground surface) include a surficial veneer of typically fine-grained topsoil and/or undocumented artificial fill soils, which are underlain fine-grained, native alluvial sediments..

Based on a review of the information presented in LGC's, 2013 report (Report), there appears to be, from a CEQA-level evaluation, sufficient data and analyses to adequately characterize the geologic and geotechnical engineering characteristics of the subsurface materials that could impact the proposed development.

2.3.1 Topsoil

Based on test pit and boring logs in the Report, the majority of the Site appears to be mantled with topsoil, which is characterized primarily as silty clay that is dark brown in color, moist to very moist, soft, and porous and contains scattered roots and rootlets. These deposits reportedly vary in thickness from about 2- to 5-feet-thick, and are considered unsuitable for foundation support for new roadways and structures (i.e., buildings and walls).

If not removed and/or replaced with properly engineered/ compacted fill beneath proposed buildings, the foundations and/or structural elements could experience moderate to significant distress.

2.3.2 Undocumented Artificial Fill

Undocumented artificial fill soils occur along a narrow, north-south-trending strip near the eastern margin of the Site. These man-made deposits are reported to consist mainly of silty and clayey soils that are mottled brown and dark brown, moist to very moist, soft, and contain scattered brick and concrete debris. These soils are considered "undocumented" because there apparently was no observation or density testing by a qualified geotechnical engineering firm when they were placed, and are likely derived from native topsoil materials.

Due to the lack of adequate documentation, as well as direct observations by LGC, these soils are considered to be compressible, and therefore subject to consolidation. If not removed and/or replaced with properly engineered/ compacted fill beneath proposed buildings, the foundations and/or structural elements could experience moderate to significant distress.

2.3.3 Holocene-Age Alluvial Deposits

Holocene age alluvial, and likely fluvial (i.e., flood plain) sediments directly underlie the surficial topsoil and undocumented fill soils beneath the entire Site. These natural soils consist of layers and lenses of fine-grained mixtures of silt, sand, and clay with lesser amounts of gravel. In general these silty sand layers are dusky brown to brown in color, moisture varying from dry to damp, generally medium dense, slightly porous to porous, contain scattered gravel, and occasional white-colored stringers of undetermined composition. Limited laboratory testing of these particular soils indicate they are normally consolidated, are not susceptible to hydroconsolidation, and are therefore suitable for foundation support for roadways and lightly loaded structures.

Layers of silty clay and clayey silt soils are generally light brown to reddish brown in color, moist to very moist, soft to firm, non-porous to porous, contain scattered small pebbles and some caliche (i.e., calcium carbonate) and medium-grained sand, occasional white-colored stringers of undetermined composition. Laboratory testing of these soils indicate they are normally consolidated and not susceptible to collapse/ hydroconsolidation, but are classified by 2001, California Building Codes (Table 18-I-B) as having a medium expansion potential.

Expansive soils are types of soil that shrink or swell as the moisture content decreases or increases. Structures built on these soils without mitigation may experience shifting, cracking, and tilting damage as the soils shrink or expand. Limited corrosion testing of these finer grained soils indicate they are very to extremely corrosive. Corrosive soils have the potential to damage metal and/or concrete. However, overall, these soils are suitable for foundation support for roadways and light loaded structures; additional reinforcement appropriate with the expansive nature of these soils will be necessary.

From a geotechnical perspective, the Holocene-age, and older, alluvial soils have sufficient soil engineering strengths to provide foundation support for the proposed two- to three-story structures. Given the relatively dense nature of these deposits, and the absence of any

significant slopes, there are no significant constraints associated with seismically-induced landsliding, lurching, or lateral spread. However, laboratory testing and analyses by LGC indicates that some isolated sandy layers are subject to seismically-induced liquefaction, which will be discussed later in Section 3.2.3 of this report.

2.4 GROUNDWATER

The uppermost 80 to 90 feet of the alluvium that underlies the Site is composed primarily of fine-grained sediments, similar to those encountered by LGC in their exploratory borings, which correlate to the Bellflower Aquitard. An aquitard is defined as a layer of rock or soil, which can be fully or partially saturated with groundwater, but has such low permeability they do not readably transmit water to wells, springs, or adjacent aquifers. Below depths of 80 to 90 feet below ground surface, groundwater is typically encountered within layers of water-bearing clayey silts, sandy silts, or fine-grained sands. In the immediate vicinity of the Site, moderately to high permeability sands and gravelly sands have been reported from well data below depths of 100 feet below the ground surface, which are correlative with either the Gage or Jefferson Aquifer. Historic high groundwater levels beneath the Site varies from about 30 feet in the western most portion of the Site, to about 60 feet along the eastern margin of the Site. However, more recent groundwater level data in the vicinity indicates depth to groundwater varies between about 70 to 80 feet.

No evidence of springs or seeps has been documented within or adjacent to the Site.

2.5 MINERAL RESOURCES

There are no documented metallic or non-metallic mineral deposits, oil or gas prospects, or significant paleontological (i.e., fossils) sites within the Site. The nearest oil fields are the Santa Fe Springs and Whittier fields that lie approximately 1.5 miles, and 1.8 miles northeast of the Site, respectively. According to a State of California Division of Oil, Gas, and Geothermal Resources District 1 (State) map, the closest plugged/ abandoned “dry” exploratory oil well is located approximately 700 feet east of the Site. This well is identified as Humble Oil and Refining Co, Whittier Opr., Unit one -1 (#141).

3.0 GEOLOGIC HAZARDS AND GEOTECHNICAL CONSTRAINTS

3.1 GENERAL

Given the historic land use, current site soil conditions, physiographic setting, and the nature of the proposed site grading and structures, there are several geologic hazards and geotechnical constraints that can be considered significant if not mitigated. These include the following:

- Seismically-induced moderate to strong ground shaking;
- Seismically-induced settlement and/or collapse of topsoil and artificial fill soils; and
- Expansive and corrosive soils.

Although the project area is located within a highly seismically active portion of the state, there are no documented active or potentially active faults transecting or projecting towards the Site. Moreover, there are no documented landslides within or adjacent to the project area.

Given the generally dense nature of the alluvial sediments, elevation and distance from the Pacific Ocean, and depth to groundwater, secondary seismic hazards due to moderate to strong earthquake ground motions from future earthquakes in the region (such as liquefaction, lateral spreading, seismically-induced landsliding, lurching, seiche, and tsunami run-up) do not present impacts on the proposed development.

Land subsidence that has in the past been associated with withdrawals of oil from a number of oil and gas fields in southern California (i. e. Wilmington, Inglewood, Buena Vista Hills, Santa Fe Springs, and Huntington Beach) has not been identified within the nearby Whittier oil and gas fields. Assuming the Santa Fe Springs and Whittier oil fields do not experience a large resurgence in oil production, the likelihood of land subsidence in the future is not considered significant.

3.2 FAULTING AND SEISMICITY

The project area is situated within a highly seismically active area of southern California referred to as the Los Angeles Basin. Hazards associated with earthquakes include primary seismic hazards, such as ground shaking and surface rupture, and secondary seismic

hazards, such as liquefaction, lateral spreading, seismically-induced settlement and landsliding, tsunamis, and seiche.

In accordance with the CGS, a fault is defined as a fracture in the crust of the earth along which rocks on one side have moved relative to those on the other side. Most faults are the result of repeated displacements over a long period of time. An inactive fault is a fault that has not experienced earthquake activity within the last three million years. In comparison, an active fault is one that has experienced earthquake activity in the past 11,000 years. A fault that has moved within the last two to three million years, but has not been proven by direct evidence to have moved within the last 11,000 years, is considered potentially active. No active or potentially active faults, which are capable of producing surface-fault rupture are located within or project towards the project area.

The Alquist-Priolo Act of 1972 (now the Alquist-Priolo Earthquake Fault Zoning Act, Public Resources Code 2621-2624, Division 2 Chapter 7.5) regulates development near active faults in order to mitigate the hazard of surface fault-rupture. Under the Act, the State Geologist is required to delineate “special study zones” along known active faults in California. The Act also requires that, prior to approval of a project, a geologic study be conducted to define and delineate any hazards from surface rupture. A geologist registered by the State of California, within or retained by the lead agency for the project, must prepare this geologic report.
A 50-foot setback from any known trace of an active fault is required. The project area is not currently known to be located within an Alquist-Priolo Earthquake Fault Zone, according to the CGS. The closest Earthquake Fault Zone to the Site is the Whittier fault located about one mile to the north. Other seismically active faults in the surrounding region are presented in Table 1- LIST OF ACTIVE FAULTS.

Ground shaking accompanying earthquakes on nearby faults can be expected to produce the potential for strong ground motion during the design life of the proposed project. However, the intensity of ground shaking would depend upon the magnitude of the earthquake, the distance to the epicenter, and the geology of the area between the epicenter and the Site. The Modified Mercalli Intensity (MMI) scale was developed in 1931 and measures the intensity of an earthquake’s effects in a given locality, and is perhaps much more meaningful to the layman because it is based on actual observations of earthquake effects at specific places. On the MMI scale, values range from I to XII. The most commonly used adaptation covers the

range of intensity from the conditions of: "I "- not felt except by very few, favorably situated, to "XII" – damage total, lines of sight disturbed, and objects thrown into the air." While an earthquake has only one magnitude, it can have multiple intensities, which decrease with distance from the epicenter. In the case of the 1994 Northridge earthquake, the Santa Clarita/ Newhall area experienced MMIs between VII and VIII (Dewey, et. al., 1995). Ground motions, on the other hand, are often measured in percentage of gravity (percent g), where g = 32 feet per second per second (980 cm/sec^2) on the earth.

3.2.1 Strong Seismically-Induced Ground Motion

Based on the California Division of Mines and Geology Seismic Hazard Zone Report for the Whittier 7.5-Minute Quadrangle that applies peak ground acceleration (PGA) for 10% exceedance in 50 years for alluvial-type soils, the level of ground motion at the Site is expected to be approximately 0.46 g. However, based on site-specific studies presented in the Report that follow the guidelines in Chapter 16 Section 1613 of the 2010 California Building Code (CBC), the design PGA for the Site is somewhat higher, 0.53g.

A listing of active faults considered capable of producing moderate to strong ground motion at the Site, their closest distances to the property, and the maximum expected earthquake (M) along each fault, as well as fault characteristics are presented in Table 1.

The greatest amount of ground shaking at the site would be expected to accompany large earthquakes on the nearby Whittier fault, the Puente Hills (Santa Fe Springs or Coyote Hills segments) Blind Thrust faults, Newport-Inglewood fault, or the Sierra Madre fault. Earthquake magnitudes in the range of M6.5 to M7.3 could produce MMI in the range of VIII to XI within the property.

The potential for secondary earthquake hazards, including ground deformation associated with liquefaction, lurching, lateral spreading, seismically-induced settlement, earthquake-induced landsliding, tsunamis, and seiche, is considered nil.

3.2.2 Liquefaction

Seismic ground shaking of relatively loose, granular soils that are saturated or submerged can cause the soils to liquefy and temporarily behave as a dense fluid. Liquefaction is caused by a sudden temporary increase in pore water pressure due to seismic densification or other

displacement of submerged granular soils. Liquefaction more often occurs in earthquake-prone areas underlain by young (i.e., Holocene age) alluvium where the groundwater table is less than 50 feet below ground surface.

The CGS has designated certain areas within California as potential liquefaction hazard zones. As shown on the State of California's Seismic Hazard Zone Map for the Whittier 7.5' Quadrangle, the southern and southwestern portions of the Site lie within an area that is considered to be potentially susceptible to earthquake-induced liquefaction. Site-specific liquefaction analyses performed by LGC (2013) for the Site indicate that isolated sandy layers in the underlying Holocene-age alluvium are subject to liquefaction during a significant earthquake. However, the results of their analyses, using the historic high groundwater level of 30 feet below the existing ground surface, indicates that the total seismic-related soil settlements would be on the order of approximately ½ inch within the upper 50 feet of the alluvium. Given this very limited amount of potential settlement that would be spread over a soil thickness of 50 feet, the potential impact to proposed structures and roads would be negligible.

3.2.3 Lateral Spreading

Lateral spreading involves the dislocation of near surface soils generally along a near-surface liquefiable layer. In many cases, this phenomenon of shallow landsliding occurs on relatively flat or gently sloping ground adjacent to a "free face," such as a river embankment. Given the Site conditions, the likelihood for lateral spread beneath the Site during a major earthquake in the area is nonexistent.

3.2.4 Seismically-Induced Landsliding

Based on the essentially flat nature of the Site, and the absence of any adversely-oriented, weak clay beds, the potential for seismically-induced landsliding within the Site is considered nil.

3.2.5 Seismically-Induced Soil Settlement

Strong ground shaking can cause settlement by allowing sediment particles to become more tightly packed, thereby reducing pore space. Unconsolidated, loosely packed sediments, such as native topsoil and young alluvial are especially susceptible to this phenomenon. Poorly compacted (i.e, non-engineered) artificial fills may also experience seismically-induced

settlement. Based on the subsurface data obtained by LGC (2013), the Site is mantled with topsoil and in certain places by non-engineered (i.e., loose) fill soils that are likely subject to seismically-induced settlement and/ or development of ground cracking.

Remedial grading during Site development will remove these settlement-prone soils and replace them with properly compacted engineered fill. Therefore, once the Site is graded, the potential for seismically-induced settlement and ground cracking is considered nil.

3.2.6 Flooding/Tsunami Run-Up

Flood hazards include storm-induced flooding, and those caused by earthquakes, namely tsunami and dam failure. According to the latest (September 26, 2008) Flood Insurance Rate Map (FIRM) prepared by Federal Emergency Management Agency (FEMA), the Site does not lie within either a 100-year or 500-year flood area, or within a dam inundation area.

The California Emergency Management Agency (Cal EMA), in cooperation with CGS, produced a Tsunami Inundation Map (dated March 1, 2009) that depicts the Site and surrounding neighborhood lying outside a tsunami inundation area.

3.2.7 Ground Lurching

Lurching is a phenomenon in which loose to poorly consolidated deposits move laterally as a response to strong ground shaking during an earthquake. Lurching is typically associated with soil deposits on or adjacent to steep slopes. Lurching that occurred in the Santa Monica and Santa Susana mountains during the 1994 Northridge earthquake usually was attributed to the outer two to eight feet of loose fill soils that spilled over the edges of graded pads cut into bedrock. Graded and compacted housing pads did not experience lurching during this very damaging earthquake.

Certain soils have been observed to move in a wave-like manner in response to intense seismic ground shaking, forming ridges or cracks on the ground surface. Areas underlain by thick accumulations of young alluvium appear to be more susceptible to ground lurching than bedrock. Under strong seismic ground motion conditions, lurching can be expected within loose, cohesionless soils, or in clay-rich soils with high moisture content. Generally, only lightly- loaded structures such as pavement, fences, pipelines and walkways are damaged by ground lurching; more heavily loaded structures appear to resist such deformation. Based on

the proposed grading, and the relatively dense nature of the alluvial soils, the likelihood that lurching will affect the proposed development is considered remote.

3.2.8 Seiching

Seiching involves an enclosed body of water oscillating due to ground shaking, usually following an earthquake. Lakes and water towers are typical bodies of water affected by seiching. Given that there are no large, enclosed open bodies of water or reservoirs upgradient of the project area, the potential for seiching is nil.

3.2.9 Other Geological Hazards

Corrosive Soils

Corrosive soils contain chemical constituents that can react with construction materials, such as concrete and ferrous metals, that may cause damage to foundations and buried pipelines. One such constituent is water-soluble sulfate which, if in high enough concentration, can react with and damage concrete. Electrical resistivity and pH levels are indicators of the soil's tendency to corrode ferrous metals. According to limited laboratory testing by LGC (2013), near-surface native alluvial soils have a neutral to relatively mildly alkaline pH value (7.1 to 7.6), and minimum resistivity values ranging from 650 to 1.500 ohm-cm, indicating these soils are considered severely corrosive to ferrous metals in contact with these soils. Laboratory tests by LGC (2013) also indicate water soluble sulfate contents of less than 0.02 percent, which is an indication of negligible sulfate exposure. As such, no particular recommendations for cement type or water ratio were deemed necessary by LGC (2013) to provide sulfate resistance.

Overall, LGC (2013) indicates that onsite soils are corrosive to ferrous metals. Recommendations to mitigate this condition will involve the developer/builder hiring a corrosion specialist to develop a mitigation plan upon completion of rough grading. This is a typical approach to this potentially significant issue.

Expansive Soils

Expansive soils are clay-rich soils that can undergo significant increase in volume with increased water content and significant decrease in volume with a decrease in water content.

Significant changes in moisture content within moderately to highly expansive soil can produce cracking, differential heave, and other adverse impacts to structures constructed on such soils. Based on the results of laboratory testing performed by LGC (2013) on seven (7) clayey soil samples, the native soils exhibit a "moderate expansion potential." Therefore, the potential for expansive soils to impact new development is considered significant.

Subsidence

The extraction of groundwater or oil from sedimentary source rocks can cause the permanent collapse of pore space that was previously occupied by the removed fluid. The compaction of subsurface sediments resulting from fluid withdrawal can and has caused the ground surface overlying fluid reservoirs to subside. If sufficiently great, the subsidence can cause significant damage to nearby engineered structures. As stated earlier in this report, the Site is not situated within an active or historic oil or gas field, but does lie within about 1.5 miles from two existing oil fields.

According to Lamar (et. al., 1973), no significant subsidence associated with fluid withdrawal (groundwater or petroleum), peat oxidation, or hydrocompaction has occurred in the City of Whittier.

Soil Erosion

Soil erosion is most prevalent in more weakly consolidated (i.e., highly weathered) surficial soils, which are prone to downcutting during and after heavy rainstorms. Strong wind forces can also produce varying amounts of soil erosion of the more weakly consolidated surficial soils.

The short-term effects of soil erosion during rough grading for the project are not considered significant given that the Site is essentially flat and does not possess site conditions necessarily conducive to soil erosion. It is anticipated that temporary screen walls will be placed around the perimeter of the Site during rough grading, which should help mitigate wind-related soil erosion. Therefore, the potential for short-term soil erosion is considered nil.

Sloughing or Caving of Excavations

During construction for the project, excavations associated with remedial grading will encounter topsoil and existing artificial fill soils. If unsupported, these soils may be subject to sloughing and caving, hence creating a short-term hazard to construction workers and equipment.

Slope Stability

The current grading plan prepared by FUSCO Engineering indicates construction of a number of proposed 2:1 (horizontal to vertical), 6- to 16-foot-high fill slopes associated with construction of a number of large pads, and 3- to-5-foot-high 2:1 fill slopes along the margins of the Site. Although there has not been slope stability analyses performed for these slopes, a forthcoming 40-scale geotechnical review report for the latest grading plan will address both the long-term surficial and gross stability of the slopes, together with grading recommendations to provide an adequate factor of safety against either form of instability.

Prime Farmland

There is no indication that the Site was used for farming or other agricultural purposes.

4.0 THRESHOLDS OF SIGNIFICANCE

Earth resource and/or topographic impacts resulting from the proposed project could be considered significant if any of the following occur:

- Exposure of people or property to substantial geological hazards, such as flooding due to dam or reservoir failure, landslides, mudslides, ground failure or similar hazards; or soil and/or seismic conditions so unfavorable that they could not be overcome by design using reasonable construction and/or maintenance practices;

- Location of a structure within a mapped hazard area or within a structural setback zone;

- Location of a structure within an Alquist-Priolo Fault-Rupture Hazard Zone, or an area characterized by surface rupture that might be related to a fault;
- Triggering or acceleration of geologic processes, such as landslides or erosion that could result in slope or embankment/levee failures;
- Substantial irreversible disturbance of the soil materials at the Site or adjacent sites, such that their use is compromised;
- Modification of the surface soils such that abnormal amounts of windborne or waterborne soils are removed from the Site;
- Earthquake-induced ground shaking capable of causing ground rupture, liquefaction, soil settlement, landsliding resulting in substantial damage to people and/or property;
- Deformation of foundations by expansive soils (those characterized by shrink/swell potential) or collapsible soils; and/or
- Modification of the on-site topography (i.e., grading) in a manner that results in decreased stability for adjacent residential, commercial or industrial enclaves.

5.0 IMPACTS

The level of geotechnical and landform information contained herein is adequate to analyze the potential project effects on earth resources and landforms, and to determine appropriate mitigation measures for the proposed development. In accordance with CEQA case law, these later additional refinements are not a deferral of mitigation. Rather, it is a design refinement, consistent with the commitment to mitigation included in this EIR.

Essentially, there are only several impacts to the current physical/geological setting that can generally be expected from grading and development activities associated with the proposed development. These are discussed in Section 5.2.

5.1 EFFECTS FOUND NOT TO BE SIGNIFICANT

Based on the results of the information reviewed for this study, those geologic hazards that are not considered to represent significant impacts due to their absence or low potential within the Site include: ground surface rupture associated with active faulting, secondary seismic hazards, soil erosion, landsliding, subsidence from oil and groundwater extraction, topography, and loss of prime farmland.

Grading activities associated with the development and construction of new buildings, underground utilities, and associated parking areas would create minor changes to the current topography. The greatest changes to existing topography would occur from construction of new slopes and structures. Only by avoidance can impacts to topography and taller building(s) be mitigated and/or reduced to a less-than-significant level.

5.2 POTENTIALLY SIGNIFICANT CONSTRUCTION-RELATED IMPACTS

The most significant potential impacts to the project are those resulting from strong seismically-induced ground motion, settlement of topsoil and undocumented fill soils, expansive soils, sloughing/caving of excavations, and soils that are considered to be severely corrosive to metallic pipes.

5.2.1 Strong Seismically-Induced Ground Motion

Given the proximity to major active faults, severe ground motion should be expected at the site. All structures associated with the proposed development should be designed to withstand the “design-level” earthquake as set forth in the latest edition of the CBC. Potential adverse impacts to new structures due to strong, seismically-induced, vibratory ground motion can be reduced to a less-than-significant level with proper seismic design.

5.2.2 Seismically-Induced Soil/Ground Settlement

The susceptibility of topsoil and undocumented fill soils to seismically-induced settlement presents a potentially significant impact to the Site. The methods to mitigate seismically-induced soil settlement involve removal and replacement of these soils with properly engineered fill.

5.2.3 Slope Stability

Proposed fill slopes may be prone to instability either statically, or during severe ground shaking from a future seismic event. Based on the CBC (2010) all 2:1 slopes will be required to have a minimum factor of safety of 1.5.

5.2.4 Corrosive Soils

Near surface native alluvial soils are corrosive to metals in contact with these soils. It is anticipated that the future geotechnical engineering studies to be performed for the proposed development will further evaluate the nature and extent of these types of soils. At a minimum, buried metal piping should be protected with suitable coatings, wrapping, or seals; and a corrosion engineer should be consulted during future, site-specific geotechnical studies.

5.2.5 Expansive Soils

Expansive soils are clay-rich soils that can undergo significant increase in volume with increased water content and significant decrease in volume with decreased water content. Significant changes in moisture content within moderately to highly expansive soil can produce cracking, differential heave, and other adverse impacts to structures constructed on such soils. Based on the results of the laboratory test performed by LGC, both native and undocumented clay soils are anticipated to exhibit a moderate expansion potential and, therefore, the potential for expansive soils to impact new development is considered high.

As recommended by LGC (2013), clayey soils are not suitable for backfill behind any proposed retaining walls, and import of sandy soils meeting project specifications will be required. In addition, for footings, foundations and slab-on-grade floors for new structures mitigation measures, such as post tensioned slabs, additional reinforcement of footings, etc., will be required.

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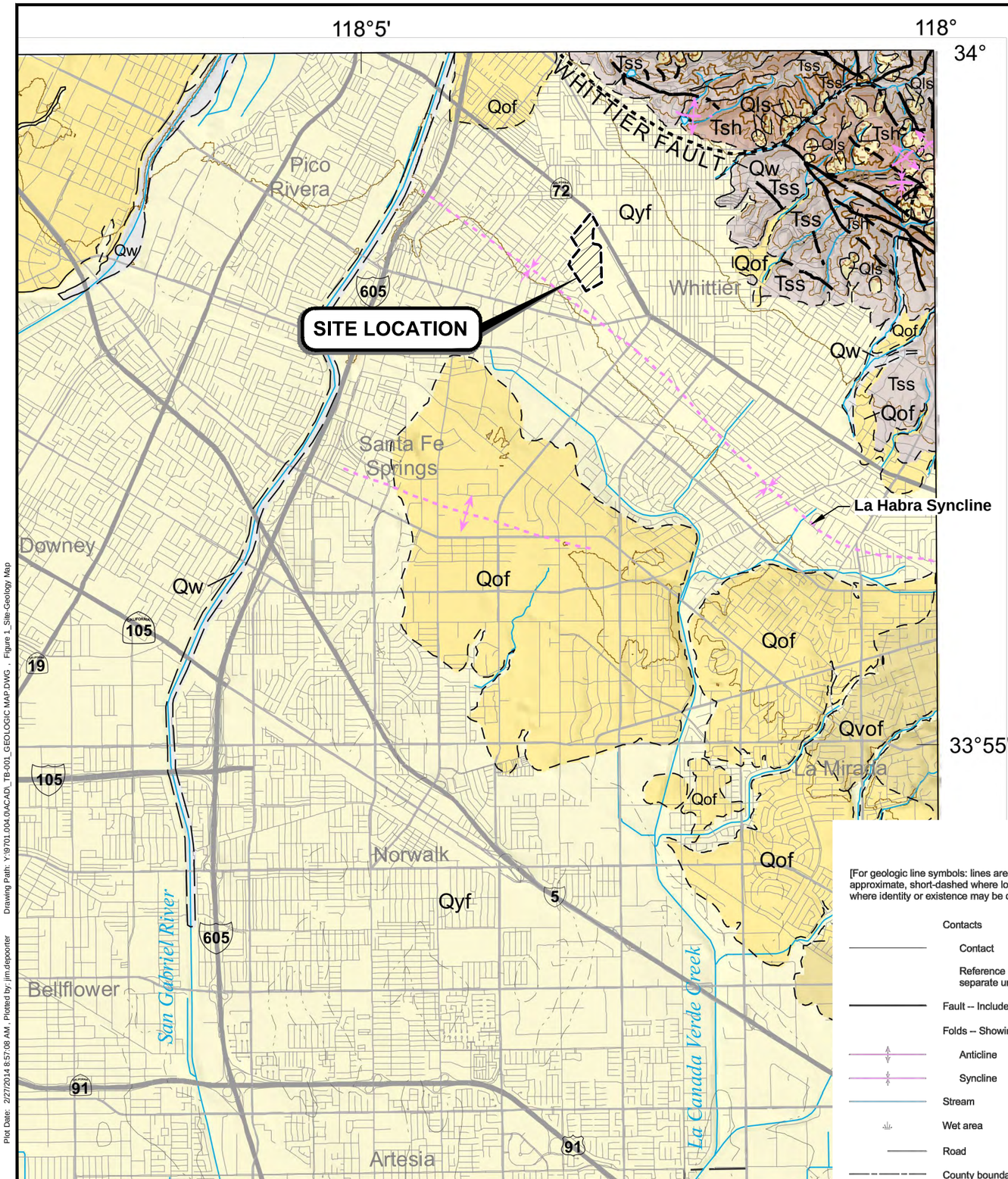
AERIAL PHOTOGRAPHS REVIEWED

Source	Date of Photographs	Flight No/ Frame No.	Scale
Fairchild	1928	K403	1"= 500'
Laval	1938	?	1"= 555'
Fairchild	1947	?	1"= 666'
USGS	1989	?	1"= 666'
Eagle Aerial	2/28/11	2011 S/ Los Angeles	Regional

TABLE 1: LIST OF ACTIVE FAULTS

Seismic Fault	Source Model	Closest Distance (km)	Moment Magnitude (Mw)	Fault Mechanism	Fault Dip Angle (degrees)	Fault Dips to	Site Lies
Whittier	USGS 2008	1.69	7.849	Oblique SS	75-90	NE	SW
Puente Hills (Santa Fe Springs)	USGS 2008	5.36	6.7	Reverse	29	N	N
Puente Hills	USGS 2008	6.62	7.1	Reverse	25	N	Above
Puente Hills (Coyote Hills)	USGS 2008	7.29	6.9	Reverse	26	N	NW
Puente Hills (LA)	USGS 2008	7.31	7	Reverse	27	N	E
Elysian Park (Upper)	USGS 2008	11.28	6.7	Reverse	50	N	SE
Raymond	USGS 2008	16.9	6.8	Reverse	79	N	S
San Jose	USGS 2008	17.19	6.7	Strike Slip	74	NW	SW
Verdugo	USGS 2008	19.36	6.9	Reverse	55	NE	SE
Newport-Inglewood	USGS 2008	20.88	7.5	Strike Slip	88-90	NE	NE
Sierra Madre	USGS 2008	21.11	7.2	Reverse	53	N	S
Sierra Madre Connected	USGS 2008	21.11	7.3	Reverse	51	N	S
Clamshell-Sawpit	USGS 2008	22.57	6.7	Reverse	50	NW	S
Hollywood	USGS 2008	22.68	6.7	Strike Slip	70	N	SE
Chino	USGS 2008	23.25	6.8	Strike Slip	50-65	SW	W
Santa Monica	USGS 2008	25.87	7.4	SS|R	44-75	N	E
Palos Verdes	USGS 2008	31.87	7.3	Strike Slip	90	--(a)	NE
Palos Verdes Connected	USGS 2008	31.87	7.7	Strike Slip	90	--	NE
San Joaquin Hills	USGS 2008	33.47	7.1	Reverse	23	SW	N
Cucamonga	USGS 2008	33.82	6.7	Reverse	45	N	SW
Sierra Madre (San Fernando)	USGS 2008	40.11	6.7	Reverse	45	N	SE
San Gabriel	USGS 2008	43.13	7.3	Strike Slip	61	NE	SE
Malibu Coast	USGS 2008	44.02	7	Strike Slip	74-75	N	E
Anacapa-Dume	USGS 2008	46.4	7.2	Reverse	41-45	N	E
Northridge	USGS 2008	46.54	6.9	Reverse	35	S	SE
Southern San Andreas	USGS 2008	54.35	8.2	Strike Slip	58-90	N,NE	SW
Santa Susana, alt 1	USGS 2008	56.05	6.9	Reverse	55	N	SE
San Jacinto	USGS 2008	56.74	7.875	Strike Slip	90	--	NW
Holser, alt 1	USGS 2008	65.13	6.8	Reverse	58	S	SE
Cleghorn	USGS 2008	65.37	6.8	Strike Slip	90	--	SW
Simi-Santa Rosa	USGS 2008	69.44	6.9	Strike Slip	60	N	SE
Oak Ridge Connected	USGS 2008	73.71	7.4	Reverse	53.1	S	SE

Note: (a) Not Applicable for Strike-Slip Fault



0 2500 5000

Approximate Scale in Feet

MAP UNITS

- All units may not apply -

Late Holocene (Surficial Deposits)

af — **Artificial Fill** - deposits of fill resulting from human construction, mining, or quarrying activities; includes engineered fill for buildings, roads, dams, airport runways, harbor facilities, and waste landfills

Qls — **Landslide Deposits** - may include debris flows and older landslides of various earth material and movement types; unconsolidated to moderately well-consolidated

Qb — **Beach Deposits** - unconsolidated marine beach sediments consisting mostly of fine- and medium-grained, well-sorted sand

Qw — **Alluvial Wash Deposits** - unconsolidated sandy and gravelly sediment deposited in recently active channels of streams and rivers; may contain loose to moderately loose sand and silty sand

Qa — **Alluvial Valley Deposits** - unconsolidated clay, silt, sand, and gravel recently deposited parallel to localized stream valleys and/or spread more regionally onto alluvial flats of larger river valleys; sandy sediment generally more dominant than gravelly sediment

Ql — **Lacustrine, Playa, and Estuarine (Paralic) Deposits** - mostly unconsolidated fine-grained sand, silt, mud, and clay from fresh water (lacustrine) lakes, saline (playa) dry lakes that are periodically flooded, and estuaries; deposits may contain salt and other evaporites

Qe — **Eolian and Dune Deposits** - unconsolidated, generally well-sorted wind-blown sand; may occur as dune forms or sheet sand

Holocene to Late Pleistocene (Surficial Deposits)

Qyf — **Young Alluvial Fan Deposits** - unconsolidated to slightly consolidated, undissected to slightly dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon

Qya — **Young Alluvial Valley Deposits** - unconsolidated to slightly consolidated, undissected to slightly dissected clay, silt, sand, and gravel along stream valleys and alluvial flats of larger rivers

Qyl — **Young Lacustrine, Playa, and Estuarine (Paralic) Deposits** - unconsolidated to slightly consolidated, undissected to slightly dissected fine-grained sand, silt, mud and clay from lake, playa, and estuarine deposits of various types

Qye — **Young Eolian and Dune Deposits** - unconsolidated to slightly consolidated, undissected to slightly dissected wind-blown sands

Late to Middle Pleistocene (Surficial Deposits)

Qof — **Old Alluvial Fan Deposits** - slightly to moderately consolidated, moderately dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon

Qoa — **Old Alluvial Valley Deposits** - slightly to moderately consolidated, moderately dissected clay, silt, sand, and gravel along stream valleys and alluvial flats of larger rivers

Qol — **Old Lacustrine, Playa, and Estuarine (Paralic) Deposits** - slightly to moderately consolidated, moderately dissected fine-grained sand, silt, mud, and clay from lake, playa, and estuarine deposits of various types

Qoe — **Old Eolian and Dune Deposits** - slightly to moderately consolidated, moderately dissected wind-blown sands

Middle to Early Pleistocene (Surficial Deposits)

Qvof — **Very Old Alluvial Fan Deposits** - moderately to well-consolidated, highly dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon

Quaternary (Bedrock)

Qss — **Coarse-grained formations of Pleistocene age and younger** - primarily sandstone and conglomerate

Qsh — **Fine-grained formations of Pleistocene age and younger** - includes fine-grained sandstone, siltstone, mudstone, shale, siliceous and calcareous sediments

Tertiary (Bedrock)

Tss — **Coarse-grained Tertiary age formations** - primarily sandstone and conglomerate

Tsh — **Fine-grained Tertiary age formations** - includes fine-grained sandstone, siltstone, mudstone, shale, siliceous and calcareous sediments

Tv — **Tertiary age formations of volcanic origin**

Mesozoic and Older (Bedrock)

pKm — **Cretaceous and pre-Cretaceous metamorphic formations of sedimentary and volcanic origin**

Basemap modified from "Geologic Compilation of Quaternary Surficial Deposits in Southern California Onshore Portion of the Long Beach 30' x 60' Quadrangle", July 2010, www.conservation.ca.gov/cgs

SYMBOL EXPLANATION

[For geologic line symbols: lines are solid where location is accurate, long-dashed where location is approximate, short-dashed where location is inferred, dotted where location is concealed. Queries added where identity or existence may be questionable.]

Contacts

Contact

Reference contact -- Used to delineate geologic units that were mapped as separate units on the original source map, but are consolidated on this map.

Fault -- Includes strike-slip, normal, reverse, oblique, and unspecified slip

Folds -- Showing direction of plunge where appropriate

Anticline

Syncline

Stream

Wet area

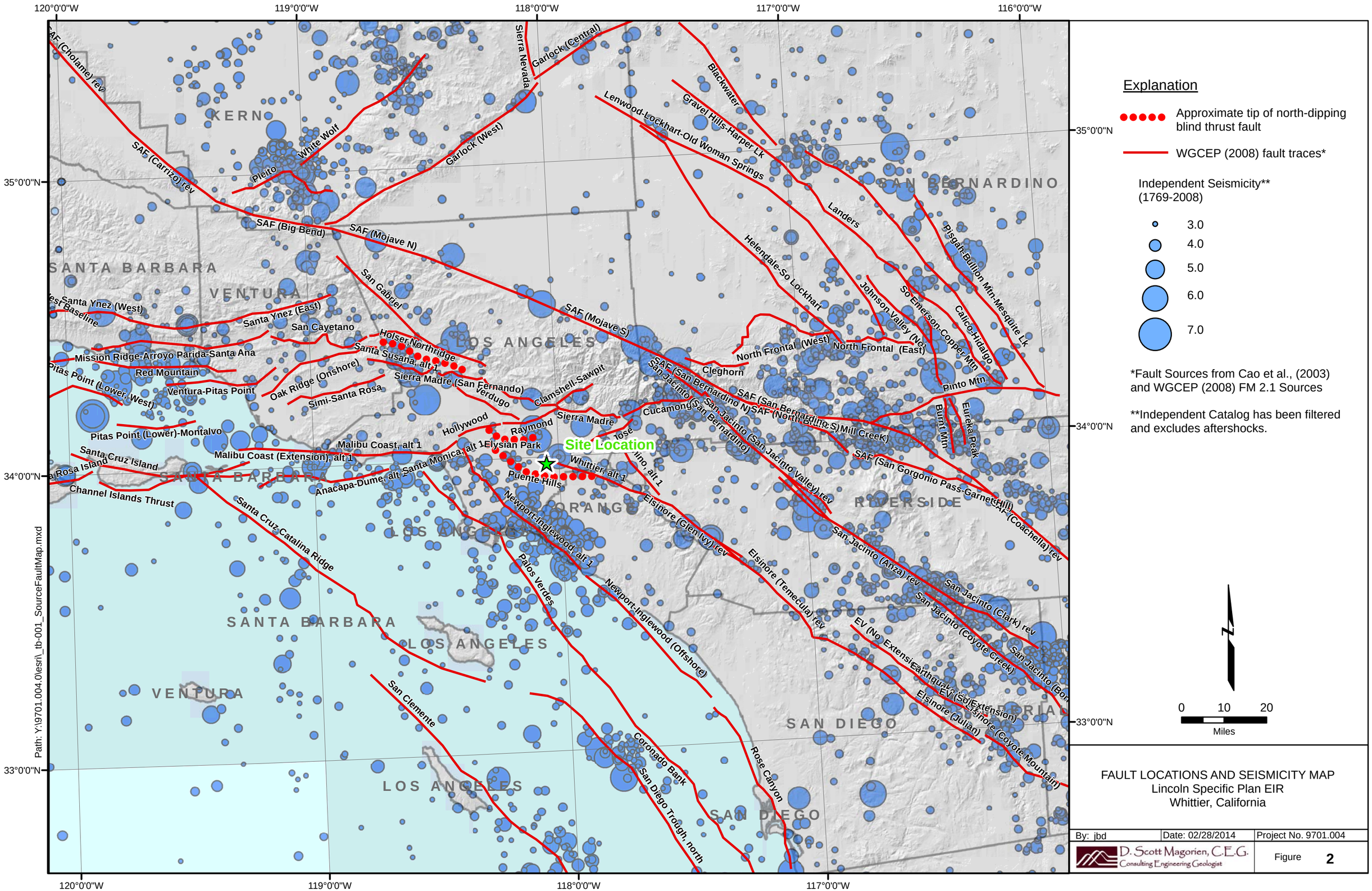
Road

County boundary

SITE LOCATION AND GEOLOGIC MAP
Lincoln Specific Plan EIR
Whittier, California

By: jbd	Date: 02/28/2014	Project No. 9701.004
D. Scott Magorien, C.E.G. Consulting Engineering Geologist		Drawing 1

Plot Date: 2/27/2014 8:57:08 AM , Plotted by: jim.deporter Drawing Path: Y:\9701.004.0\ACAD_TB-001_GEOLOGIC MAP.DWG , Figure 1_Site-Geology Map



Explanation

Approximate tip of north-dipping blind thrust fault

WGCEP (2008) fault traces*

Independent Seismicity**
(1769-2008)

- 3.0
- 4.0
- 5.0
- 6.0
- 7.0

*Fault Sources from Cao et al., (2003) and WGCEP (2008) FM 2.1 Sources

**Independent Catalog has been filtered and excludes aftershocks.

N

0 10 20
Miles

FAULT LOCATIONS AND SEISMICITY MAP
Lincoln Specific Plan EIR
Whittier, California

By: jbd	Date: 02/28/2014	Project No. 9701.004
D. Scott Magorien, C.E.G. Consulting Engineering Geologist	Figure	2

Preliminary Geotechnical Evaluation Report,
Proposed Development, Fred C. Nelles Site,
11850 East Whittier Boulevard,
City of Whittier, California

Dated: April 26, 2013

Project No. 12204-01

Prepared For:

Brookfield Homes
3090 Bristol Street, Suite 200
Costa Mesa, CA 92626

LGC Geotechnical, Inc.

April 26, 2013 Project No. 12204-01

Mr. Dave Bartlett
Brookfield Homes
3090 Bristol Street, Suite 200
Costa Mesa, CA 92626

Subject: ***Preliminary Geotechnical Evaluation of Proposed Development of Fred C. Nelles Site, 11850 East Whittier Boulevard, City of Whittier, California***

In accordance with your request and authorization, LGC Geotechnical, Inc. has performed a preliminary geotechnical evaluation for the proposed development of the approximately 75-acre Fred C. Nelles property located at 11850 East Whittier Boulevard in the City of Whittier, California. The findings, conclusions, and preliminary recommendations presented within this report are provided for future residential and commercial/retail development of the site. Additional subsurface exploration, laboratory testing, and geotechnical analyses may be required depending on proposed grading plan and/or improvements.

We appreciate the opportunity to be of service on this project. Should you have any questions regarding the content of this report, please do not hesitate to contact our office.

Respectfully,

LGC Geotechnical, Inc.

REGISTERED PROFESSIONAL ENGINEER
DENNIS BORATYNEC
No. 2770
Exp. 12/31/14
GEOTECHNICAL
STATE OF CALIFORNIA

Dennis Boratynec, GE 2770
Vice President

DJB/kmh

Distribution: (5) Addressee (wet-signed copies)

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Appendix C – Laboratory Test Results
Appendix D – Seismic Analyses Summary
Appendix E – General Earthwork & Grading Specifications

1.0 INTRODUCTION

1.1 Purpose and Scope of Services

This report presents the results of the geotechnical evaluation of the Fred C. Nelles property located at 11850 East Whittier Boulevard in the City of Whittier, California (see Site Location Map, Figure 1). The purpose of work was to perform a preliminary evaluation of the subsurface soil and geologic conditions at the site with respect to the proposed development, and to provide preliminary design recommendations for construction. This report is an update of the previous referenced report by Lawson and Associates Geotechnical Consulting (Lawson, 2005). The scope of services included the following:

- Review readily available background information including in-house regional geologic maps and published geotechnical literature, aerial photographs, and consultation with the site personnel regarding past site usage.
- Field investigation consisting of 14 hollow stem auger borings drilled to depths of approximately 26.5 to 51.5 feet, 9 CPT soundings advanced to depths of approximately 50 feet, and 12 backhoe test trenches excavated to depths of approximately 2 to 11 feet.
- Relatively undisturbed samples and bulk (disturbed) samples were obtained from the borings and test trenches for further classification and laboratory testing. The laboratory testing included in-place moisture and dry density, Atterberg Limits, consolidation, direct shear, maximum dry density and optimum moisture content, expansion index, R-value, and testing for corrosivity.
- Compilation and geotechnical analysis of the data obtained from the background review, field investigation, and laboratory testing, including analyses for the following:
  - Suitability of the site for the proposed development from a geotechnical standpoint.
  - Description of the site geology, and subsurface soil and groundwater conditions.
  - Evaluation of the seismic conditions at the site, including seismic design criteria based on the 2010 California Building Code (CBC).
  - Evaluation of the potential for earthquake induced liquefaction at the site.
  - Preliminary design parameters for foundations, slabs, exterior hardscape, and roadway pavement based upon the anticipated site conditions following grading.
- Preparation of this preliminary geotechnical report presenting the findings, conclusions and preliminary recommendations for the design and construction of the proposed residential project improvements.

Note: This investigation and report only addresses the geotechnical issues associated with future site development, and does not address any environmental issues.

1.2 Existing Site Conditions and Proposed Redevelopment

The site consists of an irregular shaped parcel encompassing approximately 75 acres of relatively level to gently sloping land. The study area is bordered to the northeast by Whittier Boulevard, to the northwest by Sorensen Avenue, to the east by commercial development, and to the south and west by residential development. Existing site improvements include numerous single story dormitory buildings, classroom buildings, maintenance buildings and other structures, which were part of the

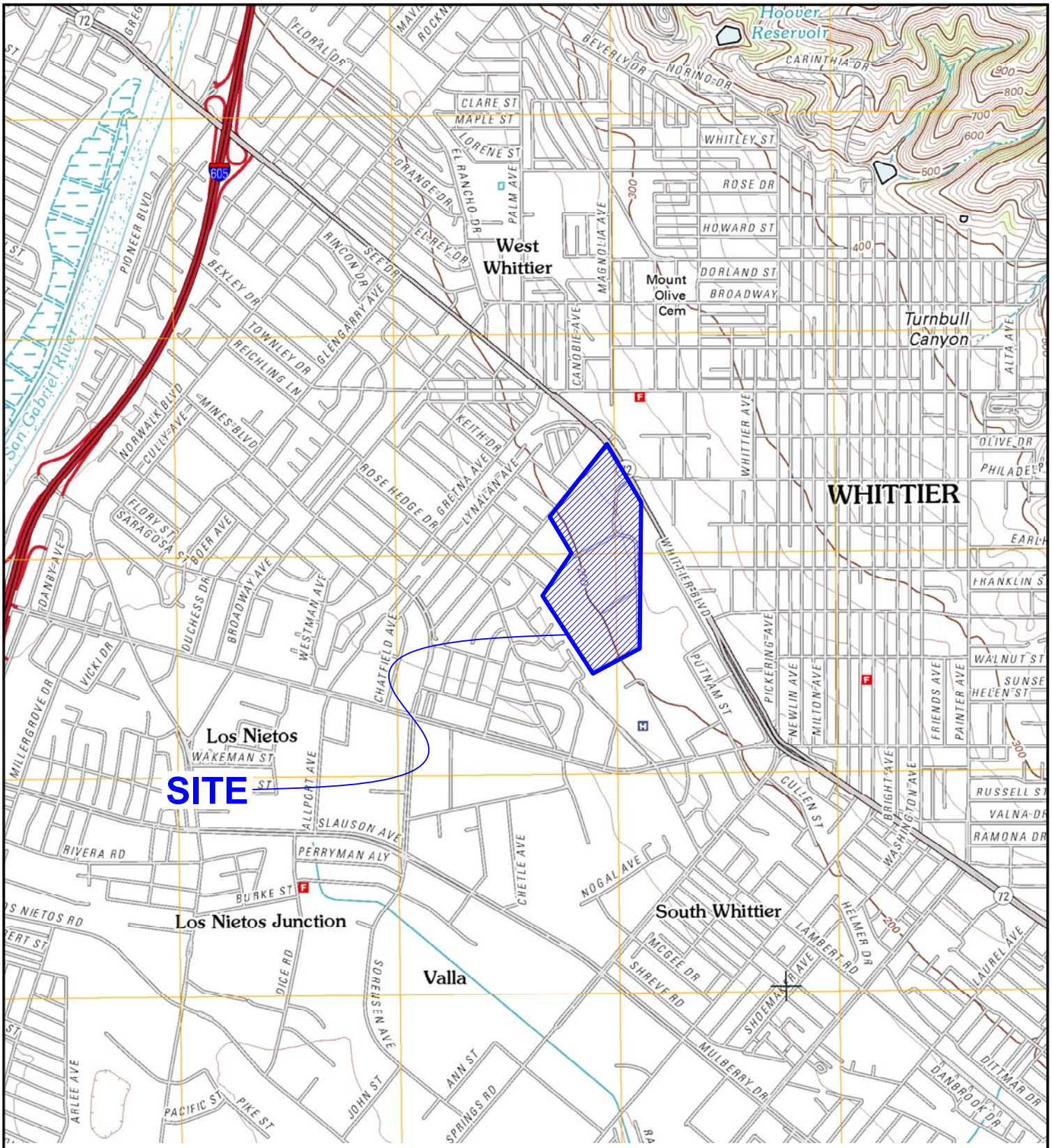
former Fred C. Nelles Youth Correctional facility. Extensive interior roadways, asphalt and concrete pavement areas, and buried site utility pipelines also exist throughout the property, in addition to numerous mature trees.

Although the overall topographic relief across the site is on the order of approximately 45 feet (from north to south), significant slopes are not present. The site generally slopes very gently toward the southwest. The relative highest elevation is located within the paved parking lot located at the north corner of the site adjacent to Whittier Boulevard, and the relatively lowest elevation is located within the grassy field along the southwest corner of the site.

We understand that future site development will consist of demolishing and/or relocating the existing buildings and improvements in preparation for new construction. Conventional grading operations will be performed to create new building pad areas and new interior roadways. Associated future residential and commercial/retail site improvements will also include paved parking, sidewalks, common area hardscape, and buried utility lines. The construction of significant slopes is not anticipated. Significant changes in grades are not anticipated. A grading plan depicting planned cuts and fills was not available at the time of this report.

Proposed structures are anticipated to be at-grade (i.e., no basements) and relatively light weight with maximum column and wall (dead plus live) loads of 40 kips and 4 kips per linear foot, respectively.

The recommendations given in this report are based upon the estimated structural and grading information above. We understand that the project plans are currently being developed at this time; LGC Geotechnical should be provided with updated project plans and any changes to structural loads and grading when they become available, in order to either confirm or modify the preliminary recommendations provided herein.



LGC Geotechnical, Inc.

FIGURE 1
Site Location Map

PROJECT NAME	Brookfield Whittier - Nelles
PROJECT NO.	12204-01
ENG. / GEOL.	DJB / TJL
SCALE	1" = 2,000'
DATE	April 2013

1.3 Subsurface Evaluation

The subsurface evaluation consisted of 12 test pits excavated with a backhoe up to depths of approximately 11 feet below existing grade, 9 CPT soundings to depths of approximately 50 feet below existing grade, and 14 hollow-stem auger borings to depths of approximately 26.5 to 51.5 feet. The field exploration was conducted on January 12, 13, and 18 of 2005. The approximate locations of the test pits, CPT soundings, and borings are depicted on Figure 2. The test pits were backfilled and compacted with the soils excavated. The CPT soundings and borings were backfilled with bentonite chips and site soils, respectively, and capped with asphalt patch in pavement areas. Some settlement of the backfill may occur over time.

Information obtained from the CPT soundings includes the ratios of Sleeve Friction (Fs) to Tip Resistance (Qt), which provides a general indication of the 'Soil Behavior Type' of the underlying soils (see Appendix B). Graphic CPT data logs are presented in Appendix B.

During the excavation of the test pits and borings, representative bulk and relatively undisturbed samples were obtained for laboratory testing. The test pits and borings were sampled using California Ring Samples (Ring) at selected intervals. Standard Penetration Tests (SPT) were also taken from the borings. The samples were driven using a 140-pound hammer with an approximately 30-inch drop for a total penetration of 18 inches, with blow-counts recorded for later correlation to relative density. A Certified Engineering Geologist logged the test pits and collected the samples. Geologic and soil description of the materials encountered and sampled are recorded on the test pit and borings logs, which are included in Appendix B.

1.4 Laboratory Testing

Laboratory testing was performed on representative soil samples obtained from the subsurface exploration and included in-situ moisture and density tests, Atterberg Limits (liquid limit and plastic limit), consolidation, direct shear, maximum density, expansion index, R-value, and corrosivity (sulfate, chloride, pH, and resistivity). A discussion of the tests performed and a summary of the results are presented in Appendix C.

The following is a summary of the laboratory test results.

- Seven Atterberg Limit (liquid limit and plastic limit) tests were performed. Results of the tests indicated Plasticity Index values ranging from 17 to 20.
- Seven gradation and fines content tests indicated a fines content (percent passing No. 200 sieve) ranging from approximately 63 to 79 percent. Based on the Unified Soils Classification System (USCS), the tested samples would be classified as "fine-grained."
- Seven consolidation tests were performed. The stress vs. deformation plots are provided in Appendix C.
- Two direct shear tests were performed on samples remolded to 90 percent relative compaction. The plots are provided in Appendix C.
- Seven Expansion Index (EI) tests were performed. Results were EI values ranging from 53 to 79, corresponding to "Medium" expansion potential.
- Two R-value tests indicated values of 9 and 10.

- Six corrosion suites were performed indicating soluble sulfate content values less than 0.02 percent, chloride content ranging from 100 parts per million (ppm) to 580 ppm, pH values ranging from 7.1 to 7.6 and minimum resistivity values ranging from 650 to 1,500 ohm-cm.

A summary of the laboratory test results are presented in Appendix C. The moisture and density test results are presented on the test pit and boring logs in Appendix B.

2.0 GEOTECHNICAL CONDITIONS

2.1 Site Geology and Subsurface Conditions

The majority of the subject site is underlain by Holocene-age sedimentary deposits, as mapped by the California Department of Mines and Geology (CDMG, 1998). Regional mapping by the United States Geological Survey (USGS), in cooperation with the California Geological Survey (CGS, formerly CDMG), describes the materials underlying the site as Holocene-age alluvial fan and valley deposits (flood plain sediments).

Earth materials encountered to the depths explored during the subsurface work at the site include fine-grained sediments consisting predominantly of layered clay, silt, sandy silt, and minor amounts of silty sand. Detailed descriptions of the subsurface conditions are presented on the trench and boring logs in Appendix B.

Regional geologic mapping and local topographic expressions do not indicate the presence of large-scale landslides within or adjacent to the project area.

2.2 Groundwater

Groundwater was not encountered during the field investigation to the maximum explored depth of approximately 50 feet below existing grade. Historic high groundwater levels reported by the CDMG (1998) range from approximately 30 to 50 feet below the ground surface, with the shallowest levels recorded along the southwest portion of the site. Future groundwater conditions below the site may be variable, depending on numerous factors including seasonal rainfall, local irrigation and groundwater pumping, among others.

2.3 Ground Rupture

Prompted by the 1971 San Fernando Earthquake in Southern California, State legislation and policies concerning the classification and land-use criteria associated with faults have been developed to prevent the construction of urban developments across the trace of active faults. The result is the Alquist-Priolo Earthquake Fault Zoning Act of 1972. The provisions of the Act, an index to maps of Earthquake Fault-Rupture Zones (formerly Alquist-Priolo Special Studies Zones), and current revisions are presented in CDMG Special Publication 42 by Hart and Bryant (1997), with supplements 1 and 2 added in 1999 and supplement 3 added in 2003. According to the State Geologist, an active fault is defined as one, which has had surface displacement within the Holocene Epoch (roughly the last 11,000 years). A potentially active fault is defined as any fault, which has had surface displacement during Quaternary time (last 1,600,000 years), but not within the Holocene. Earthquake Fault-Rupture Zones have been delineated along the traces of active faults within California. Where developments for human occupation are proposed within these zones, the state requires detailed fault investigations be performed so that engineering geologists can mitigate the hazards associated with active faulting by identifying the location of active faults and allowing for a setback from the zone of previous ground rupture.

The site is not located within a mapped Earthquake Fault-Rupture Zone per compiled maps released by the CDMG (2000a), and no known active or potentially active faults cross the site. The possibility of

damage due to ground rupture, as a result of faulting, is considered remote since active faults are not known to cross the site. The nearest known active fault is the Whittier fault located approximately 2 kilometers north/northeast of the proposed development (Jennings, 1994).

2.4 Seismicity

The site is located in a seismically active area, as is the majority of California, and the potential for strong ground motion within the project area is considered significant during the design life of the proposed improvements. The site seismic characteristics were evaluated per the guidelines set forth in Chapter 16, Section 1613 of the 2010 California Building Code (CBC). Representative site coordinates of latitude 33.9792 degrees north and longitude -118.0508 degrees west, were utilized in the analyses. The maximum considered earthquake (MCE) spectral response accelerations (S_{MS} and S_{M1}) and adjusted design spectral response acceleration parameters (S_{DS} and S_{D1}) for Site Class D are provided in Table 1.

TABLE 1

Seismic Design Parameters

Selected Parameters from the 2010 CBC, Section 1613 - Earthquake Loads	**Seismic Design Values**
Site Class per Table 1613.5.2	D
Spectral Acceleration for Short Periods (S_S)*	1.983g
Spectral Accelerations for 1-Second Periods (S_1)*	0.703g
Site Coefficient F_a per Table 1613.5.3(1)	1.0
Site Coefficient F_v per Table 1613.5.3(2)	1.5
Site Modified Spectral Acceleration for Short Periods (S_{MS}) for Site Class D [Note: $S_{MS} = F_aS_S$]	1.983g
Site Modified Spectral Acceleration for 1-Second Periods (S_{M1}) for Site Class D [Note: $S_{M1} = F_vS_1$]	1.054g
Design Spectral Acceleration for Short Periods (S_{DS}) for Site Class D [Note: $S_{DS} = (^2/_3)S_{MS}$]	1.322g
Design Spectral Acceleration for 1-Second Periods (S_{D1}) for Site Class D [Note: $S_{D1} = (^2/_3)S_{M1}$]	0.703g

* From USGS, 2011

Section 1802.2.7 of the 2010 CBC states that the design peak ground acceleration (PGA) for a site may be defined as S_{DS}/2.5. Therefore, the PGA = 1.322g/2.5 = **0.53g.**

A deaggregation of the PGA based on a 2,475-year average return period indicates that an earthquake

magnitude of 7.0 at a distance of approximately 3 km from the site would contribute the most to this ground motion (USGS, 2008).

Secondary effects of seismic shaking resulting from large earthquakes on the major faults in the Southern California region, include soil liquefaction, dynamic settlement, ground lurching, shallow ground rupture, and seiches and tsunamis. These secondary effects of seismic shaking are a possibility throughout portions of the Southern California region and are dependant on the distance between the site and causative fault and the onsite geology. A discussion of these secondary effects and their potential impact on the site is provided in the following sections.

2.4.1 Lurching

Soil lurching refers to the movement of soil toward an open or unsupported face, such as the side of a steep slope or embankment. The rolling motion on the ground surface causes lurching by the passage of seismic surface waves. Effects of this nature are not likely to be significant within flat lying areas, such as the subject site.

2.4.2 Liquefaction and Dynamic Settlement

Liquefaction is a seismic phenomenon in which loose, saturated, granular soils behave similarly to a fluid when subject to high-intensity ground shaking. Liquefaction occurs when three general conditions coexist: 1) shallow groundwater; 2) low density non-cohesive (granular) soils; and 3) high-intensity ground motion. The potential effects of liquefaction on level ground include settlement, sand boils, and bearing capacity failures below structures. Dynamic settlement of dry sands can also occur as the sand particles tend to settle and densify as a result of a seismic event.

The southern and southwestern portions of the site are located within a mapped Seismic Hazard Zone for soils considered potentially susceptible to earthquake-induced liquefaction (CDMG, 1999). Preliminary liquefaction analyses performed utilizing the limited data indicate isolated sand layers are subject to liquefaction during a significant earthquake. Preliminary analyses were performed utilizing the historic high groundwater level of 30 feet below the existing ground surface. Seismically induced settlements were estimated by the procedure outlined by Ishihara and Yoshimine (1992). The results of the analyses indicate total seismic settlements on the order of approximately $^{1}/_{2}$-inch for the upper 50 feet. Differential seismic settlement can be estimated as half of the total estimated settlement over a horizontal span of about 30 feet (see Appendix D).

2.4.3 Lateral Spreading

Lateral spreading is a type of liquefaction-induced ground failure associated with the lateral displacement of surficial blocks of sediment resulting from liquefaction in a subsurface layer. Once liquefaction transforms the subsurface layer into a fluid mass, gravity plus the earthquake inertial forces may cause the mass to move downslope towards a free face (such as a river

channel or an embankment). Lateral spreading may cause large horizontal displacements and such movement typically damages pipelines, utilities, bridges, and structures.

Due to the lack of an adjacent free face to drive lateral spreading, the potential for lateral spreading is considered very low.

2.4.4 Tsunamis and Seiches

Based on the distance of the study area from large bodies of open water, the possibility of seiches and/or tsunamis affecting the site is considered to be remote.

3.0 CONCLUSIONS

Based on geotechnical data gathered and the results of the analyses, it is our opinion that development of the subject site for future residential and commercial/retail construction is considered feasible from a geotechnical standpoint. The major geotechnical items to be considered in the design, and ultimately in the construction of the proposed project, are summarized below:

- Following demolition and removal of the existing site improvements (buildings, asphalt pavement, shallow utilities), the condition of the near-surface soils will need to be improved for the support of future site development. Such improvement should consist of the removal of existing fill materials and the upper native soils and replacement with engineered compacted fill. Preliminary earthwork recommendations are presented in the following Recommendation sections of this report.
- Based on preliminary laboratory testing, site soils have Medium expansion potential. Mitigation measures are required for foundations and site improvements like concrete flatwork to minimize the impacts of expansive soils.
- The site is not located within a mapped Earthquake Fault-Rupture Zone and, based upon the review of published geologic mapping and aerial photographs, no active or potentially active faults are known to exist within or in the immediate vicinity of the site. Therefore, the potential for ground rupture as a result of faulting is considered remote.
- Seismic hazards associated with a significant earthquake generated from one of the active regional faults include ground shaking. The estimated peak horizontal ground acceleration per the 2010 California Building Code is 0.53g. New improvements will need to be designed for seismic forces in accordance with current building codes and regulations. However, there is still a risk that the proposed structures and associated improvements could be damaged as a result of an earthquake. Repair of the planned residential and commercial/retail structures may be needed after a seismic event.
- The site is located within a mapped Seismic Hazard Zone for liquefaction. Based upon preliminary analyses, isolated underlying sand layers are susceptible to liquefaction during a significant earthquake. The results of the analyses indicate total seismic settlements on the order of approximately $^{1}/_{2}$-inch for the upper 50 feet. Differential dynamic settlement can be estimated at half of the total settlement over a horizontal span of 30 feet.
- Site generally contains clayey soils with high fines content and expansion potential that are not suitable for backfill of any site retaining walls. Therefore, import of sandy soils meeting project recommendations will be required.
- Based on laboratory testing for chloride content, onsite soils are considered corrosive. Soils are considered corrosive if the pH is 5.5 or less, or the chloride concentration is 500 ppm or greater, or the sulfate concentration is 2,000 ppm or greater based on Caltrans Corrosion Guidelines (2003).

4.0 RECOMMENDATIONS

The following recommendations are to be considered preliminary, and should be confirmed upon completion of grading and earthwork operations. In addition, they should be considered minimal from a geotechnical viewpoint, as there may be more restrictive requirements from the architect, structural engineer, building codes, governing agencies, or the owner. A grading plan review should also be performed by LGC prior to the start of earthwork activities. Additional and/or revised recommendations may be provided at the conclusion of plan review, including recommendations for additional subsurface investigation and laboratory testing.

4.1 Site Earthwork

We anticipate that earthwork at the site will generally consist of removal and re-compaction of the near-surface soils for foundation and roadway support. Shallow trenching for construction of slab-on-grade type foundations and utilities is also anticipated. Site earthwork should be performed in accordance with the following recommendations, in addition to those contained in the CBC, and the General Earthwork and Grading Specifications for Rough Grading included in Appendix E. In case of conflict, the following recommendations shall supersede those included as a part of Appendix E.

4.1.1 Site Preparation

Prior to grading, the proposed construction area should be stripped of vegetation, cleared of surface obstructions, pavements, and foundation and slab elements from the existing site improvements. Vegetation, debris, and excessive organic material should be removed and properly disposed of offsite. Holes resulting from the removal of buried obstructions should be replaced with suitable compacted fill material. Soft or yielding subgrade materials encountered within bottom excavations should also be removed to a depth that exposes firm soil.

4.1.2 Removal Depths and Limits

The upper soils are generally soft, loose and compressible and not suitable for uniform support of structures or pavements in their current state. We recommend that these materials be removed, moisture-conditioned, and replaced as properly placed compacted fill. In general, the depth of removals will depend on existing and proposed grades as well as the depth of footings. The provided estimated removal depths are preliminary, and should be updated based on proposed precise grading and foundation plans.

Building Pad Areas: Removals should extend a minimum of 5 feet below existing grade or 2 feet below planned footings, whichever is greater. Removals should extend laterally a minimum distance of 5 feet beyond the edges of the proposed building pad improvements.

Free-Standing Walls, Pavement and Hardscape Areas: For minor site structures such as free-standing and screen walls, the removals should extend at least 3 feet beneath the existing grade or 2 feet beneath the base of footings, whichever is deeper. Within pavement and hardscape

areas, removals should extend to a depth of at least 2 feet below the existing grade. The over-excavation in any design cut areas of the pavement may be reduced by the depth of the design cut, but should not be less than 1-foot below the finished subgrade (i.e., below the bottom of the aggregate base). In general, the envelope for over-excavation should extend laterally a minimum distance of 2 feet beyond the edges of the proposed improvements.

Local conditions (such as soft materials) may be encountered during excavation that could require additional over-excavation beyond the above noted minimum in order to obtain an acceptable subgrade. The actual depths and lateral extents of grading will be determined by the geotechnical consultant, based on subsurface conditions encountered during grading. Areas to be over-excavated should be accurately staked in the field by the Project Surveyor.

4.1.3 Temporary Excavations

Excavations 5 feet or greater in depth should either be laid back or shored in accordance with Cal/OSHA requirements before personnel are allowed to enter. Soil conditions should be frequently observed by a representative of LGC Geotechnical, not only to confirm the geotechnical conditions but to also help provide early warning of potential failures. The contractor will be responsible for providing the "competent person" required by Cal/OSHA standards to evaluate soil conditions. Close coordination with the geotechnical consultant should be maintained to facilitate construction while providing safe excavations. Excavation safety is the sole responsibility of the contractor.

Once an excavation has been initiated, it should be backfilled as soon as practical. Prolonged exposure of temporary excavations may result in some localized instability. Excavations should be planned so that they are not initiated without sufficient time to shore/fill them prior to weekends, holidays, or forecasted rain.

It should be noted that any excavation that extends below a 1:1 (horizontal to vertical) projection of an existing foundation will remove existing support of the structure foundation. If requested, temporary shoring parameters will be provided.

4.1.4 Removal Bottoms and Subgrade Preparation

Removal bottoms and areas to receive fill should be observed and accepted by the geotechnical consultant prior to subsequent fill placement. In general, over-excavated removal bottom areas and areas to receive compacted fill should be scarified to a minimum depth of 6 inches, brought to a near-optimum moisture condition, and re-compacted per project requirements.

4.1.5 Material for Fill

From a geotechnical perspective, the onsite soils are generally considered suitable for use as general compacted fill, provided they are screened of organic materials, construction debris and any oversized material (larger than 8 inches in greatest dimension).

Retaining wall backfill should consist of sandy soils with a maximum of 35 percent fines (passing the No. 200 sieve) per American Society for Testing and Materials (ASTM) Test Method D1140 (or ASTM D6913/D422) and a Very Low expansion potential (EI of 20 or less per ASTM D4829). Soils should also be screened of organic materials, construction debris and any material greater than 3-inches. The site contains soils that are not suitable for retaining wall backfill due to their clay content and expansion potential, therefore import will likely be required by contractor for obtaining suitable retaining wall backfill soil. Contractor should anticipate required import of soils for any retaining wall backfill.

From a geotechnical viewpoint, any required import soils should consist of clean, relatively granular soils of Very Low to Low expansion potential (expansion index 50 or less based on ASTM D4829) and no particles larger than 6 inches in greatest dimension. Source samples of planned importation should be provided to the geotechnical consultant a minimum of 3 working days prior to any planned importation for required laboratory testing.

Aggregate base (crushed aggregate base or crushed miscellaneous base) should conform to the requirements of Section 200-2 of the Standard Specifications for Public Works Construction ("Greenbook") for untreated base materials (except processed miscellaneous base) or Caltrans Class 2 aggregate base.

4.1.6 Fill Placement and Compaction

Material to be placed as fill should be brought to near optimum moisture content (generally within optimum and 2 percent above optimum moisture content) and re-compacted to at least 90 percent relative compaction (per ASTM D1557). Moisture conditioning of site soils will be required in order to achieve adequate compaction. 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 compacted thickness. Each lift should be thoroughly compacted and accepted prior to subsequent lifts. Generally, placement and compaction of fill should be performed in accordance with local grading ordinances and with observation and testing by the geotechnical consultant. Oversized material as previously defined should be removed from site fills.

Fill placed on any slopes greater than 5:1 (horizontal to vertical) should be properly keyed and benched into firm and competent soils as it is placed in lifts.

Aggregate base material should be compacted to a minimum of 95 percent relative compaction at or slightly above optimum moisture content per ASTM D1557. Subgrade below aggregate base should be compacted to a minimum of 90 percent relative compaction per ASTM D1557 at or slightly above optimum moisture content.

4.1.7 Trench and Retaining Wall Backfill and Compaction

The onsite soils may generally be suitable as trench backfill, provided the soils are screened of rocks, organic matter, and other material greater than 6 inches in diameter. If trenches are shallow, or the use of conventional equipment may result in damage to the utilities, sand having

a sand equivalent (SE) of 30 or greater may be used to bed and shade the pipes. Sand backfill within the pipe bedding zone may be densified by jetting or flooding and then tamping to ensure adequate compaction. Subsequent trench backfill should be compacted in uniform thin lifts by mechanical means to at least 90 percent relative compaction (per ASTM D1557). On-site soils are not considered suitable for densification by means of jetting.

Retaining wall backfill should consist of predominantly granular, sandy soils as outlined in above Section 4.1.5. The limits of select sandy backfill should extend at minimum ½ the height of the retaining wall or the width of the heel (if applicable), whichever is greater. Refer to Figure 3. Retaining wall backfill soils should be compacted in relatively uniform thin lifts to a minimum of 90 percent relative compaction (per ASTM D1557). Jetting or flooding of retaining wall backfill materials should not be permitted.

A representative from LGC Geotechnical should observe, probe, and test the backfill to verify compliance with the project recommendations.

4.1.8 Soil Shrinkage

Volumetric changes in earth quantities will likely occur when excavated onsite earth materials are processed as compacted fill. The estimate for shrinkage of the upper soils used for compacted fill is based upon a comparison of in-situ dry density values versus the estimated average degree of relative compaction achieved during grading. Based on the results of our limited laboratory testing, we estimate a 5 to 10 percent volume reduction of the existing upper 5 feet of earth materials (undocumented fill, topsoil, and alluvium). We stress that this estimate is based upon a comparison of limited data obtained from widely spaced excavations, and that actual shrinkage factors will vary across the site. Furthermore, the shrinkage of onsite soils processed as compacted fill will depend primarily on the type of compaction equipment and method of compaction used by the grading contractor. The shrinkage estimate is intended as an aid for the project civil engineer in determining preliminary earthwork quantities. However, this estimate should be used with caution since it is not an absolute value. Contingencies should be made for balancing earthwork quantities based on actual shrinkage and subsidence that occurs during grading.

4.2 Preliminary Foundation Design Parameters

Based on the anticipated site geotechnical conditions, and provided that the remedial grading recommendations provided herein are implemented, the site may be considered suitable for the support of the residential and commercial/retail structures using a shallow foundation system. Preliminary conventional and post-tensioned foundation recommendations are provided in the following sections. Recommended soil bearing and estimated static settlement are provided in Section 4.3. Please note that the following foundation recommendations are preliminary and must be confirmed by LGC Geotechnical at the completion of project plans (i.e., foundation, grading and site layout plans) as well as the completion of earthwork.

4.2.1 Provisional Conventional Foundation Design Parameters

Conventional foundations may be designed in accordance with Wire Reinforcement Institute (WRI) procedure for slab-on-ground foundations per Section 1808 of the 2010 CBC to resist expansive soils. The following preliminary soil parameters may be used:

- Effective Plasticity Index: 30
- Climatic Rating: Cw = 15
- Reinforcement: Per structural designer
- Minimum Perimeter Embedment: 18 inches below lowest adjacent grade
- Moisture condition (presoak) slab subgrade to 120% of optimum moisture content to a minimum depth of 18 inches. The recommended moisture content should be maintained up to the time of concrete placement.

Other types of stiff slabs may be used in place of the WRI design procedure provided that, in the opinion of the foundation structural designer, the alternative type of slab is at least as stiff and strong as that designed by the WRI to resist expansive soils.

4.2.2 Provisional Post-Tensioned Foundation Design Parameters

The minimum geotechnical parameters provided in Table 2 may be used for post-tensioned slab foundations. These parameters have been determined in general accordance with and the Post-Tensioning Institute (PTI) Standard Requirements for Design of Shallow Post-Tensioned Concrete Foundations on Expansive Soils referenced in Chapter 18 of the 2010 CBC. In utilizing these parameters, the foundation engineer should design the foundation system in accordance with the allowable deflection criteria of applicable codes and the requirements of the structural designer/architect. Other types of stiff slabs may be used in place of the CBC post-tensioned slab design provided that, in the opinion of the foundation structural designer, the alternative type of slab is at least as stiff and strong as that designed by the CBC/PTI method. Increasing the stiffness of the foundation system in excess of the minimum parameters provided herein will decrease the potential for post-construction movement.

Our design parameters are based on our experience with similar projects and the anticipated nature of the soil (with respect to expansion potential). Please note that implementation of our recommendations will not eliminate foundation movement (and related distress) should the moisture content of the subgrade soils fluctuate. It is the intent of these recommendations to help maintain the integrity of the proposed structures and reduce (not eliminate) movement, based upon the anticipated site soil conditions. Should future owners not properly maintain the areas surrounding the foundation, for example by overwatering, then we anticipate for highly expansive soils the maximum differential movement of the perimeter of the foundation to the center of the foundation to be on the order of a couple of inches. Soils of lower expansion potential are anticipated to show less movement.

4.2.3 Post-Tensioned Foundation Subgrade Preparation and Maintenance

Moisture conditioning of the subgrade soils is recommended prior to trenching the foundation. The recommendations, specific to anticipated site soil conditions, are presented in Table 2. The subgrade moisture condition of the building pad soils should be maintained at the recommended moisture content up to the time of concrete placement. This moisture content should be maintained around the immediate perimeter of the slab during construction and up to occupancy of the building structures.

The geotechnical parameters provided herein assume that if the areas adjacent to the foundation are planted and irrigated, these areas will be designed with proper drainage and adequately maintained so that ponding, which causes significant moisture changes below the foundation, does not occur. Our recommendations do not account for excessive irrigation and/or incorrect landscape design. Plants should only be provided with sufficient irrigation for life and not overwatered to saturate subgrade soils. Sunken planters placed adjacent to the foundation, should either be designed with an efficient drainage system or liners to prevent moisture infiltration below the foundation. Some lifting of the perimeter foundation should be expected even with properly constructed planters.

In addition to the factors mentioned above, future owners/property management personnel should be made aware of the potential negative influences of trees and/or other large vegetation. Roots that extend near the vicinity of foundations can cause distress to foundations. Future owners (and the owner's landscape architect) should not plant trees/large shrubs closer to the foundations than a distance equal to half the mature height of the tree or 20 feet, whichever is more conservative unless specifically provided with root barriers to prevent root growth below the building foundation.

It is the owner's responsibility to perform periodic maintenance during hot and dry periods to insure that adequate watering has been provided to keep soil from separating or pulling back from the foundation. Future owners and property management personnel should be informed and educated regarding the importance of maintaining a constant level of soil-moisture. The owners should be made aware of the potential negative consequences of both excessive watering, as well as allowing potentially expansive soils to become too dry. Expansive soils can undergo shrinkage during drying, and swelling during the rainy winter season, or when irrigation is resumed. This can result in distress to building structures and hardscape improvements. The builder should provide these recommendations to future owners and property management personnel.

TABLE 2

Provisional Geotechnical Parameters for Post-Tensioned Foundation Slab Design

Parameter	**PT Slab with Perimeter Footing**	**PT Mat with Thickened Edge**
Expansion Index	Medium[1]	Medium[1]
Thornthwaite Moisture Index	-20	-20
Constant Soil Suction	PF 3.6	PF 3.6
Center Lift Edge moisture variation distance, e_m Center lift, y_m	9.0 feet 0.5 inches	9.0 feet 0.6 inches
Edge Lift Edge moisture variation distance, e_m Edge lift, y_m	4.7 feet 1.1 inch	4.7 feet 1.3 inch
Modulus of Subgrade Reaction, k (assuming presoaking as indicated below)	125 pci	125 pci
Minimum perimeter footing/thickened edge embedment below finish grade	18 inches	6 inches

1. Assumed for preliminary design purposes. Further evaluation is needed at the completion of grading.
2. Moisture condition to 120% of optimum moisture content to a depth of 18 inches prior to trenching.

4.2.4 Slab Underlayment Guidelines

The following is for informational purposes only since slab underlayment (e.g., moisture retarder, sand or gravel layers for concrete curing and/or capillary break) is unrelated to the geotechnical performance of the foundation. Post-construction moisture migration should be expected below the foundation. The foundation engineer/architect should determine whether the use of a capillary break (sand or gravel layer), in conjunction with the vapor retarder, is necessary or required by code. Sand layer thickness and location (above and/or below vapor retarder) should also be determined by the foundation engineer/architect.

4.3 Soil Bearing and Lateral Resistance

An allowable soil bearing pressure of 1,500 psf may be used for the design of footings having a minimum width of 12 inches and minimum embedment of 18 inches below lowest adjacent ground surface. This value may be increased by 300 psf for each additional foot of embedment or 100 psf for each additional foot of foundation width to a maximum value of 2,500 psf. These allowable bearing

pressures are applicable for level (ground slope equal to or flatter than 5H:1V) conditions only. Bearing values indicated above are for total dead loads and live loads. The above vertical bearing may be increased by one-third for short durations of loading which will include the effect of wind or seismic forces.

In utilizing the above allowable bearing capacity and provided our earthwork recommendations are implemented, foundation settlement due to structural loads is anticipated to be 1-inch or less. Differential settlement may be taken as half of the total settlement (i.e., ½-inch over a horizontal span of 40 feet). Estimated dynamic settlement is provided in Section 2.4.2.

Resistance to lateral loads can be provided by friction acting at the base of foundations and by passive earth pressure. For concrete/soil frictional resistance, an allowable coefficient of friction of 0.25 may be assumed with dead-load forces. An ultimate passive lateral earth pressure of 330 psf per foot of depth (or pcf) to a maximum of 3,300 psf may be used for the sides of footings poured against properly compacted fill. This passive pressure is applicable for level (ground slope equal to or flatter than 5H:1V) conditions only. Frictional resistance and passive pressure may be used in combination without reduction. We recommend that the upper foot of passive resistance be neglected if finished grade will not be covered with concrete or asphalt. The provided passive pressure is an ultimate value, so appropriate safety factor (i.e., minimum of 1.5) should be applied by the structural designer.

4.4 Lateral Earth Pressures for Retaining Walls

Lateral earth pressures for approved native sandy soils meeting indicated project requirements are provided below. Lateral earth pressures are provided as equivalent fluid unit weights, in psf per foot of depth (or pcf). These values do not contain an appreciable factor of safety, so the retaining wall designer should apply the applicable factors of safety and/or load factors during design. A soil unit weight of 120 pcf may be assumed for calculating the actual weight of soil over the wall footing.

The following lateral pressures are presented on Table 3 for approved select granular soils a maximum of 35 percent fines (passing the No. 200 sieve per ASTM D1140) and an Expansion Index of 20 or less per ASTM D4829. The retaining wall designer should clearly indicate on the retaining wall plans the required select sandy soil backfill.

TABLE 3

Lateral Earth Pressures – Approved Select Material

Conditions	**Equivalent Fluid Unit Weight (pcf)**
	Level Backfill
	Approved Soils
Active	35
At-Rest	55

The lateral earth pressures provided above may be increased by a factor of 1.5 for a 2:1 (horizontal to

vertical) sloping backfill condition.

If the wall can yield enough to mobilize the full shear strength of the soil, it 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. The equivalent fluid pressure values assume free-draining conditions. If conditions other than those assumed above are anticipated, the equivalent fluid pressure values should be provided on an individual case basis by the geotechnical engineer. Retaining wall structures should be provided with appropriate drainage and appropriately waterproofed. The outlet pipe should be sloped to drain to a suitable outlet. Refer to Figure 3.

Surcharge loading effects from adjacent structures should be evaluated by the retaining wall designer. In general, structural loads within a 1:1 (horizontal to vertical) upward projection from the bottom of the proposed retaining walls will surcharge the proposed retaining structure. For estimation of earth pressures due to surface loads from adjacent foundations, uniform surcharges may be estimated using the applicable coefficient of earth pressure using a rectangular distribution. A factor of 0.5 and 0.33 may be used for at-rest and active conditions, respectively. If applicable, typical vehicle traffic may be estimated as equivalent to 2 feet of compacted fill, a vertical pressure of 240 psf. The retaining wall designer should contact the geotechnical engineer for any required geotechnical input in estimating any applicable surcharge loads.

If required, the retaining wall designer may use a seismic lateral earth pressure increment of 16 pcf. This increment should be applied in addition to the provided static lateral earth pressure using a triangular distribution with the resultant acting at H/3 in relation to the base of the retaining structure (where H is the retained height). Per Section 1803.5.12 of the 2010 CBC, the seismic earth pressure is applicable to "structures assigned to Seismic Design Category D, E, or F in accordance with Section 1613." This seismic lateral earth pressure is estimated using the procedure outlined by the Structural Engineers Association of California (Lew, et al, 2010). The provided seismic lateral earth pressure is for a level backfill condition; a sloping backfill condition is not anticipated. However, if a sloping backfill condition is proposed, the retaining wall designer should contact the geotechnical engineer for specific seismic lateral earth pressure increments based on the configuration of the planned retaining walls.

Soil bearing and lateral resistance (friction coefficient and passive resistance) are provided in Section 4.3. Earthwork considerations (temporary backcuts, backfill, compaction, etc.) for retaining walls are provided in Section 4.1 (Site Earthwork) and the subsequent earthwork related sub-sections.

4.5 ***Soil Corrosivity***

Although not corrosion engineers (LGC Geotechnical is not a corrosion consultant), several governing agencies in Southern California require the geotechnical consultant to determine the corrosion potential of soils to buried concrete and metal facilities. We therefore present the results of corrosion testing for the use of the client and other consultants, as they determine necessary.

Laboratory test results indicated soluble sulfate content values less than 0.02 percent, chloride content ranging from 100 parts per million (ppm) to 580 ppm, pH values ranging from 7.1 to 7.6 and

minimum resistivity values ranging from 650 to 1,500 ohm-cm. Based on Caltrans Corrosion Guidelines (2003), soils are considered corrosive if the pH is 5.5 or less, or the chloride concentration is 500 ppm or greater, or the sulfate concentration is 2,000 ppm or greater. Based on test results for chloride content, onsite soils are considered corrosive based on Caltrans criteria.

Based on laboratory sulfate test results, the near surface soils have a severity categorization of "Not Applicable" and are designated to a class "S0" per ACI 318, Table 4.2.1 with respect to sulfates. Concrete in direct contact with the onsite soils can be designed according to ACI 318, section 4.3 using the "S0" sulfate classification. This must be verified based on as-graded conditions.

4.6 Non-Structural Concrete Flatwork

Concrete flatwork (such as walkways, patio slabs, etc.) has a potential for cracking due to changes in soil volume related to soil-moisture fluctuations. To reduce the potential for excessive cracking and lifting, concrete should be designed in accordance with the guidelines outlined in Table 4. These guidelines will help reduce the potential for irregular cracking and promote cracking along construction joints, but will not eliminate all cracking or lifting. Thickening the concrete and/or adding additional reinforcement will further reduce cosmetic distress.

TABLE 4

Preliminary Geotechnical Parameters for Non-structural Concrete Flatwork

	Homeowner Sidewalks	**Private Drives**	**Patios/Entryways**	**City Sidewalk Curb and Gutters**
Minimum Thickness (in.)	4 (nominal)	4 (full)	4 (full)	City/Agency Standard
Moisture Condition	Pre-moisten prior to placing concrete	Pre-moisten prior to placing concrete	Pre-moisten prior to placing concrete	City/Agency Standard
Reinforcement	—	No. 3 at 24 inches on centers	No. 3 at 24 inches on centers	City/Agency Standard
Thickened Edge (in.)	—	8 x 8 (total thickness)	—	City/Agency Standard
Crack Control Joints	Saw cut or deep open tool joint to a minimum of 1/3 the concrete thickness	Saw cut or deep open tool joint to a minimum of 1/3 the concrete thickness	Saw cut or deep open tool joint to a minimum of 1/3 the concrete thickness	City/Agency Standard
Maximum Joint Spacing	5 feet	10 feet or quarter cut whichever is closer	6 feet	City/Agency Standard
Aggregate Base Thickness (in.)	—	—	—	City/Agency Standard

To reduce the potential for sidewalks to separate from the building slab, the owner may elect to install dowels to tie these two elements together.

4.7 Preliminary Pavement Sections

Based on an R-value of 9, we recommend the following provisional minimum street sections for Traffic Indices of 4.5, 5, and 6. These recommendations should be confirmed with R-value testing of representative near-surface soils at the completion of grading and after underground utilities have been installed and backfilled. The project civil engineer should confirm design Traffic Index in accordance with the City of Whittier requirements. In addition, additional sections can be provided based on other desired traffic indices.

Assumed Traffic Index	4.5	5	6
R -Value Subgrade	9	9	9
AC Thickness	4.0 inches	4.0 inches	5.0 inches
Aggregate Base Thickness	5.0 inches	8.0 inches	9.0 inches

The above recommendations are based on the assumption that proper maintenance and irrigation of the areas adjacent to the pavement will occur through the design life of the pavement. Failure to maintain a proper maintenance and/or irrigation program may jeopardize the integrity of the pavement.

Earthwork recommendations regarding aggregate base and subgrade are provided in the previous section “Site Earthwork” and the related sub-sections of this report.

4.8 Control of Surface Water and Drainage Control

Positive drainage of surface water away from structures is very important. Water should not be allowed to pond adjacent to buildings or to flow freely down a graded slope. Per the 2010 CBC, positive drainage may be accomplished by providing drainage away from buildings at a gradient of at least 5 percent for earthen surfaces for a distance of at least 10 feet away from the face of a wall. If a distance of 10 feet cannot be achieved, an alternative of a gradient of at least 5 percent to an area drain or swale having a gradient of 2 percent is acceptable. Where necessary, drainage paths may be shortened by use of area drains and collector pipes. Eave gutters are recommended and should reduce water infiltration into the subgrade soils if the downspouts are properly connected to appropriate outlets.

Planters with open bottoms adjacent to buildings should be avoided. Planters should not be designed adjacent to buildings unless provisions for drainage, such as catch basins, liners, and/or area drains, are made. Overwatering must be avoided.

4.9 Subsurface Water Infiltration

Recent regulatory changes have recommended that low flow runoff be infiltrated rather than discharged via conventional storm drainage systems. In general, the vast majority of geotechnical distress issues are directly related to improper drainage. In general, distress in the form of movement of improvements could occur as a result of soil saturation and loss of soil support, expansion, internal soil erosion, collapse and/or settlement. Infiltrated water may enter underground utility pipe zones and migrate along the pipe backfill, potentially impacting other improvements located far away from the point of

infiltration.

The site primarily consists of fine-grained (i.e., silt and clay) soils that are not considered permeable and thereby not good for infiltration. We do not recommend that water be intentionally infiltrated at this site.

4.10 Grading Plan Review

Grading and foundation plans should be reviewed by LGC Geotechnical when they are available. Additional and/or revised recommendations from this report may be required. Additional field investigation and laboratory testing may also be required.

4.11 Geotechnical Observation and Testing During Construction

The recommendations provided in this report are based on limited subsurface observations and geotechnical analysis. The interpolated subsurface conditions should be checked in the field during construction by a representative of LGC Geotechnical. Geotechnical observation and testing is required per Section 1704.7-9 of the 2010 California Building Code (CBC).

Geotechnical observation and/or testing should be performed by LGC Geotechnical at the following stages:

- During grading (removal bottoms, etc) and during placement of any fill;
- During utility trench backfill and compaction;
- After presoaking building pads and other concrete-flatwork subgrades, and prior to placement of aggregate base or concrete;
- Preparation of pavement subgrade and placement of aggregate base;
- After excavation for the retaining walls and during retaining wall backfill;
- After building and wall footing excavation and prior to placing concrete and/or reinforcement; and
- When any unusual soil conditions are encountered during any construction operation subsequent to issuance of this report.

5.0 LIMITATIONS

Our services were performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable soils engineers and geologists practicing in this or similar localities. No other warranty, expressed or implied, is made as to the conclusions and professional advice included in this report.

This report is based on data obtained from limited observations of the site, which have been extrapolated to characterize the site. While the scope of services performed is considered suitable to adequately characterize the site geotechnical conditions relative to the proposed development, no practical evaluation can completely eliminate uncertainty regarding the anticipated geotechnical conditions in connection with a subject site. Variations may exist and conditions not observed or described in this report may be encountered during grading and construction.

This report is issued with the understanding that it is the responsibility of the owner, or of his/her representative, to ensure that the information and recommendations contained herein are brought to the attention of the other consultants (at a minimum the civil engineer, structural engineer, landscape architect) and incorporated into their plans. The contractor should properly implement the recommendations during construction and notify the owner if they consider any of the recommendations presented herein to be unsafe, or unsuitable.

The findings of this report are valid as of the present date. However, changes in the conditions of a site can and do occur with the passage of time, whether they be due to natural processes or the works of man on this or adjacent properties. The findings, conclusions, and recommendations presented in this report can be relied upon only if LGC Geotechnical has the opportunity to observe the subsurface conditions during grading and construction of the project, in order to confirm that our preliminary findings are representative for the site. This report is intended exclusively for use by the client, any use of or reliance on this report by a third party shall be at such party's sole risk.

In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and modification.

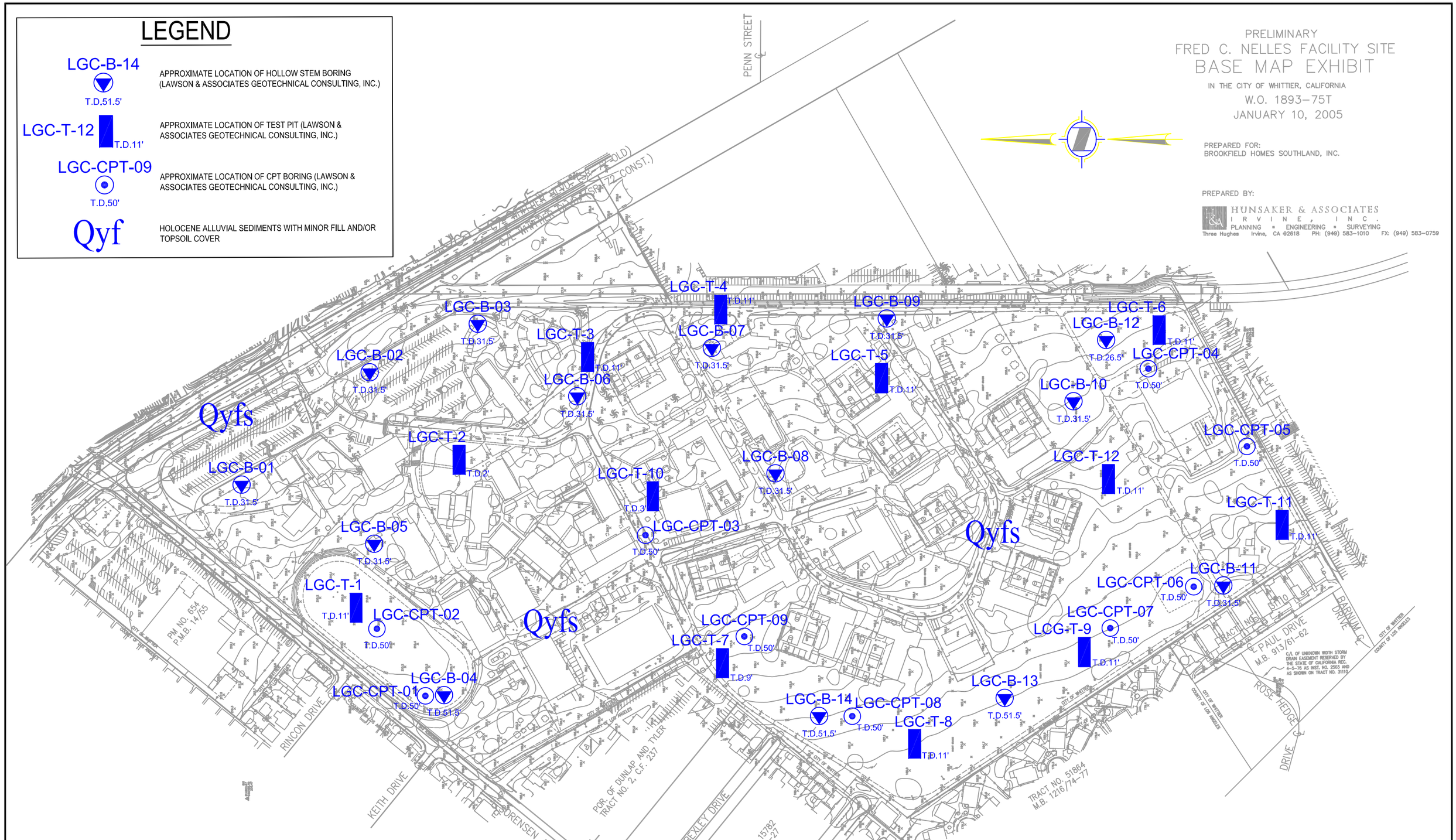


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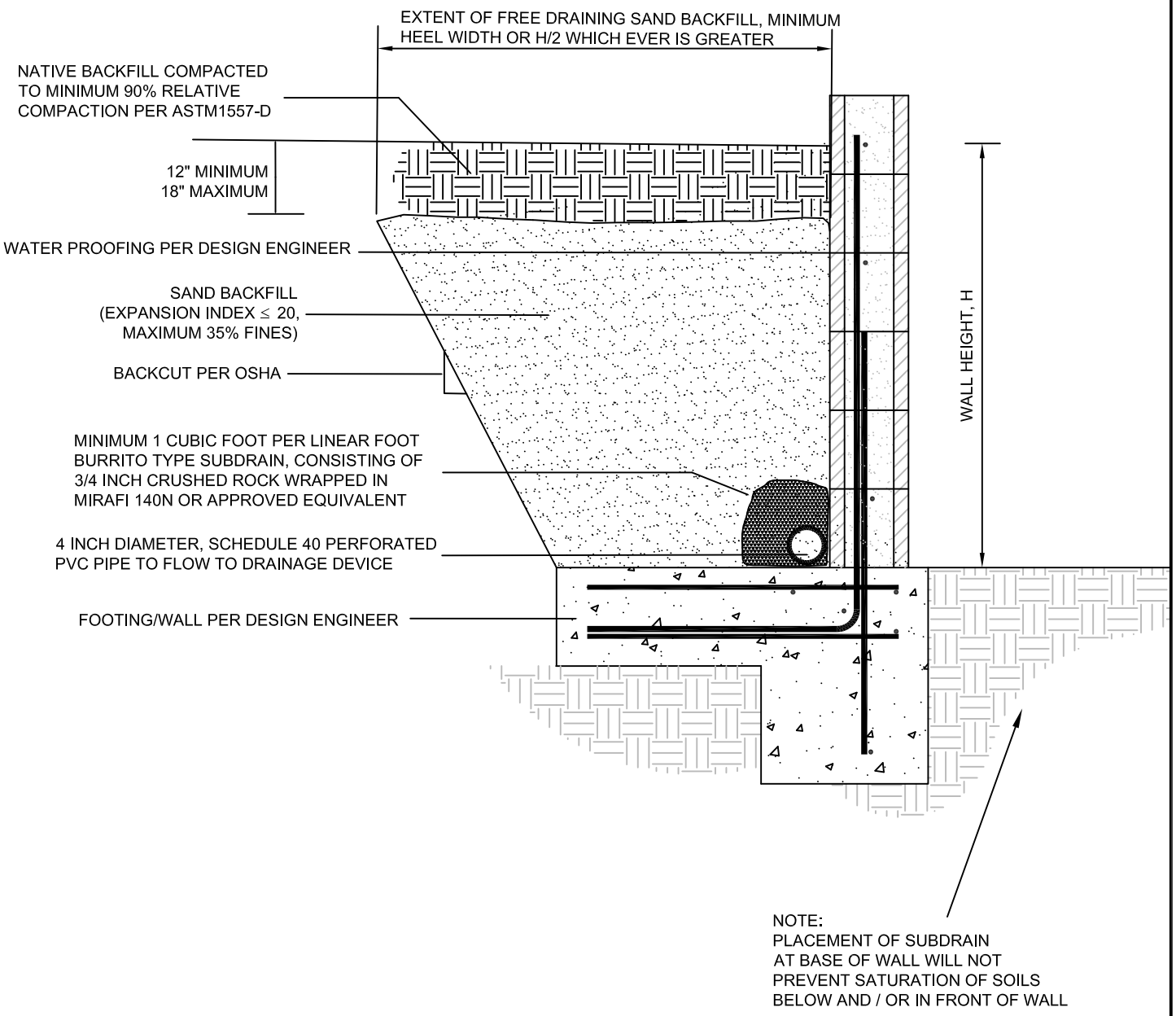
FIGURE 2
Boring, Test Pit, CPT Location Map

CLIENT:
Brookfield Homes

CIVIL ENGINEER:
Hunsaker & Associates

PROJECT NAME	*Brookfield Whittier - Nelles*
PROJECT NO.	*12204-01*
ENG. / GEOL.	*DJB / TJL*
SCALE	*1 inch = 250 feet*
DATE	*April 2013*



NOTE:
PLACEMENT OF SUBDRAIN AT BASE OF WALL WILL NOT PREVENT SATURATION OF SOILS BELOW AND / OR IN FRONT OF WALL

LGC Geotechnical, Inc.

FIGURE 3
Retaining Wall Backfill and Subdrain Detail

PROJECT NAME	Brokkfield Whittier - Nelles
PROJECT NO.	12204-01
ENG. / GEOL.	DJB / TJL
SCALE	Not to Scale
DATE	April 2013

Appendix A
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APPENDIX A

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