LGC Geotechnical, Inc.

April 28, 2017 Project No. 14057-01

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Subject: ***Preliminary Geotechnical Report for Proposed Rancho La Habra Residential Development, VTTM 17845, City of La Habra, California***

In accordance with your request and authorization, LGC Geotechnical, Inc. has prepared a preliminary geotechnical report for the proposed "Rancho La Habra" residential development generally located south of Imperial Highway and an adjacent commercial center between Beach Boulevard and Idaho Street within the City of La Habra, California. The purpose of our study was to evaluate the existing onsite geotechnical conditions and to confirm that the site can be developed from a geotechnical perspective.

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

Respectfully Submitted,

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1.0 INTRODUCTION

1.1 Purpose and Scope of Services

This report presents the results of our preliminary geotechnical evaluation for the proposed "Rancho La Habra" residential development, Vesting Tentative Tract No. 17845, located in the City of La Habra, California. The preliminary grading plan by Hunsaker and Associates, Inc. (Hunsaker, 2017) was utilized as a base map for our Geotechnical Map (Sheet 1), and Geotechnical Cross-Sections (Sheets 2 & 3).

The purpose of our study was to evaluate the existing onsite geotechnical conditions and to confirm that the site can be developed from a geotechnical perspective. As part of this report, we have: 1) reviewed available geotechnical reports, geologic maps, and air photos pertinent to the site (Appendix A); 2) performed a subsurface geotechnical evaluation of the site; 3) prepared a geotechnical map of the site incorporating available geotechnical information; 4) prepared geotechnical cross-sections depicting the interpreted subsurface conditions of the site relative to the proposed design; 5) performed global slope stability analysis in support of the proposed design; and 6) prepared this summary report presenting our preliminary findings and conclusions for the proposed development.

The findings and conclusions presented herein should be considered preliminary and will need to be confirmed as part of a grading plan review report to be provided at a later date. It should be noted that LGC Geotechnical does not provide environmental consulting services.

1.2 Existing Conditions

The approximately 151-acre site is currently an 18-hole golf course at the location depicted on the Site Location Map, Figure 1 (Page 7). The golf course includes a clubhouse, associated parking areas and cart paths, a driving range, and three lake features. The irregular-shaped parcel is bound on the west by Beach Boulevard, to the north by existing commercial and residential development located along Imperial Highway, to the east by Idaho Street, and to the south by existing residential Tracts 15031 and 15030. An existing street, Trevino Avenue, traverses the site and provides access to the clubhouse at the central portion of the site and gated access to the residential tracts south of the site.

The site has moderate relief, the lowest in the northern portion at an approximate elevation of 220 feet, the highest at the southeastern corner up to an approximate elevation of 470 feet. The existing residential tracts to the south of the site are generally at higher elevations and separated from the subject site by an ascending slope.

1.3 Project Description

The Applicant proposes to construct on the approximately 151-acre Westridge Golf Course property, 422 homes, including 277 single-family homes and 145 multi-family residences, either a maximum of 20,000 square feet of retail and restaurant uses or 49 multi-family dwelling units adjacent to Beach Boulevard and the existing Westridge Plaza, and open space, trails, and public parks.

Access to the proposed Project site would be provided at three locations. The primary entrance is proposed from Beach Boulevard on the west side of the Project Site by adding a fourth leg to an existing three leg signalized intersection on Beach Boulevard with the Hillsborough Apartment complex. The eastern entry to the community would add a fourth leg to an existing three leg signalized intersection on Idaho Street at Sandlewood Avenue. The third entry to the proposed development is from the north from La Habra Hills Drive, which is the existing entry to the Westridge Golf Course. Access to all of the residential neighborhoods would be gated and all internal streets private. La Habra Hills Drive would be a public street (non-gated) extending south to the proposed Community Center and public park. Farther to the south, La Habra Hills Drive would extend to the Westridge neighborhood to continue to provide access to that community.

The proposal includes a water quality basin located in the western portion of the site with a maximum design water elevation of 208 feet above sea level (Hunsaker, 2017). A second surface detention/water quality basin is located at the eastern entry adjacent to Idaho Street. The maximum design water elevation for this basin is approximately 311 feet above sea level (Hunsaker, 2017). A below-grade modular wetland system is proposed in the northern portion of the site adjacent to La Habra Hills Drive.

Numerous Mechanically Stabilized Earth (MSE) retaining walls up to approximately 20 feet in height are proposed within the development including two mid-slope MSE retaining walls. In addition, numerous relatively small conventional retaining walls are proposed.

The maximum proposed cut and fill slopes are approximately 80 and 40 feet, respectively. The grading plan depicts planned cuts and fills (not including required remedial grading) up to approximately 55 and 35 feet, respectively. No changes are depicted at the existing central clubhouse, and minor changes depicted on the southern of two existing lake features near the clubhouse. The northern of the two lake features near the existing clubhouse and an existing lake feature at the western portion of the site are planned to be removed as part of the proposed development.

1.4 <u>Background</u>

The subject site is located within a portion of the former West Coyote Oil Field located along the southern boundary of La Habra. Petroleum production by Chevron and affiliated companies started in the region in the early 1900's and ceased in approximately 1995. As part of oil production activities, site topography was altered slightly with construction of oil extraction pads and roads throughout the hills (Continental, 2014). The transition from an oil field to a golf course was undertaken over several years with the involvement of various geotechnical and environmental consultants, as detailed below. Please note that the available reports and maps obtained from the City of La Habra documenting the geotechnical history of the site are incomplete in several cases, and some documents referenced within those reports were not available for review.

On October 1, 1968 while the site was still an active oil field, a fault ruptured offsite and to the east of the site near Idaho Street. Subsequent extensive fault trenching in 1970 by Converse, Davis, & Associates (Converse) allowed for detailed mapping of the approximately 3-inch, vertically offset, fault trace. A series of relatively short exploratory trenches were excavated across the observed ground surface rupture. The well-defined lineament was documented, evaluated, and became known as the "Unnamed West Coyote Hills Fault" as presented in the referenced Fault Evaluation Report and Supplemental Report published by California Department of Mines and Geology (CDMG, 1977 & 1978). CDMG, 1978, suggests that the probable cause of fault rupture was due to oil field operations (subsidence due to oil extraction and groundwater injection activities). The fault was subsequently assigned a State of California Earthquake Fault Zone in accordance with the Alquist-Priolo Act (CDMG, 1991). As is typical, the Earthquake Fault Zone extends well beyond the lineament of the fault in order to ensure that any potential fault splays are evaluated by a geologist prior to construction of habitable structures within the regulatory zone. The entire Earthquake Fault Zone has since been improved or developed for the current residential uses, roadways, park, golf course, etc.

Leighton and Associates (Leighton) performed a fault study of the region in the early 1990's by trenching the area of the subject site and the two proposed adjacent tracts to the south (Tracts 15030 and 15031). Numerous existing inactive north-south trending faults that characterize the uplifted Coyote Hills geologic structure for this area, are presented on a map and cross sections by Leighton (1992: Figures Only) that were reviewed as part of this study. None of the faults except the Unnamed West Coyote Hills Fault are considered to be active or potentially active. Leighton depicted the (offsite) location of the historically active Unnamed West Coyote Hills Fault as mapped by Converse and a recommended structural setback extending 100 feet from either side of the mapped fault lineament. The approximate fault lineament, the recommended structural setbacks, and the limits of the Earthquake Fault Zone are presented on the Geotechnical Map (Sheet 1).

A geotechnical field evaluation was performed in 1996 by Goffman, McCormick and Urban, Inc., (GMU, 1996: Text Only), as part of a grading plan review for the proposed golf course and adjacent Tracts 15030 and 15031 to the south. The field evaluation consisted of 60 test pits and 36 large-diameter borings up to approximately 66 feet below grade. Although the geotechnical figures showing boring logs and locations were not available as part of our review of the original report of geotechnical studies by GMU, those figures were obtained from a subsequent response to comments report by another consulting firm. Geosoils, Inc. became the geotechnical consultant of record for construction of the Westridge Golf Course and responded to geotechnical review comments in the referenced report (Geosoils, 1997). Selected borings logged by GMU are presented herein on the Geotechnical Map, Sheet 1 (In Pocket), and boring logs in Appendix B.

During the period between the fault evaluation report by Leighton, 1992, and the geotechnical investigation by GMU, 1996, demolition of concrete features was reportedly ongoing at the site. Limited information is available regarding an approximate 25-foot-deep excavation where alternating layers of concrete rubble and soil was placed under observation and testing by Leighton, as described in GMU, 1996. The approximate location of the concrete disposal area is depicted on the Geotechnical Map, Sheet 1; however, the actual elevations and details regarding the material are not available.

The existing golf course was rough graded between 1997 and 1999 during grading of adjacent residential Tracts 15030 and 15031 under geotechnical observation and testing by GeoSoils, Inc. (1999). Grading of the golf course generally involved cuts and fills up to approximately 35 feet, greater in select areas of the westernmost portion of the site. Remedial grading included removal and stockpiling of crude oil-affected native soils. Buttress keyways were constructed for stabilization of ascending slopes to the south of the golf course, and smaller stabilization fill keyways were constructed for the 2:1 (Horizontal to Vertical) slopes along the northern perimeter of the golf course. During grading, approximately nine ancient "major" and numerous "minor" northwest and northeast trending normal faults (typical for the region) were encountered and mapped by GeoSoils. Fault locations were generally as anticipated from the fault map presented in Leighton, 1992. As stated above, these faults were not considered active or potentially active and reportedly all residential lots within Tracts 15030 and 15031 exposing them were overexcavated and capped with a minimum of 5 feet of compacted fill, in order to minimize the potential for differential settlement (GeoSoils, 1999).

Notably, as shown on the Geotechnical Map (Sheet 1), no faults were mapped by GeoSoils during grading that are within or adjacent to the State of California Earthquake Fault Zone.

Grading of the golf course included remedial measures and placement of a subdrain system. Grading was performed with placement of both structural fill (i.e., compacted fill with a required minimum relative compaction of 90 percent) and non-structural fill considered unsuitable for support of structures (i.e., fill placed with a required minimum relative compaction of 85 percent). Structural fill was reportedly placed along the southern perimeter of the golf course in support of the adjacent developments (Tracts 15030 and 15031), below structures and utility alignments across the golf course, and within adjacent perimeter slopes and roadways (Geosoils, 1997). Non-structural artificial fill was placed within the central area of the western portion of the subject site and within several small canyons at the eastern portion of the site. Removals of the near-surface weathered alluvial deposits were reportedly limited in these areas and not specifically removed to competent native soils prior to fill placement (Geosoils, 1999).

Three controlled zones called environmental Re-Use Areas (RUA) are located within non-structural artificial fill at the western half of the site, where specifically "diluted" crude oil-affected soils were placed under environmental observation and testing by Miller Brooks Environmental, Inc. (1999). As part of remedial grading activities, the affected soils were stockpiled, blended as necessary and placed as artificial fill at specific locations in general accordance with project specifications. Evaluation of the crude oil-affected soils and potential impacts to future grading are being addressed by other members of the design team.

Selected subsurface investigation borings by GMU, previously constructed keyways and subdrains, geologic mapping by others, limits of structural and non-structural artificial fill, and limits of the concrete disposal area and environmental Re-Use Areas (RUA) are presented on the Geotechnical Map, Sheet 1 (In Pocket).

In 2001, a geotechnical evaluation was performed for the adjacent commercial center located northwest of the site (Eberhart, 2001). The commercial center is approximately 57 acres with large retail stores and associated parking. The field evaluation consisted of 121 hollow-stem borings (mostly shallow) and two large-diameter borings excavated up to approximately 50 feet below grade. Select borings were reviewed as part of this study.

In 2003, a geotechnical evaluation was performed for a banquet room addition to the existing golf clubhouse by Pacific Geosoils, Inc. (2003). Two small-diameter hand-auger borings were drilled to depths of approximately 12 and 13 feet below existing grade. Pacific Geosoils reported that the subsurface soils in the area of the proposed clubhouse construction consist of fill soils underlain by bedrock. The geotechnical report concluded that existing fill soils were unsuitable for support of foundations and recommended that the building addition be supported on drilled piers into site bedrock, in lieu of performing earthwork removals.

1.5 __Subsurface Geotechnical Evaluation__

LGC Geotechnical performed a subsurface geotechnical evaluation of the site consisting of the excavation of six large-diameter bucket auger borings, eighteen hollow-stem auger borings and eleven Cone Penetration Test (CPT) soundings to evaluate onsite geotechnical conditions.

Six bucket auger borings (B-1 through B-6) were drilled by Al-Roy Drilling and Haven Geotechnical under subcontract to LGC Geotechnical. The maximum depth of the bucket auger borings was approximately 115 feet below existing grade. The bucket auger borings were excavated to evaluate the geologic structure of the bedrock materials and to obtain samples for laboratory testing. The large-diameter boreholes were surface logged during excavation and downhole logged by an engineering geologist in order to obtain structural geologic information. Borings were subsequently backfilled with cuttings and tamped.

Eighteen hollow-stem borings (HS-1 through HS-18) were drilled by 2R Drilling, Inc. under subcontract to LGC Geotechnical. The depth of the hollow-stem borings ranged from approximately 21.5 to 76.5 feet below existing grade. An LGC Geotechnical representative observed the drilling operations, logged the borings, and collected soil samples for laboratory testing. The borings were excavated using a limited access drill rig equipped with 8-inch-diameter hollow-stem augers. Driven soil samples were collected by means of the Standard Penetration Test (SPT) and Modified California Drive (MCD) sampler generally obtained at 5-foot vertical increments. The MCD is a split-barrel sampler with a tapered cutting tip and lined with a series of 1-inch-tall brass rings. The SPT sampler and MCD sampler were driven using a 140-pound automatic hammer falling 30 inches to advance the sampler a total depth of 18 inches or until refusal. The raw blow counts for each 6-inch increment of penetration were recorded on the boring logs. Bulk samples were also collected and logged at select depths for laboratory testing. At the completion of drilling, the borings were backfilled and tamped.

Eleven Cone Penetration Test (CPT) soundings (CPT-1A through CPT-11B) were performed by Kehoe Testing and Engineering, Inc. under subcontract to LGC Geotechnical. CPT soundings were pushed to depths ranging between approximately 5 to 69 feet below existing grade, to practical refusal. The CPT soundings were pushed using an electronic cone penetrometer in general accordance with the current ASTM standards (ASTM D5778 and ASTM D3441). The CPT equipment consisted of a cone penetrometer assembly mounted at the end of a series of hollow sounding rods. The interior of the cone penetrometer is instrumented with strain gauges that allow the simultaneous measurement of cone tip and friction sleeve resistance during penetration. The cone penetration assembly is continuously pushed into the soil by a set of hydraulic rams at a standard rate of 0.8 inches per second while the cone tip resistance and sleeve friction resistance are recorded at approximately every 2 inches and stored in digital form. A specially designed all-wheel drive 25-ton truck provides the required reaction weight for pushing the cone assembly.

The approximate locations of borings and CPT soundings are shown on the Geotechnical Map (Sheet 1). Boring and CPT logs are presented in Appendix B.

1.6 ***<u>Laboratory Testing</u>***

Representative bulk and driven samples were retained for laboratory testing during our field evaluation. Laboratory testing included in-situ moisture content and in-situ dry density, Atterberg Limits, grain size analysis, consolidation, direct shear, torsional ring residual shear, torsional ring fully softened shear, expansion index, laboratory compaction and corrosion (sulfate, chloride, pH and minimum resistivity).

The following is a summary of the laboratory test results.

- Dry density of the samples collected ranged from approximately 95 pounds per cubic foot (pcf) to 134 pcf, with an average of 113 pcf. Field moisture contents ranged from approximately 1 percent to 26 percent, with an average of 13 percent.
- Twenty-Five Atterberg Limit (liquid limit and plastic limit) tests were performed. Results indicated Plasticity Index values ranging from 1 to 48.
- Twenty-Two fines content tests were performed and indicated fines content (passing No. 200 sieve) of approximately 7 to 84 percent. According to the Unified Soils Classification System (USCS), fifteen of the tested samples are classified as "coarse-grained" soil.
- Direct shear tests were performed on select driven samples and samples remolded to 90 percent relative compaction. The plots are provided in Appendix C.
- Two torsional ring shear tests were performed on respective grab samples of site clayey materials; one landslide rupture surface clay and one clayey bedrock sample. The plots are provided in Appendix C.
- Consolidation tests were performed on select samples. The deformation versus vertical stress plots are provided in Appendix C.
- Three Expansion Index (EI) tests were performed. Results indicate EI values ranging from 15 to 37, corresponding to "Very Low" and "Low" expansion potential.
- Laboratory compaction testing of five bulk samples indicated maximum dry density values ranging from 115.0 to 140.0 pounds per cubic foot (pcf) and optimum moisture contents ranging from 5.5 to 14.5 percent.
- Corrosion testing indicated soluble sulfate contents ranging from approximately 0.01 to 0.05 percent, chloride contents ranging from approximately 33 to 175 parts per million (ppm), pH values ranging from 7.6 to 8.2, and minimum resistivity values ranging from 478 to 1,898 ohm-cm.

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

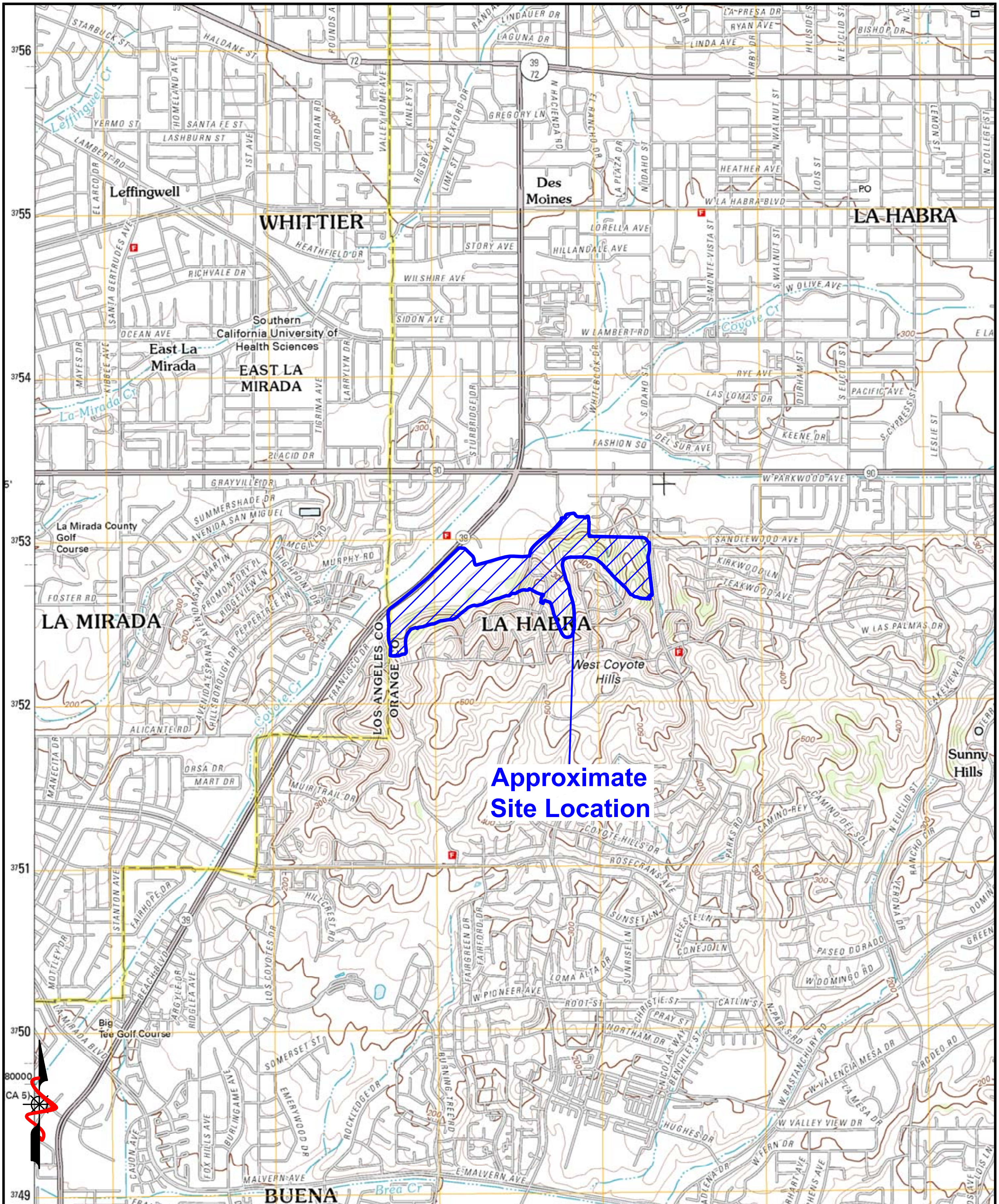


FIGURE 1
Site Location Map

PROJECT NAME	Rancho La Habra - VTTM 17845
PROJECT NO.	14057-01
ENG. / GEOL.	DJB / KTM
SCALE	Not to Scale
DATE	April 2017

2.0 GEOTECHNICAL CONDITIONS

2.1 Regional Geology

The site is located at the eastern edge of the Los Angeles Basin, within the Peninsular Ranges Geomorphic Province. The Los Angeles Basin consists of a thick sequence of sedimentary deposits partially derived from erosion of the nearby hills and mountain ranges to the north and east of the subject site. The La Habra Heights are a portion of the northwest-trending hills north of the subject area that are bound along their south edge by the active Whittier Fault. The smaller Coyote Hills that underlie the subject site are also of tectonic origin. The Coyote Hills produced oil from structural traps created by faulting and the presence of an east-west domal anticline that coincides with the topographic crest of the hills to the south of the site. The site is located on the gently north-dipping homoclinal limb of the anticline. In general, the region has a complex geologic history influenced by periods of uplift, subsidence, sea level changes, folding, and faulting (Morton, 2004). No active or potentially active faults are known to specifically transect the site.

2.2 Site-Specific Geology

The subject site is located partially on alluvium deposits and partially within the uplifted bedrock that forms the low hills of the Coyote Hills geologic complex south of the site (Morton, 2004). The comparatively young alluvium locally incised a broad, very old alluvial fan that spans the La Habra area to the north. The local alluvium consists of interfingered deposits derived from upstream to the northeast and from the low hills ascending to the south of the site. The bedrock unit underlying the site consists of Quaternary San Pedro Formation, and two existing landslides derived from this material have been identified within the limits of the site. Also, both structural and non-structural artificial fills mantle portions of the site. A brief description of these geologic units is presented in the following sections (from youngest to oldest) and their approximate lateral extents are depicted on the site Geotechnical Map (Sheet 1).

2.2.1 Artificial Fill – Older (Map Symbol - Afo)

Older artificial fill soils encountered at the subject site are considered structural, reportedly having been placed in relatively thin lifts, at near optimum moisture content, and compacted with heavy construction equipment to achieve a minimum relative compaction of at least 90 percent (GeoSoils, 1999). Structural fill was placed at certain locations for support of golf course structures and utility alignments, along the southern perimeter of the site for support of adjacent tracts to the south, and along the northern perimeter in areas of 2:1 (Horizontal to Vertical) slopes. The material consists of variable layers of silty clay to clayey sand with some gravel, generally moist to very moist, stiff to very stiff/dense.

2.2.2 Artificial Fill – Unsuitable (Map Symbol - Afu)

Unsuitable artificial fill soils were encountered within the western-central area of the subject site and within several small canyon fill areas within the eastern portion of the site as reported in GeoSoils, 1999. The unsuitable fill is considered non-structural as it was compacted to a minimum relative compaction of 85 percent. Reportedly, the non-structural fill was placed directly on native soils with minimal remedial grading, with the exception of areas where removals were performed for environmental mitigation (GeoSoils, 1999). The material consists of variable layers of silty clay to clayey sand with some gravel, generally moist to very moist, stiff to very stiff/medium dense to dense. Environmental Re-Use Areas (RUA), aka crude oil affected soils, and concrete disposal area are located within this material. The approximate limits of the RUA's and concrete disposal area are depicted on Geotechnical Map, Sheet 1.

2.2.3 Quaternary Alluvium (Map Symbol – Qal)

Quaternary alluvium underlies the majority of the western portion of the site (GeoSoils, 1999), undifferentiated from colluvium observed by others within the smaller canyons of the Coyote Hills to the south. It generally consists of medium dense silty and clayey sands and stiff to very stiff sandy clays with minor amounts of gravel.

2.2.4 Quaternary Landslide Deposit (Map Symbol – Qls)

Several bedrock-block type landslides were encountered in the vicinity of the subject site during previous investigations and grading activities by others (Goffman, 1996; GeoSoils, 1997 and 1999). Two landslides were removed during grading of the adjacent tracts to the south and one on-site landslide at the western side of the site was stabilized with shear keyways and left in place. Another small, relatively thin landslide was identified in a boring by others at the northeastern edge of the site that was subsequently determined to have been left in place based on Cross-Section 7-7' (Sheet 2).

Where encountered, the landslide material was observed to be similar to the bedrock unit at the site, but highly fractured and weathered. One landslide rupture surface was observed during downhole logging of boring B-6 to be based on a very thin clay bed oriented parallel to bedding.

2.2.5 Quaternary San Pedro Formation (Map Symbol – Qsp)

The sedimentary bedrock unit that underlies the site is the Pleistocene-age Quaternary San Pedro Formation, derived from a shallow marine depositional environment. The formation is broken into four units that vary in dominant material type, variably exposed between the faults that intersect the site. The fossiliferous material generally consists of sandy siltstone and minor amounts of clayey siltstone interbedded with medium to coarse, weakly to well-cemented sandstone. The eastern portion of the site has more sandstone, while the central and western portions of the site have a more variable mix of interbedded siltstone, sandstone, and clayey siltstone. Bedrock mapped by GeoSoils, 1999, that was indicated to be sandstone-dominated or an interbedded sandstone and siltstone, is differentiated on the Geotechnical Map, Sheet 1.

2.3 Geologic Structure

Geologic structure of the bedrock is controlled by the east-west trending domal anticline that crests with the original topographic high of the Coyote Hills to the south of the site. Bedding from GeoSoils, 1999, and as observed during our recent site investigation, indicates a gentle, west-northwesterly dip within the western portion of the site and an east-northeasterly dip within the eastern portion of the site. Bedding attitudes vary due to localized folding and faulting. Bedding was noted to have zones of high-energy depositional environments producing rip-up clasts and other pre-lithification sedimentary structures. Bedrock cementation ranged from non-cemented siltstone and friable sandstone to well cemented layers up to approximately 2 feet thick. Gypsum, manganese oxide and iron oxide were observed in the upper weathered portion of site bedrock.

An inactive fault encountered at depth within LGC-B-4 was observed along a steeply-dipping, tight, clay-lined shear that marked a sharp change in material type and bedding inclination. The observed fault is consistent with the many ancient faults identified by others prior to and during grading of the golf course and residential tracts to the south (Leighton, 1992 & Geosoils, 1999). Geologic information collected by others is presented on the Geotechnical Map (Sheet 1) along with the results of the recent subsurface investigation.

The landslide complex at the western portion of the site likely occurred as the result of a weak bedding layer, or "clay seam" becoming exposed over time as erosion of the north facing hillside eventually daylighted the weak feature. The geometry of the hillside, several ancient faults and the inclination of the weak bedding surface (out of slope) were likely triggers for the landslide. The landslide appears to be very old based on the muted geomorphology and lack of topographic expression. Landslide rupture surface (clay) was observed in Boring LGC-B-6.

2.4 <u>*Groundwater*</u>

During our subsurface evaluation, groundwater was encountered at a depth of approximately 29 feet in HS-17 and as seepage at depth within the large-diameter borings LGC-B-4, LGC-B-5 and LGC-B-6. Groundwater was not encountered in the remaining hollow-stem borings to the maximum explored depth of approximately 76.5 feet below existing grade. Groundwater was encountered at 46 to 49 feet below existing grade within the alluvium to the north of the site as investigated and reported by Eberhart (2001) as part of construction of the commercial development to the north of the site. The referenced grading plan review report by GMU, 1996, stated that "subsurface flow was encountered at depths of about 20 feet within the older alluvium underlying the major valley bottom area."

Seasonal fluctuations of groundwater elevations should be expected over time. In general, groundwater levels fluctuate with the seasons and local zones of perched groundwater may be present within the near-surface deposits due to local seepage or during rainy seasons. Local perched groundwater conditions or surface seepage may develop once site development is completed and landscape irrigation commences.

2.5 <u>*Faulting and Seismic Hazards*</u>

The Coyote Hills region surrounding the subject site has a complex geologic history influenced by periods of uplift, gentle folding, and faulting (Morton, 2004). Former uses of the subject site included a working oil field that was active for many years, as described in detail in the section titled "Background" (Section 1.4). On October 1, 1968 while the site was still an active oil field, a fault ruptured just east of Idaho Street as defined by approximately 3 inches of vertical displacement. Refer to the Geotechnical Map, Sheet 1, for the location of the fault with respect to the subject site.

Extensive fault trenching by Converse, Davis, & Associates in 1970 allowed detailed mapping of the fault trace including its lateral limits. The well-defined lineament was documented, evaluated and became known as the “Unnamed West Coyote Hills Fault” as presented in the referenced Fault Evaluation Report and Supplemental Report (CDMG, 1977 & 1978). CDMG, 1978 suggests that the probable cause of fault rupture was due to oil field operations (subsidence due to oil extraction and groundwater injection activities).

The Alquist Priolo Earthquake Fault Zoning Act of 1972 was adopted by the State of California to mandate areas of special geotechnical study with respect to potential surface rupture of active faults. The purpose of the regulatory zone is to promote careful geologic evaluation prior to construction of habitable structures. State of California Earthquake Fault Zones were established around “active” faults; the zones are defined by turning points connected by straight lines and they extend hundreds of feet beyond a hypothesized fault location. Per the State Geologist, a fault is generally considered “active” if evidence of surface rupture in Holocene time (the last approximately 11,000 years) is present. The Unnamed West Coyote Hills Fault has been evaluated to be “historically active,” because displacement has occurred within the last 200 years, therefore an Earthquake Fault Zone was established for the fault. More specifically, a minimum 100-foot-wide structural setback zone was recommended on either side of the Unnamed West Coyote Hills Fault as it was determined that the fault had been accurately located (CDMG, 1978 & Leighton, 1992). As shown on the Geotechnical Map (Sheet 1), the 100-foot wide recommended setback occurs on both sides of the mapped fault, for a total of 200 feet wide, while the “width” of the Earthquake Fault Zone is a total of approximately 500 feet wide. While the Earthquake Fault Zone establishes the boundary of the regulatory area, it does not restrict or limit development within the zone, provided that an evaluation is performed by a geotechnical professional in accordance with CGS (2007) to confirm the absence of active faults. For example, numerous residential structures have been constructed within the limits of the Earthquake Fault Zone in the area to the east of the proposed development.

The previously existing oil field at the subject site was re-developed in the late 1990’s into the Westridge Golf Course and adjacent residential communities to the south. These sites were rough graded under the observation and testing of GeoSoils (1997 &1999). The portion of the golf course that is included within the Earthquake Fault Zone was mapped by GeoSoils during grading at that time and no faults were reported there (Geosoils, 1999).

As shown on Sheet 1, proposed residential Lots 12, 28 and 29 are located within the limits of the Earthquake Fault Zone. However, the proposed residential structures for all three lots are located at least 200 feet away from the reported surface trace of the Unnamed West Coyote Hills Fault, which is significantly more than the previously recommended 100-foot structural setback (CDMG, 1978 & Leighton, 1992). Based on the information shown on Sheet 1 and our review of the referenced documents, it is our opinion that proposed Lots 12, 28 and 29 are not underlain by an active fault. However, those three lots will require verification geologic mapping during grading prior to construction of habitable structures. Recommendations relative to verification mapping are presented in the section titled “Preliminary Recommendations.” If an active fault is identified on-site within the limits of the Earthquake Fault Zone, no habitable structures shall be constructed within 50 feet of it.

Due to the distance from the proposed structures to the Unnamed West Coyote Hills Fault, the possibility of damage due to ground rupture is considered low, provided that the portion of development located within the limits of the Earthquake Fault Zone is verified to be lacking fault indicators in accordance with the current standards of practice, State of California guidelines, and City of La Habra guidelines.

Nearby active faults and corresponding monument magnitude are summarized in Table 1 below.

TABLE 1

Near Site Fault Parameters - Westridge VTTM 17845

Fault	Approximate Distance from Site (km)	Maximum Earthquake Magnitude (Mmax)
Puente Hills (Coyote Hills)	4.4	6.8*
Elsinore (Whittier Section)	5.2	6.9*
Puente Hills (Santa Fe Springs)	3.2	6.6*
Unnamed West Coyote Hills	<0.1	2.2 to 2.6**

* from State of California fault database (Caltrans, 2015b)
** from CDMG, 1977

The City of La Habra and surrounding communities experienced a Magnitude 5.1 earthquake on March 28, 2014, and numerous smaller foreshocks and aftershocks. The earthquake epicenter was estimated to be located about one mile east of La Habra, at a depth of approximately 3 miles below ground. Based on USGS reports, the earthquake was associated with the Puente Hills Blind Thrust Fault System, the same fault that caused the 1987 Whittier Narrows Earthquake (USGS, 2015), but not related to the Unnamed West Coyote Hills Fault that is located east of the project site. The north-dipping Puente Hills Blind Thrust Fault System extends north and west toward Los Angeles at depth. A blind thrust fault has no surface rupture and is therefore not zoned for inclusion in the State of California Earthquake Fault Zone maps. It's our current understanding that no changes to the Earthquake Fault Zone maps have been proposed as a result of the event.

The Seismic Hazards Mapping Act of 1990 mandated "Zones of Required Investigation" relative to the hazards presented by regional seismic shaking (hazards other than surface rupture). The site is located within an area with potential for seismic hazards as presented on Figure 2, Seismic Induced Hazard Map (Rear of Text). Secondary effects of seismic shaking resulting from large earthquakes on the major faults in the Southern California region, which may affect the site, include ground lurching, soil liquefaction, dynamic settlement and earthquake induced landslides. These secondary effects of seismic shaking are a possibility throughout the Southern California region and are dependent on the distance between the site and causative fault and the onsite geology. A discussion of these secondary effects and proposed mitigation in accordance with the provisions of the Seismic Hazards Mapping Act is provided in the following sections.

2.5.1 Liquefaction and Dynamic Settlement

Liquefaction is a seismic phenomenon in which loose, saturated, granular soils behave similar 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. Studies indicate that saturated, loose, near surface cohesionless soils exhibit the highest liquefaction potential, while dry, dense, cohesionless soils and cohesive soils exhibit low to negligible liquefaction potential. In general, cohesive soils are not considered susceptible to liquefaction (Bray & Sancio, 2006). Effects of liquefaction on level ground include settlement, sand boils, and bearing capacity failures below structures. Dynamic settlement of dry loose sands can occur as the sand particles tend to settle and densify as a result of a seismic event.

A portion of the site is located within a State of California Seismic Hazard Zone (CDMG, 1998) for liquefaction potential; refer to Figure 2 (rear of text). The majority of the developed site will consist of compacted fill over dense/hard bedrock and is not considered susceptible to liquefaction. However, the west portion of the site contains alluvial soils that may be susceptible to liquefaction depending primarily on their apparent density (e.g., loose to dense) and plasticity. The majority of the alluvial soils tested are considered to be cohesive and not considered to be susceptible to liquefaction based on their saturated moisture content compared to their Liquid Limit (Bray & Sancio, 2006); refer to Table 1 provided in Appendix D. Dense to very dense sandy soils are also present that are not considered susceptible to liquefaction. However, the data obtained from our field evaluation indicates that the site contains isolated sandy layers susceptible to liquefaction in the upper 50 feet. Liquefaction potential was evaluated using the procedures outlined by Special Publication 117A (SCEC, 1999 & CGS, 2008) and the applicable seismic criteria (e.g., 2016 CBC). Liquefaction analysis is based on a conservative groundwater elevation of 195 feet using the program CLiq (GeoLogismiki, 2017). The soil type interpretations of CPT soundings show good agreement to laboratory testing of fines content and Plasticity Index from samples obtained from adjacent borings, refer to Figure 3 and 4. Changes in design grade (either cut or fill) are incorporated into the calculations by adjustment of the total and effective stress for the induced cyclic stress ratio. Results indicate total seismic settlement on the order of 2 inches or less. Differential seismic settlement can be estimated as half of the total estimated settlement over a horizontal span of about 30 feet. Liquefaction calculations are provided in Appendix D.

2.5.2 <u>Lateral Spreading</u>

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.

Site sandy soils generally have a normalized clean sand tip resistance well above 70. A normalized clean sand tip resistance of 70 corresponds to a blow count $(N_1)_{60}$ of at least 15. Soils with a corrected SPT $(N_1)_{60}$ blow count of 15 or greater are generally not considered susceptible to lateral spreading (Youd, Hansen, Bartlett, 2002). Evaluation of lateral spreading potential were performed on Cross Sections 11-11', 12-12' and 15-15'. In our analysis, we modeled alluvium using the lower of the liquefied residual strength from the CPT's or the static shear strength based on the stress range. Liquefied shear strength values are based on Robertson, 2010. Plots of liquefied S_u/σ'_v ratios are provided in Appendix D. The seismic coefficient (Kh) is based on the scaled design earthquake peak ground acceleration using a 5-cm displacement threshold per Special Publication 117A. Lateral spreading analysis indicates adequate factors of safety. Refer to Appendix D.

2.5.3 <u>Earthquake Induced Landslide</u>

A portion of the site is located within a State of California Seismic Hazard Zone (CDMG, 1998) for earthquake-induced landslide, as depicted on Figure 2 (rear of text). The hillside zones depicted on the seismic hazard potential map were originally delineated on the pre-existing topography of the region and are no longer applicable. The hillside remedial grading performed in the late 1990's as reported in GeoSoils, 1999, specifically mitigated this seismic hazard potential. Remedial grading for the golf course and the existing residential tracts to the south consisted of buttress keyways and replacement fill slopes as evaluated during this phase of site investigation and presented on the Geotechnical Map and Cross-Sections (Sheets 1 through 3). For this reason, potential for earthquake-induced landslide at the site is considered low.

2.6 Seismic Design Criteria

The site seismic characteristics were evaluated per the guidelines set forth in Chapter 16, Section 1613 of the 2016 CBC. Since the site contains soils that are susceptible to liquefaction (refer to Section 2.5.1), ASCE 7 which has been adopted by the CBC requires that site soils be assigned Site Class "F" and a site-specific response spectrum be performed. However, in accordance with Section 20.3.1 of ASCE 7, if the fundamental periods of vibration of the planned structure are equal to or less than 0.5 second (anticipated for the planned residential structures), a site-specific response spectrum is not required and ASCE 7/2016 CBC site class and seismic parameters may be used in lieu of a site-specific response spectrum. It should be noted that the seismic parameters provided herein are not applicable for any structure having a fundamental period of vibration greater than 0.5 second. Representative site coordinates of latitude 33.9146 degrees north and longitude -117.9640 degrees west were utilized in our 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 2 on the following page.

Seismic design criteria as detailed above are based on the currently applicable 2016 CBC (utilizing the 2008 National Seismic Hazard Maps and updated by the Building Seismic Safety Council, 2009). The national seismic hazard maps were updated in 2014 by USGS. It is our understanding that these updates reflect minor improvements in fault probability evaluations (USGS, 2014). If the 2014 National Seismic Hazard Maps are adopted by future editions of the California Building Code, the seismic design parameters presented herein will be updated as necessary. The actual seismic design will be based on the CBC that the City of La Habra has adopted at the time of plan check/permitting.

TABLE 2

Seismic Design Parameters

Selected Parameters from 2016 CBC, Section 1613 - Earthquake Loads	**Seismic Design Values**
Site Class per Chapter 20 of ASCE 7	D*
Risk-Targeted Spectral Acceleration for Short Periods (S_S)**	1.973g
Risk-Targeted Spectral Accelerations for 1-Second Periods (S_1)**	0.713g
Site Coefficient F_a per Table 1613.3.3(1)	1.0
Site Coefficient F_v per Table 1613.3.3(2)	1.5
Site Modified Spectral Acceleration for Short Periods (S_{MS}) for Site Class D [Note: $S_{MS} = F_a S_S$]	1.973g
Site Modified Spectral Acceleration for 1-Second Periods (S_{M1}) for Site Class D [Note: $S_{M1} = F_v S_1$]	1.070g
Design Spectral Acceleration for Short Periods (S_{DS}) for Site Class D [Note: $S_{DS} = (^2/_3)S_{MS}$]	1.315g
Design Spectral Acceleration for 1-Second Periods (S_{D1}) for Site Class D [Note: $S_{D1} = (^2/_3)S_{M1}$]	0.713g
Mapped Risk Coefficient at 0.2 sec Spectral Response Period, C_{RS} (per ASCE 7)	0.951
Mapped Risk Coefficient at 1 sec Spectral Response Period, C_{R1} (per ASCE 7)	0.970
PGA_M (Section 11.8.3 of ASCE 7)	0.760g

* Site Class F modified due to building period ≤ 0.5 second, refer to discussion above.

** From USGS, 2017

A deaggregation of the PGA based on a 2,475-year average return period indicates that an earthquake magnitude of 6.7 at a distance of approximately 4.9 km from the site would contribute the most to this ground motion. A deaggregation of the PGA based on a 475-year average return period indicates that an earthquake magnitude of 6.7 at a distance of approximately 5.1 km from the site would contribute the most to this ground motion (USGS, 2008).

2.7 Soil Shear Strength Parameters

The soil shear strength parameters utilized in our slope stability analysis are based on laboratory testing of the onsite materials, previous site shear strength parameters and published shear strength data (CDMG, 2001). The soil shear strength for the landslide rupture surface is based on results of a residual torsional ring shear test from rupture surface clay materials obtained during downhole logging of B-6. Where applicable, soil shear strength parameters were increased (less than composite peak strength values) for seismic loading conditions. Laboratory test results are provided in Appendix C.

TABLE 3

Soil Shear Strength Parameters for Static Slope Stability Analysis

Soil Type	**ϕ (Degrees)**	**Cohesion (psf)**
Qsp – Cross Bedding	32	300
Qsp - Along Bedding	27	200
Compacted Fill	30	150
Landslide Material	24	200
Landslide Rupture Surface	13	0
Alluvium	28	200

2.8 Global Slope Stability Analyses

Global slope stability analyses were performed on cross-sections positioned throughout the site based on the proposed design profile. Slope stability analysis was performed using the computer program GSTABL7 with STEDwin version 2.005.3 (Gregory Geotechnical Software, 2013). Potential rotational and block surfaces were analyzed using Bishop's Modified Method and Janbu's Simplified Method, respectively. A minimum factor of safety of 1.5 is typically required for static loading conditions. Seismic slope stability analysis was performed in accordance with the County of Orange Grading Manual (1991). Where applicable, the County of Orange Grading Manual requires a horizontal seismic coefficient (Kh) of 0.15 with a minimum resulting factor of safety of 1.1. Since the landslide rupture plane is less than 12 degrees from the horizontal, pseudostatic (seismic) slope stability was not performed for the landslide on Cross Section 1-1' and 2-2' in accordance with County of Orange Grading Manual.

Geogrid reinforcement (beyond requirements for local stability) is required for the planned mid-slope MSE retaining wall located below Lots 241 through 245 for adequate global factors of safety (Cross-Section 13-13').

For the proposed water quality basin depicted on Cross-Sections 1-1' and 2-2', slope stability analysis was performed for a "rapid draw-down" condition based on the peak water level. Rapid draw-down is considered a short-term condition. Based on preliminary information provided by the project civil engineer, the water quality basin will have a peak water storage elevation of approximately 208 feet above mean sea level.

Based on the proposed grading plan, slope stability analysis indicates a global factor of safety greater than 1.5 and 1.1 for static and pseudo-static (seismic) loading conditions, respectively. Slope stability analysis is provided in Appendix E.

Slope stability analyses for temporary back-cuts were performed on Cross Section 5, 8, 12 and 13. Temporary backcuts are a maximum of 1.6:1 (horizontal to vertical). Slope stability analysis indicated factors of safety of at least 1.25 for temporary conditions. Refer to Appendix E.

Additional slope stability analysis may need to be performed once the 40-scale rough grading plans have been prepared and more specific details are available regarding finalized slopes and MSE wall configurations, etc. This additional analysis may include additional cross-sections and static and pseudo-static slope stability analysis for confirmation of localized stabilization recommendations prior to earthwork activities.

2.9 ***Expansion Potential***

Based on the results of previous nearby and current laboratory testing, site soils have a "Very Low" to "Very High" expansion potential. Final expansion potential of site soils should be determined at the completion of grading. Results of expansion testing at finish grades will be utilized to confirm final foundation design.

2.10 ***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 on buried concrete and metal facilities. We therefore present the results of our testing with regard to corrosion for the use of the client and other consultants, as they determine necessary. Preliminary corrosion testing indicated soluble sulfate contents ranging from 0.01 to 0.05 percent, chloride contents ranging from 33 to 175 parts per million (ppm), pH values ranging from 7.6 to 8.2 and minimum resistivity values ranging from 478 to 1,898 ohm-cm. Sulfate testing from the adjacent tracts to the south indicated sulfate contents ranging from approximately 0.02 to 0.56 percent (GSI, 1999). Based on Caltrans Corrosion Guidelines, soils are considered corrosive to structural elements 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 (0.2 percent) or greater (Caltrans, 2015a).

Based on preliminary and previous laboratory sulfate test results, the near-surface soils have a severity categorization of "Not Applicable" to "Severe" and are designated to classes "S0" and "S2" per ACI 318, Table 4.2.1 with respect to sulfates. Determination of design elements with respect to sulfates must be based on as-graded conditions.

2.11 ***Infiltration Potential***

Based on our site evaluation and subsurface investigation, the majority of site soils (i.e., bedrock, fill and alluvium) are predominately fine-grained silts and clays that are known to have a very low hydraulic conductivity and therefore have very low infiltration rates. Within the majority of small and large-diameter borings excavated during our subsurface field work, groundwater was not encountered in spite of the substantial amount of water utilized at the golf course every year. Isolated seepage was observed at depth in several borings; however, no signs of surface infiltration were observed. Although we understand that the City of La Habra considers the geologic formation underlying the golf course to be a source of recharge to the City's groundwater basin (Malcolm Pirnie, 2010), it is our opinion that infiltration of surface water is not currently occurring through the surface of the golf course.

At the completion of grading, the proposed development will consist of either a thick cap of compacted fill (approximately 30 to 60 feet thick) over alluvium at the western half of the site or a thinner cap of compacted fill over bedrock at the eastern half of the site. Engineered fill is considered unacceptable for infiltration in accordance with the Orange County Technical Guidance Document "Section VII.1.6, Fill Condition," that states infiltration must extend to native soils below the fill (County of Orange, 2013). However, siltstone bedrock (native soils) below the engineered fill is also unacceptable for infiltration from a geotechnical standpoint. By definition, bedrock materials do not readily transmit water. The onsite bedrock materials are generally more than 50 percent fine-grained (i.e., clays and silts corresponding to hydrologic soil groups C and D) and bedrock structure includes numerous clayey siltstone interbeds that consist entirely of fine-grained material. Deeply buried alluvium at the western portion of the site is also not a geotechnically acceptable medium for infiltration due to that portion of the site being located in a mapped State of California Seismic Hazard Zone for liquefaction potential as depicted on Figure 2, Seismic Induced Hazard Map (Rear of Text) (CDMG, 1998).

Based on our review of the CPT and boring logs, sandy layers are not continuous. In addition, laboratory testing indicated fines content ranging approximately 29 percent to 62 percent. Soils with a combination of high fines content and high CPT tip resistances are considered to have low hydraulic conductivity. Correlation of CPT sounding soil behavior type and fines content is provided on fence diagrams (Figure 3 and 4).

Purposeful infiltration of water to the subsurface at the subject site is neither feasible nor acceptable from a geotechnical standpoint given the onsite materials and the hillside nature of the site.

3.0 CONCLUSIONS

Based on the results of our subsurface evaluation and geotechnical review of the proposed plan, it is our opinion that the proposed improvements are feasible from a geotechnical standpoint, provided that the recommendations provided here and in future reports are incorporated during site grading and development. A summary of our geotechnical conclusions are as follows:

- The bedrock geologic unit mapped on the site is the Quaternary San Pedro Formation. Artificial fill placed during original grading of the existing golf site overlies alluvium and bedrock across the majority of the site.
- The site contains soils including previously placed non-structural fill and near-surface alluvium that are not suitable for the proposed development in their present condition. Earthwork removals will be required as outlined in the recommendation section.
- Groundwater was encountered at a depth of approximately 29 feet in HS-17 and as seepage at depth within the large-diameter borings LGC-B-4, LGC-B-5 and LGC-B-6. Groundwater was not encountered in the remaining hollow-stem borings to the maximum explored depth of approximately 76.5 feet below existing grade. Design groundwater is estimated at elevation 195 feet within site alluvium. While isolated seepage was observed within the sandy layers of the San Pedro Formation bedrock, it is not considered a groundwater table. Groundwater may be encountered during grading and geotechnical recommendations provided as necessary based on field conditions.
- Proposed earthwork at the site including overexcavation of transition and design cut pads, installation of new subdrains and extension of existing canyon subdrains, and construction of replacement fill keyways with backdrain/subdrains for design cut slopes, are anticipated to mitigate much of the potential for nuisance water to develop. If localized nuisance water issues develop after grading, they should be addressed on a case-by-case basis.
- A portion of the site is located in a State of California Seismic Hazard Zone for liquefaction potential (Figure 2, rear of text). The majority of the developed site will consist of compacted fill over dense/hard bedrock and not considered susceptible to liquefaction. However, a portion of the site contains alluvial soils that are generally considered susceptible to liquefaction depending on their apparent density and plasticity. Based on lab testing, the majority of site alluvial soils is cohesive and not considered susceptible to liquefaction. However, subsurface data indicates that relatively isolated sandy layers within alluvial soils are susceptible to liquefaction and dynamic settlement within the upper 50 feet. Total dynamic settlement is estimated to be on the order of 2 inches or less. Differential dynamic settlement can be estimated at half of the total settlement over a horizontal span of 30 feet for design of foundations.
- A portion of the site is located within a State of California Seismic Hazard Zone as having potential for earthquake-induced landslides (Figure 2, rear of text). This potential hazard has been generally mitigated with remedial grading measures including buttress keyways that were constructed during grading of the residential tracts south of the golf course.
- As shown on the Geotechnical Map, Sheet 1, proposed residential Lots 12, 28 and 29 are located within the limits of a State of California Earthquake Fault Zone The proposed residential structures for all three lots are located at least 200 feet from the "Unnamed West Coyote Hills Fault," significantly more than the recommended 100-foot structural setback from the well-defined fault location (CDMG, 1978 & Leighton, 1992). Based on our review of applicable documents, it is our opinion that proposed Lots 12, 28 and 29 are not underlain by an active fault. However, this will have to be verified during grading as detailed in the following "Preliminary Recommendations" section.
- Due to the distance from the proposed structures to the surface trace of the Unnamed West Coyote Hills Fault (more than approximately 200 feet apart at the closest location) and the fact that oil production has

ceased in the immediate vicinity, the potential for renewed fault activity to impact the proposed development is considered low.

- The Unnamed West Coyote Hills Fault was hypothesized to be active due to the oil production operations and it is not believed to be linked to the major active fault systems that currently interact with the plate tectonics of Southern California. It is highly unlikely that future oil production will occur adjacent to the project, due to the presence of residential and commercial development on all sides. In our professional opinion, potential future activity on the fault would cause less ground shaking at the site than would from one of the major tectonic faults in the region.
- Proposed increase of grades over existing alluvium in portions of the site is estimated to induce on the order of 2½ inches to approximately 6¾ inches of settlement within the alluvium. It is our opinion that site clays are more over consolidated and the actual settlement will be closer to approximately 2½ inches and long-term secondary settlement is negligible. Site alluvial soils are generally very stiff sandy clays and dense clayey sands. Based on laboratory test data consisting of in-situ moisture content and Atterberg Limits (Liquidity Index) and CPT data (tip resistance and interpreted OCR ratio), clayey soils are considered over-consolidated. Significant increase in grades are proposed overlying left in place alluvium along Cross Section 5-5', near CPT-10. Based on interpretation of CPT-10 the minimum over-consolidation ratio of the left in place alluvium is approximately 5. The higher settlement estimate is based on using P'c values estimated from consolidation curves. The lower estimate is based on over-consolidated clays with an approximate average Crε value. These settlement estimates should be further evaluated as part of the grading plan review report based on actual proposed finished grades. Please note that settlement monitoring will be performed starting at the end of grading and construction of proposed structures will not commence until monitoring data indicates that future projected settlement (both primary and secondary) is within tolerable limits. Settlement calculations are provided in Appendix F. Plots of the over-consolidated ratio (OCR) based on CPT data is provided in Appendix D.
- Due to planned fill depths and/or significant increasing of grades over existing alluvium, settlement monitoring will be required at the completion of grading and prior to construction of structures in these areas. Settlement of alluvial soils is estimated to take approximately 6 to 12 months after the completion of rough grading, it is our opinion it will be closer to 6 months to complete. It should be noted that left in place alluvial soils that will experience significant grade increases are generally inter-bedded with sandy layers which will increase the rate of consolidation and decreasing total settlement duration. Refer to CPT-10 and CPT-11B located near Cross Section 5-5'. Refer to Calculations provided in Appendix F.
- The main seismic hazard that may affect the site is from ground shaking from one of the active regional faults. The subject site will likely experience strong seismic ground shaking during its design life.
- Based on the results of our evaluation and analysis provided herein, and provided our recommendations are properly implemented during construction, the proposed development of the site is not anticipated to significantly impact adjacent perimeter properties.
- Design fill slopes are anticipated to be both grossly and surficially stable, as long as they are constructed in accordance with our geotechnical recommendations and are properly landscaped and maintained throughout their design life.
- Existing native and cut slopes surrounding the development are anticipated to be grossly stable; however, minor surficial failures may occur.
- Global slope stability analysis indicates that (global) geogrid reinforcement is necessary in order to provide an adequate factor of safety for the proposed MSE walls located below Lots 241 through 245 (Cross-Section 13-13').
- From a geotechnical perspective, the existing onsite soils including existing fill are considered suitable material for use as general fill (with the exception of MSE wall backfill and conventional retaining wall backfill), provided that they are relatively free from rocks (larger than 8 inches in maximum dimension),

construction debris, and significant organic material. Significant moisture conditioning will be required to obtain the required compaction. It should be noted that portion of the site contains soils that are suitable for backfill of MSE and conventional retaining walls. However, select grading and/or stockpiling will be required. In addition, import of soils suitable for backfill of MSE and conventional retaining walls will also likely be required.

- Specific zones of crude oil-affected artificial fill will require evaluation and recommendations by the project environmental consultant. LGC Geotechnical is not an environmental consultant.
- Based on the results of previous nearby and current laboratory testing, site soils have a “Very Low” to “Very High” expansion potential. Mitigation measures will be required for any planned foundations and site improvements such as concrete flatwork to minimize the impacts of expansive soils. In addition, improvements located in close proximity to adjacent slopes will be impacted by slope creep. Final expansion potential of site soils should be determined at the completion of grading.
- Site soils (i.e., bedrock, fill and alluvium) are predominately fine-grained silts and clays which have very low permeability and therefore have very low infiltration rates. It is our opinion that infiltration of surface water is not currently occurring. At the completion of grading, the proposed development will consist of either a thick cap of compacted fill over alluvium or a thin cap of compacted fill over bedrock and therefore purposeful infiltration of water is not feasible nor recommended from a geotechnical standpoint.

4.0 PRELIMINARY RECOMMENDATIONS

A grading plan review report based on the 40-scale rough grading plans should be prepared in order to provide updated geotechnical recommendations (as necessary) for the proposed development. Additional field work and laboratory testing may be required. Additional and/or modified geotechnical recommendations may also be required.

Based on our preliminary study, the following is a summary of our preliminary geotechnical recommendations.

- The majority of artificial fill placed for construction of the existing golf course is considered non-structural and is not suitable for support of the proposed residential/commercial development in its current condition. Removals of unsuitable material up to approximately 50 feet deep below existing grades are recommended for the western portion of the site and within several isolated small canyon areas at the eastern portion of the site, as noted on the Geotechnical Map, Sheet 1. The western portion of the site (area of Cross-Sections 1-1'-1" through 5-5') is anticipated to require removal of non-structural fill and removal of approximately 5 feet of the underlying alluvium in order to expose competent material suitable for placement of structural fill. The smaller canyon areas at the eastern portion of the site are anticipated to require removal of localized alluvium (Cross-Sections 6-6', 7-7', and 8-8') and one area of relatively thin surficial landslide (Cross-Section 7-7') to expose competent bedrock. Estimated remedial grading profiles are depicted on the Cross-Sections, Sheets 2 & 3.
- As part of remedial grading, unsuitable soils should be removed to competent soils, temporarily stockpiled (where necessary) and replaced as properly compacted fill. Prior to placement as compacted fill, significant organic materials or other unsuitable materials shall be removed and properly exported offsite. The actual depths and lateral extents of grading should be determined by the geotechnical consultant, based on subsurface conditions encountered during grading. Local conditions may be encountered during excavation that could require additional removals beyond the minimums presented herein in order to obtain an acceptable subgrade preliminary, estimated removals are shown on the Geotechnical Map (Sheet 1), and as estimated remedial profiles on Cross-Sections (Sheets 2 & 3).
- Approximate existing subdrain locations, as presented in the as graded report for the golf course and adjacent residential tracts to the south (Geosoils, 1999), are presented on the Geotechnical Map, Sheet 1. The subdrains are anticipated to be encountered during the proposed rough grading operation and should be protected in place or extended to proper gravity flow outlets as necessary. Each subdrain will be handled on a case-by-case basis in order to maintain and/or improve the existing drainage system that serves both the subject site and the adjacent residential tracts to the south.
- The soils within the re-use areas (RUA 1 through 3) that are within the zone of influence of the proposed residential buildings shall be removed and replaced as properly compacted fill. The location of where these soils are placed and the thickness of the clean soil cap shall be determined by the project environmental consultant. The design cut slope potentially exposing RUA 3 (Cross-Section 1-1'-1") should be provided with a compacted replacement fill slope consisting of clean soil.
- Stabilization fill keyways should be constructed for design cut slopes that are not undercut by remedial grading. Locations of recommended stabilization fill keyways are shown on the Geotechnical Map, Sheet 1. Design cut lots, or lots with less than 5 feet of design fill, should be overexcavated a minimum of 5 feet below respective pad grades.
- Temporary backcuts during grading should be constructed at a maximum slope ratio of 1.6:1 (horizontal: vertical).

- Proposed fill slopes should be constructed at a slope ratio of 2:1 (horizontal to vertical) or flatter.
- Fills placed deeper than 40 feet below proposed grade should be compacted to an increased minimum relative compaction of 93 percent relative compaction. Fill should be moisture-conditioned to be between optimum moisture content and 2 percent over optimum moisture content, per ASTM D1557.
- Materials from the concrete disposal area shall be removed and replaced as general fill provided it is environmental suitable and crushed such that it is no larger than 8 inches in maximum dimension and well blended (i.e., no nesting and voids) into site fills. Any concrete material placed in MSE wall backfill areas should be crushed to meet gradation requirements of aggregate base per the latest edition of the Greenbook.
- Three lots at the eastern boundary of the site (Lots 12, 28 and 29) are located within the limits of the State of California Earthquake Fault Zone that encompasses a relatively wide area surrounding the offsite, active Unnamed West Coyote Hills Fault and requires geologic evaluation prior to construction of structures. During grading, the limits of the Earthquake Fault Zone should be staked by the project surveyor and the exposed bedrock carefully mapped by the project geologist. Proposed Lots 12, 28 and 29 are in design cut areas requiring finished grade to be lowered by approximately 5 to 15 feet. We recommend the design cut be performed first to expose the surrounding bedrock, and then the excavation of one backhoe trench per lot, generally perpendicular to the length of the Earthquake Fault Zone. Each trench should be excavated to a minimum of 5 feet deep in to ensure a vertical observation surface for detailed mapping. The City of La Habra's geotechnical reviewer should be notified to observe and confirm the geologic mapping. Due to previous grading that has occurred within the limits of the Earthquake Fault Zone, it's likely that soil horizons typically utilized for dating faults (to determine whether it's active or not) have already been removed. In the event that a fault splay which is deemed active is observed during grading within the limits of the Earthquake Fault Zone, a minimum 50-foot structural setback will be recommended.
- Where removals are performed to bedrock (i.e., no alluvium left in place), settlement monuments should be placed where fill depths are greater than approximately 40 feet. Fill depth is measured from removal bottom to finish grade. In addition, in areas where design grades increase 10 feet or more from existing grades and left in-place alluvium is approximately 20 feet or greater settlement monuments should be installed. Settlement monuments should be installed promptly after the completion of grading. Settlement monuments should be read by a licensed surveyor with an off-site benchmark. The survey readings should be obtained four times in the first two months, twice in the third month, and then once a month unless otherwise requested by the geotechnical consultant. Shallow footings and slab-on-grade foundations should be constructed after settlement monitoring data indicates future total settlements are within tolerable limits. Temporary surcharge loading may be used to decrease the settlement waiting period.
- Due to the height of the proposed fill slope and Mechanically Stabilized Earth (MSE) wall, additional geogrid reinforcement length (beyond local stability requirements to be determined by the MSE wall designer) will be required for adequate global stability factors of safety for the MSE wall located below Lots 241 through 245 (Cross-Section 13-13'). Preliminary slope stability analysis indicates at least 6 layers of geogrid reinforcement lengths of 60 feet, with an allowable strength (after appropriate reduction factors are applied by the manufacturer) of approximately 3.5 kips per foot, spaced at a maximum vertical spacing of 2 feet, are required for adequate global factors of safety. Further refinement of the design for required global stability geogrid will occur during preparation of the 40-scale grading plan and with input from the MSE wall designer.
- MSE walls and conventional retaining walls should be backfilled with relatively sandy soils. Portions of the onsite soils are too fine-grained and therefore are not suitable for MSE and conventional retaining wall backfill. Therefore, select grading of on-site sandy soils and/or import of sandy soils meeting project recommendations will be required. Sandy soils should comprise the geogrid zone required for

local stability as determined by the MSE wall designer. For conventional retaining walls, the sandy import zone should be a minimum of one-half the height of the retaining wall.

- Allowance in the earthwork volumes budget should be made for an estimated 5 to 10 percent reduction in volume of existing soils. It should be stressed that these values are only estimates and that an actual shrinkage factor would be extremely difficult to predetermine. Subsidence due to earthwork activities is expected to be on the order of 0.1 to 0.2 feet. These values are estimates only and exclude losses due to removal of vegetation or debris. The effective shrinkage of onsite soils will depend primarily on the type of compaction equipment and method of compaction used onsite by the contractor, and the accuracy of the topographic survey.
- Due to onsite expansive soils, mitigation measures such as stiffened and/or post-tensioned slab foundations are recommended. Pre-soaking of the subgrade soils will be required to reduce the potential impact of expansive soils.
- At completion of grading, additional testing will be required to confirm the characteristics of the fill materials including expansion potential and corrosivity characteristics. While LGC Geotechnical does not provide recommendations for corrosion, based on our experience typical mitigation measures include increased compressive strength for structural concrete, decreased water-to-cement ratio for structural concrete and/or encapsulation of post-tensioned cables. A corrosion consultant should provide recommendations for mitigation of corrosivity based on laboratory testing results of near-surface soils at completion of grading.
- Due to liquefaction potential in portions of the site, stiffened and/or post-tensioned slab foundations are recommended.
- Due to site soils being predominately compacted fill and bedrock consisting of fine grained soil interbeds (silts and clays), the hillside nature of the site, and the presence of potentially liquefiable alluvial soils, we strongly recommend against the intentional infiltration of storm water.
- After completion of site rough grading, graded slopes, existing perimeter landscaped slopes, existing and proposed new subdrain outlets, etc., will require regular maintenance in accordance with this and future geotechnical grading plan review reports.

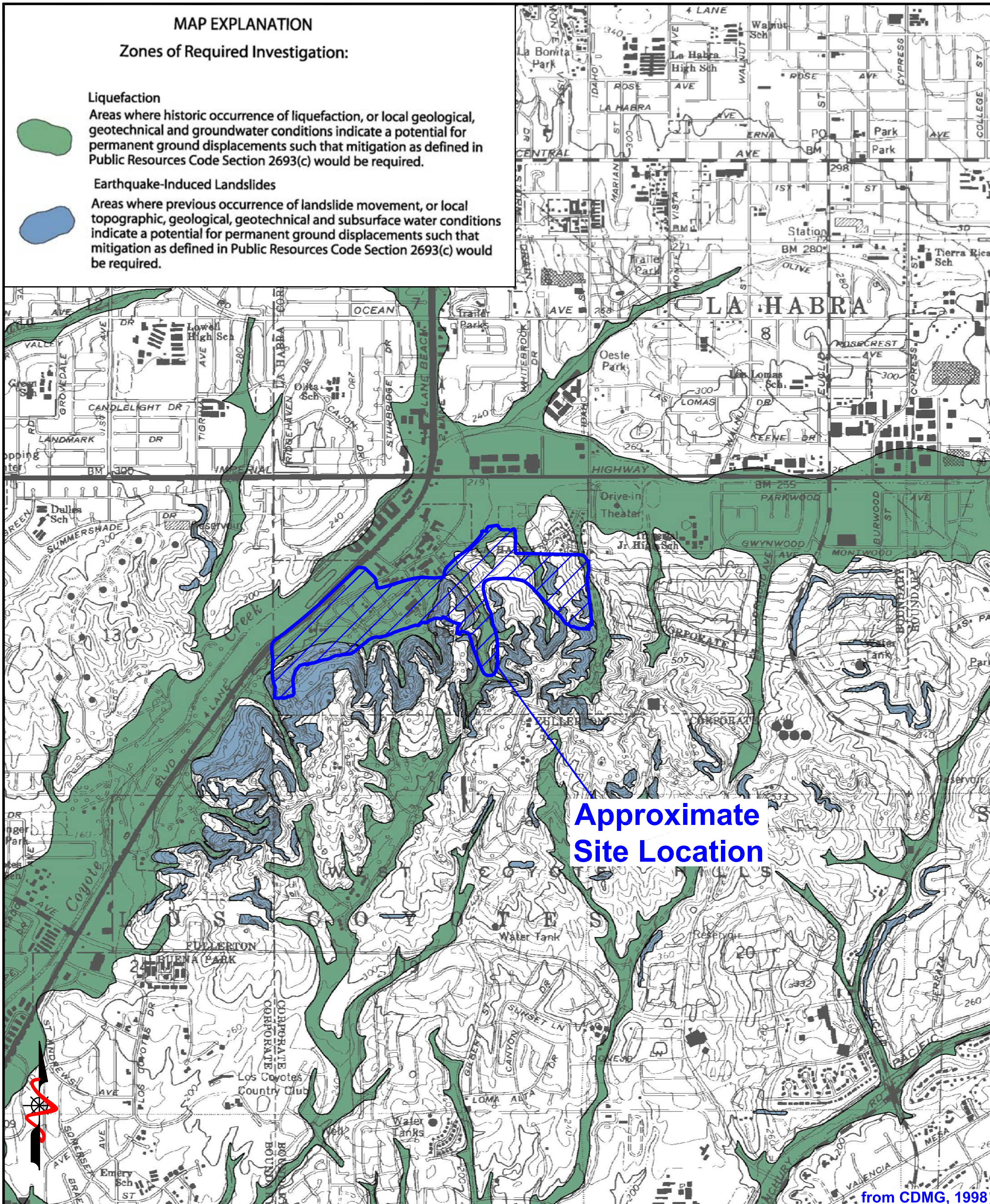
5.0 LIMITATIONS

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.

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 and conclusions 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.

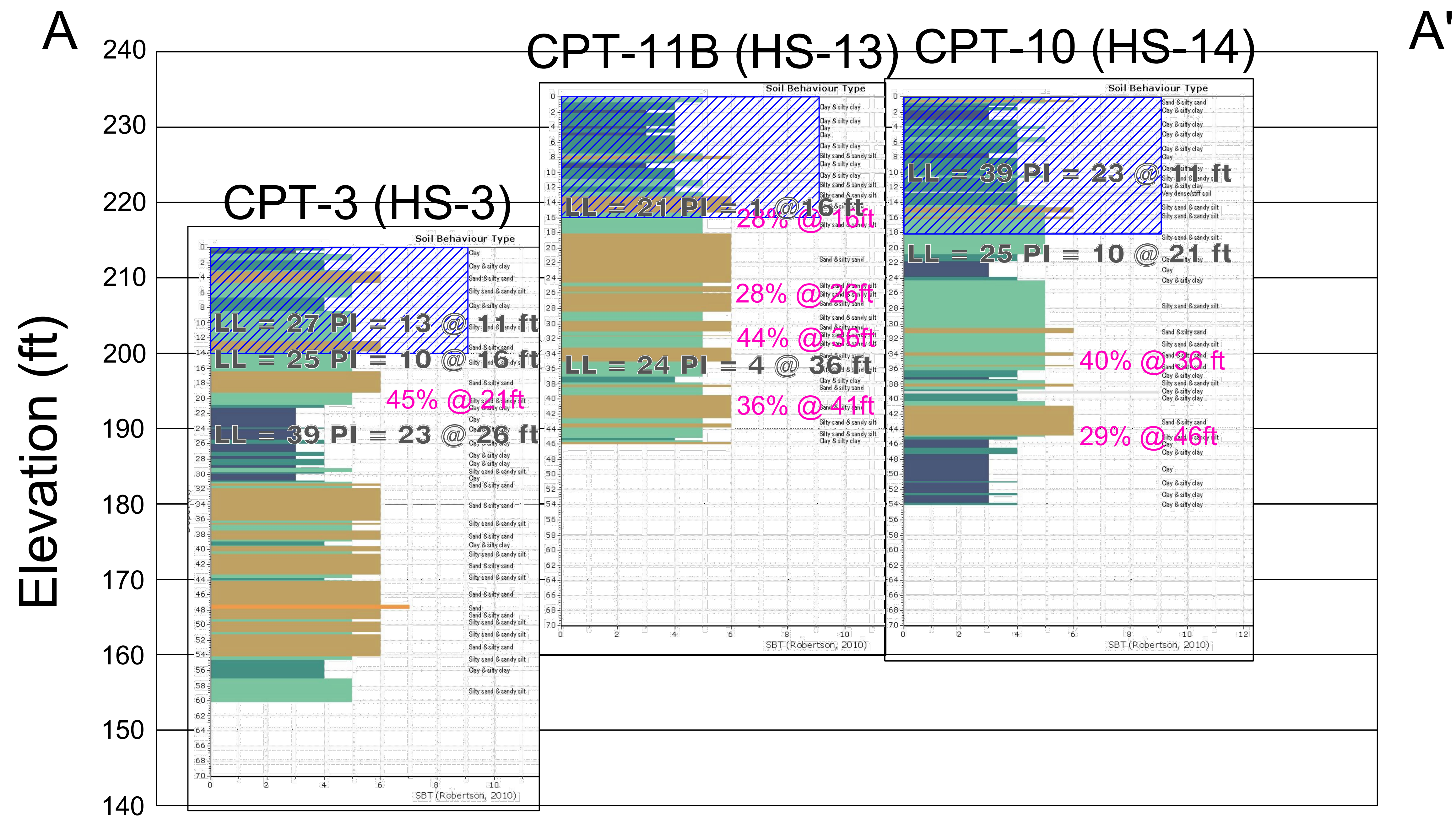
In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and modification.



LGC Geotechnical, Inc.

FIGURE 2
Seismic Induced Hazard Map

PROJECT NAME	Rancho La Habra - VTTM 17845
PROJECT NO.	14057-01
ENG. / GEOL.	DJB / KTM
SCALE	Not to Scale
DATE	April 2017



Horizontal Not to Scale

LEGEND

Soil to be Removed (Approximate)

Soil to be Removed and Recompacted (Approximate)

LL = 39 PI = 23 @ 36 ft — Atterberg Limits and Approximate Depth from Laboratory Testing

45% @ 21ft — Fines Content and Approximate Depth From Laboratory Testing

SCALE: 1"=16' *(Vertical Only)*

16' 0 16' 32'

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San Clemente, CA 92672
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FIGURE 3
CPT Fence Diagram A-A'

PROJECT NAME	*Rancho La Habra - VTTM 17845*
PROJECT NO.	*14057-01*
ENG. / GEOL.	*DJB / KTM*
SCALE	*1" = 16'*
DATE	*April 2017*

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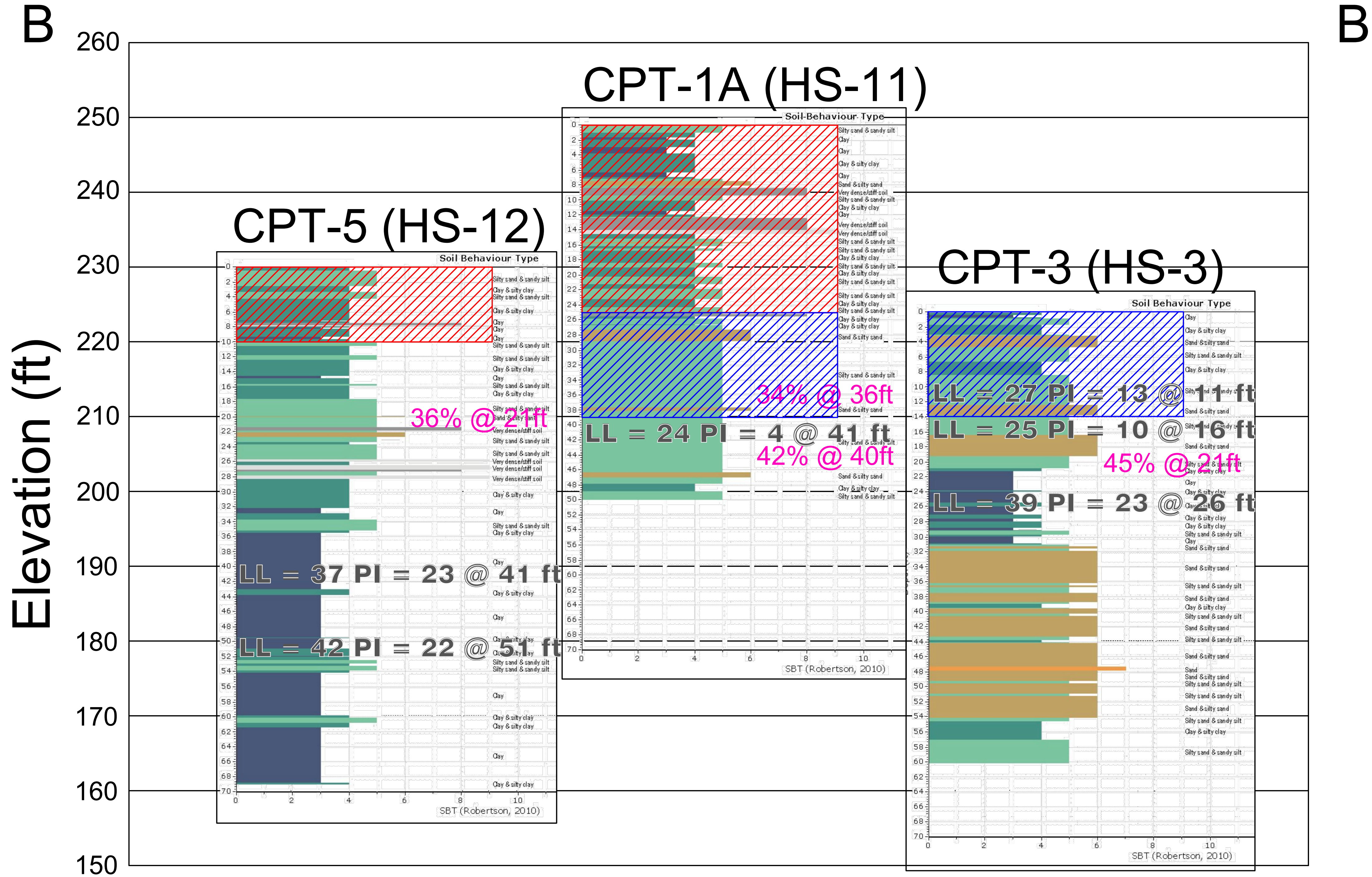

Horizontal Not to Scale

LEGEND

Soil to be Removed (Approximate)

Soil to be Removed and Recompacted (Approximate)

LL = 42 PI = 22 @ 51 ft — Atterberg Limits and Approximate Depth from Laboratory Testing

45% @ 21ft — Fines Content and Approximate Depth From Laboratory Testing

SCALE: 1"=16' *(Vertical Only)*

16' 0 16' 32'

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FIGURE 4
CPT Fence Diagram B-B'

PROJECT NAME	Rancho La Habra - VTTM 17845
PROJECT NO.	14057-01
ENG. / GEOL.	DJB / KTM
SCALE	1" = 16'
DATE	April 2017

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