April 10, 2020 Project No. 18117-01

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Subject: ***DRAFT: Preliminary Geotechnical Review of Proposed Shady View Residential Development, Tentative Tract 20317, City of Chino Hills, California***

In accordance with your request and authorization, LGC Geotechnical, Inc. has performed a geotechnical plan review of the proposed Shady View residential development, Tentative Tract No. 20317, located within the Abacherli Ranch at the southern terminus of Shady View Drive and Via La Cresta, west of the 71 Freeway, within the City of Chino Hills, California. This report presents a summary of our conclusions and preliminary recommendations relative to the proposed development of the site.

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,

LGC Geotechnical, Inc.

DRAFT DRAFT

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1.0 INTRODUCTION

1.1 Purpose and Scope of Services

This report presents the results of our geotechnical review of Tentative Tract No. 20317 and preliminary grading plans for proposed residential development of the approximately 130-acre site located at the southern terminus of Shady View Drive and Via La Cresta, west of the 71 Freeway, within the City of Chino Hills, San Bernardino County, California. (Site Location Map, Figure 1). The proposed grading plan by Hunsaker & Associates (Hunsaker, 2020) was utilized as a base map for our Geotechnical Map (Sheets 1 & 2).

The purpose of our study was to provide a preliminary geotechnical evaluation relative to the proposed residential development. As part of our scope of work, we have: 1) reviewed available geotechnical background information including in-house regional geologic maps and published geotechnical literature pertinent to the site (Appendix A); 2) performed a surface fault rupture study including excavation and detailed logging of seven fault trenches in accordance State of California guidelines for surface fault rupture evaluations, as reported in detail in the referenced report (LGC Geotechnical, 2020); 3) performed a subsurface geotechnical evaluation of the site consisting of the excavation of seven large diameter borings downhole-logged by a geologist, and excavation of eight exploratory trenches; 4) excavation of ten small-diameter borings, sampled and logged, including two used to perform preliminary field infiltration tests; 4) performed laboratory testing of select soil samples; and 5) prepared this preliminary geotechnical summary report presenting our findings, preliminary conclusions and recommendations for the development of the proposed project.

The findings and conclusions presented herein should be considered preliminary and will need to be updated as design documents and retaining wall plans are completed, and the overall assumptions must be confirmed during rough grading.

1.2 Project Description

Based on the rough grading plan (Hunsaker, 2020), the proposed residential development will consist of 159 single family lots for residential structures, a recreation center, interior streets, debris/detention basins, and other associated improvements. An aboveground bulk crude oil tank pad is proposed outside of the residential area at the northwest portion of the site, to be separated from the community by an ascending slope, an access road and a berm. Conventional retaining walls up to approximately 3 feet in height are planned throughout the site, and Mechanically Stabilized Earth (MSE) retaining walls up to approximately 30 feet in height in two locations at the eastern boundary, both located north and south of the area labeled "Not A Part". An MSE Wall up to 6 feet high is proposed along a portion of the road along the southwestern boundary of the development site. A new access road is planned to provide access to the area labeled "not a part" from an end of street within the development. The site includes approximately 72 acres of natural open space located at the southwest portion of the roughly rectangular shaped site. The rough grading plan proposes design cut slopes of up to approximately 95 feet, and fill slopes up to approximately 65 feet in height that include MSE retaining walls. Planned cuts and fills to reach design grade are anticipated to be on the order of approximately 45 feet and 60 feet, respectively, not considering remedial measures.

Based on preliminary review of proposed storm drain and water quality management plan, a series of four detention basins at the southwestern boundary of the site will collect runoff from the surrounding hillside to the west of the site. Each basin has a storm drain inlet pipe that will ultimately outlet through one of the development's two storm drain outlet areas. One outlet area is located at the northeastern corner of the site and the other is mid site along the eastern border (the approximate area currently draining the main active drainage at the site). It is our understanding that modular wetlands or similar system will be utilized to improve water quality rather than intentional infiltration to site soils (Hunsaker, 2020a).

The proposed building structures are anticipated to be relatively light-weight at-grade structures with maximum column and wall loads of approximately 30 kips and 2 kips per linear foot, respectively.

The recommendations given in this report are based upon the estimated structural loading, grading and layout 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 when they become available, in order to either confirm or modify the recommendations provided herein.

1.3 Existing Conditions

The approximately 130-acre property is located west of the 71 Freeway and south of the existing terminus of Shady View Drive and Via La Cresta streets, within the existing Butterfield Ranch residential development. Refer to the Site Location Map, Figure 1 (page 6). The site is roughly rectangular shaped with a square-shaped cut-out that is "Not a Part" of the proposed development. Originally named Abacherli Ranch, the site is accessed by an unpaved road from Mystic Canyon Drive located to the west about a third of a mile from the site. The site currently consists of undeveloped land with areas of various land uses including oil extraction collection pipelines and a set of three bulk crude oil storage tanks near the center of the proposed development site. Other uses of the generally vacant land include localized areas of equipment storage, a beehive farm, split wood storage, and soil piles. The gated access road from Mystic Canyon Drive passes across the site and splits to provide both access to the square-shaped parcel encompassed by the subject site, and access to the upper reaches of the hills to the west of the site, not open to the public. The access roads are alternately paved, and dirt topped with gravel within the site.

The subject site is surrounded by the existing Butterfield Ranch residential development to the north, and the square-shaped "not a part" area (with a prominent water tank/ cell tower), a sliver of open space, and the 71 Freeway to the east. The site abuts open space to the south and west that also includes the oil extraction facilities at scattered locations throughout the nearby hills, and an evaporation pond. The hills to the west of the site are a part of the Mahala Oil Field; Chino Hills State Park is located further west. Various pipelines on the order of 4 inches in diameter reportedly collect extracted crude oil from the facilities in the hills (and in some cases groundwater) and pipe them through the central main canyon that transects the site from west to east, to the existing set of three oil storage tanks located in the main canyon within the limits of the proposed residential development. The pipelines are visible on the surface or buried just below the surface.

Topographically the site consists of a large, slightly undulatory hillside at the southwest portion of the site and a series of low rolling, east-west trending canyons and ridges at the northeast portion of the site. The current active drainage for the local Chino Hills located west of the site, runs west to east through the upper-middle portion of the site. Within the proposed southern portion of the development area, smaller canyons between low ridges drain west to east on site. Overall elevations range from approximately 560 feet above mean sea level (msl) to a maximum of 1,080 feet at the top of the large hillside that ascends above the proposed development area to the southwest. Vegetation onsite consists of moderate to thick low bushes and few scattered trees.

1.4 Subsurface Geotechnical Evaluation by LGC Geotechnical

In order to gather information regarding potential faulting within the project site. LGC Geotechnical had previously excavated three fault trenches (FT-1 through FT-4) in 2014 and excavated an additional three fault trenches (FT-5 through FT-7) recently. Earth Consultants International (ECI) was subcontracted by LGC Geotechnical to provide peer review of all fault trenches and expertise in soil age dating analysis. LGC Geotechnical also sampled and downhole logged a total of seven large-diameter borings up to 60 feet deep, in order to supplement the fault trenching information and eventually to establish remedial measures for the proposed residential development. Geologic findings conclusions and recommendations regarding our evaluation of potential Holocene-active faults surface hazard relative to the proposed development can be found in the referenced report by LGC Geotechnical, 2020. The locations of fault trenches are shown on the Geotechnical Map (Sheets 1 & 2). However, the logs for LGC Geotechnical fault trenches FT-1 through FT-7, and ECI fault trenches ECI-FT-1 through ECI-FT-6 are presented in the referenced report (LGC Geotechnical, 2020).

For site evaluation of engineering characteristics, both large diameter and small diameter borings were excavated, along with backhoe test pits are various locations. Locations of subsurface excavation performed as part of this evaluation are presented on the Geotechnical Maps (Sheets 1 & 2.) Large diameter borings were sampled and downhole logged for structural geologic information. Small diameter borings were logged and sampled, and selected samples tested for laboratory characteristics as reported herein. Selected small diameter borings were tested for preliminary infiltration potential. Large and small diameter boring, test pit, and infiltration testing logs are presented in Appendix B.

1.5 Field Infiltration Testing

Two field percolation tests (I-1 and I-2) were performed to approximate depths of approximately 15 and 20 feet, respectively, below existing grade within native soils. The approximate locations are shown on Geotechnical Map. Field percolation testing was performed in general accordance with the guidelines set forth by the County of San Bernardino (2013). For the falling head test, a 2-inch-diameter slotted PVC pipe was placed in the boreholes and the annulus was backfilled with gravel to the surface including placement of approximately 2 inches of gravel at the bottom of the borehole. The infiltration wells were pre-soaked per the County guidelines. Based on the County of San Bernardino methodology, the infiltration rates measured summarized in Table 1 have been normalized from the threedimensional flow that occurs within the field test to a one-dimensional flow out of the bottom of the boring only. Infiltration tests are performed using relatively clean water free of particulates, silt, etc. The observed infiltration rates provided in Table 1 do not include a factor of safety. Refer to the discussion provided in Section 4.9.

TABLE 1

Summary of Field Infiltration Testing

Infiltration Test Location	Observed Infiltration Rate* (inch/hr.)
I-1	13.9
I-2	16.7

*Does not include any Factor of Safety

1.6 Laboratory Testing

Representative bulk and driven samples were retained for laboratory testing during our subsurface field evaluation. Laboratory testing included in-situ moisture content and in-situ dry density, gradation, Atterberg Limits, expansion index, direct shear, residual torsional shear, consolidation, laboratory compaction and corrosion (soluble 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 90 pounds per cubic foot (pcf) to 129 pcf, with an average of 112 pcf. Field moisture contents ranged from approximately 3 percent to 26 percent, with an average of 9 percent.
- Three gradation tests were performed and indicated fines content (passing No. 200 sieve) ranging from approximately 35 to 54 percent. According to the Unified Soils Classification System (USCS), two of the tested samples are classified as "coarse-grained" soil.
- Four Atterberg Limit (liquid limit and plastic limit) tests indicated Plasticity Index values ranging from 11 to 45.
- Two direct shear tests were performed on select driven samples. The plots are provided in Appendix C.
- A drained residual torsional ring shear test was performed on a grab sample of landslide rupture surface (clay). The plot is provided in Appendix C.
- A consolidation test was performed. The deformation versus vertical stress plot is provided in Appendix C.
- Four Expansion Index (EI) tests were performed. Results indicate EI values ranging from 0 to 86, corresponding to "Very Low" and "Medium" expansion potential.
- Laboratory compaction testing of a near-surface bulk sample indicated maximum dry density values of 123.5 pounds per cubic foot (pcf) and an optimum moisture content of

11.5 percent.

- Corrosion testing indicated soluble sulfate content less than approximately 0.01 percent, chloride content of approximately 64 parts per million (ppm), pH value of 6.92, and minimum resistivity of 2,114 ohm-cm.

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

LGC Geotechnical, Inc.

FIGURE 1
Site Location Map

PROJECT NAME	Trumark - Shady View
PROJECT NO.	18117-01
ENG.	DJB / KTM
SCALE	Not to Scale
DATE	April 2020

2.0 GEOTECHNICAL CONDITIONS

2.1 Regional Geology

Regionally the subject site is located northeast of the Santa Ana Mountains which are part of the Peninsular Ranges geomorphic province. The Chino Hills are considered to be a part of the Puente Hills which lie at the eastern margin of the Los Angeles Basin. Several regional faults have influenced formation and erosion of the mountains and hills over time including the Elsinore Fault Zone that splits into the Whittier Fault and the Chino Fault southeast of the subject site

The Santa Ana River passes to the southwest of the site within an incised drainage that has abandoned a series of stream terraces (older alluvium) at various higher elevations that partially mantle the lower portion of the Chino Hills. The Prado Dam is sited several miles southeast of the site where it was constructed across the Santa Ana River.

More specifically, the site is predominately underlain by folded or overturned and locally faulted bedrock units with minor amounts of older alluvium along the slopes and ridges at various elevations, and young alluvium in the existing drainages.

2.2 Site-Specific Geology

Based on the Geologic Map of the 7.5-minute Prado Dam Quadrangle (Dibblee, 2001) and geologic field mapping, the subject site is underlain by Quaternary Alluvial Deposits, Quaternary Old Alluvial Fan Deposits, and Tertiary Puente Formation, Sycamore Canyon Member. Notably the Regional Geologic Map, Figure 2, presents the previously used nomenclature for the Puente Formation, ("Sycamore Canyon Formation"), updated with the current evaluation. The geologic units are summarized below from youngest to oldest and their approximate lateral limits are depicted on the Geologic Map (Sheets 1 & 2).

2.2.1 Quaternary Alluvial Deposits (Map Symbol – Qal)

The Quaternary Alluvial Deposits are located in the active drainages and valleys which occupy the lowest elevations at the subject site. Typically, these unconsolidated deposits vary in thickness from a few feet to greater than 50 feet. The material is generally light orangish brown to moderate brown, silty sand with gravel and cobbles, variable moisture and density.

2.2.2 Quaternary Older Alluvium(Map Symbol – Qoa)

Quaternary Older Alluvium was encountered in localized locations, elevated above the active drainages at the southwest portion of site, becoming deeper and more widely distributed to the east and into the valley east of the 71 Freeway. The material is typically light reddish-brown clayey silt to silty sand with gravel and cobbles with interfingered zones of clayey sandy gravel and coarse sand, reddish yellow to strong brown, slightly moist, dense to very dense; indurated, faintly stratified, with some buried paleosol horizons. Alluvial deposits are interfingered with mudflow deposits consisting of clayey silt with fine sand, gravel, and cobbles, dark yellowish brown, slightly moist, stiff to very stiff, lacks structure, indurated.

Based on soil analysis by ECI (presented in LGC Geotechnical, 2020), the Quaternary Older Alluvium observed in fault trenches is approximately 200,000 to 300,000 years old.

2.2.3 Tertiary Puente Formation, Sycamore Canyon Member (Map Symbol – Tpsc)

The Tertiary Puente Formation is a Late Miocene marine deposit that consists of four members that have a total thickness of up to about 5,400 feet. From oldest to youngest, the members are the La Vida Member, the Soquel Member, the Yorba Member, and the Sycamore Canyon Member. The youngest Sycamore Canyon Member is the bedrock unit at the subject site.

The Sycamore Canyon Member of the Puente Formation (Map Symbol – Tpsc) encountered onsite consists of thin to very thick interbedded conglomeratic sandstone, and sandstone, sandy siltstone and siltstone, light greenish gray to yellowish red to very pale brown, slightly moist, dense to very dense. Conglomerate beds were observed to be cemented, resistant to weathering, with sub-rounded to subangular, granitic and metamorphic gravel and cobble clasts.

2.3 Geologic Structure

The geologic structure of the bedrock unit at the site is generally controlled by the presence of the Chino Fault. As depicted in Cross Section C-C', the Chino Fault is a right-lateral strike slip fault with a component of reverse dip-slip that is generally the reason for the escarpment/hillside at the southwest portion of the site. The Mahala Anticline that has been mapped within the Chino Hills to the west of the site, is sub-parallel with the fault. The Mahala Anticline is the source of the oil being extracted in the hills west of the site, the Mahala Oil Field.

The bedding of the Puente Formation has been very consistent where observed at the site. Strike of bedding is generally northwest, similar to the trend of the Chino Fault, and dips range between 60 and 75 degrees southwest (overturned), in accordance with regional geologic mapping (Dibblee, 2001 & Morton, 2004) and observations on-site; the bedding likely mirrors the orientation of the fault itself. Refer to Figure 2, Regional Geologic Map (rear of text). Proof in support of consistent, overturned bedding on site was observed during logging of a fault trench (LGC Geotechnical, 2020) where a contact between two materials showed rip-up clasts of the finer materials caught within the coarser material but positioned below the fine-grained material. Using the rule of "original horizontal deposition" of bedding, the higher-energy depositional environment of the coarse-grained materials ripped up materials that were already deposited within the layer below; therefore, the finer material was deposited first, the higher-energy coarse grained conglomerate second. The beds became overturned by regional structural forces likely related to the presence of the Chino Fault.

The geomorphology of the site offers some clues to the geologic structure within the large hillside at the west/central portion of the site. The hillside represents the escarpment of the reverse dip-slip portion of fault movement (the upthrown "hanging wall"). Millenia of erosion from the hillside escarpment area during active faulting in the right-lateral strike slip sense of movement created a series of partially developed "beheaded" canyons, saddles, lineaments/breaks in slope, as discussed in detail as part of the photo-lineament studies presented in Treiman, 2002 & ECI, 2008.

Erosion of the bedrock surface on the footwall side of the fault and subsequent deposition of alluvial terrace deposits derived from the Santa Ana River watershed created the angular unconformity between bedrock and older alluvium (terrace deposits) observed at various elevations at the site.

Bedrock geologic structure is generally overturned, steeply into slope, thin to very thick bedding with few very thin clay beds. Sandy siltstone beds showed more jointing and shearing that the conglomerate beds that were observed to be more resistant to weathering and have some weak cementation. Joints and minor internal shears were more prominent within the fine-grained (siltstone) materials at the site than the cobbly conglomerate materials. Calcium carbonate deposits, manganese oxide, and iron oxide staining was common in bedrock materials.

More recent landslides have mantled the central hillside area of the site. Based on downhole-logging of the borings, the basal rupture surface is relatively shallow, clay-lined feature that overrides older alluvium that is perched on the hillside, as depicted in Cross Sections C-C' and D-D' (In Pocket). Colluvium and slope wash of unknown thickness are typical in the hillside, especially around the edges of the identified landslide.

2.4 __Groundwater__

During our subsurface field evaluation, no free groundwater was encountered to the maximum explored depth of approximately 60 feet. The active drainage that runs west to east on site appears to be an ephemeral drainage with intermittent water flow based on seasonal conditions. Based on percent moisture of samples collected at depth within the center of the active wash (HS-1 and HS-2), no permanent water table was encountered in the drainage during site drilling.

Seasonal fluctuations of groundwater elevations should be expected over time. In general, ground-water 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 __Faulting and Seismic Hazards__

The trace of the *Holocene-active* Chino Fault has been identified to trend a combination of north and northwest across portions of the subject site, as approximately depicted on the Geologic Map (Sheets 1 & 2). The fault is located within a State of California Earthquake Fault Zone in accordance with the Alquist-Priolo Earthquake Fault Zoning Act (CDMG, 2003). The overall geometry of the fault is estimated to strike about 40 degrees to the west, dipping 70 degrees west at depth based on petroleum drilling data, shallower near the surface (Treiman, 2002). An evaluation of potential for surface rupture and recommendations for setbacks for habitable structures were presented in the referenced fault evaluation report (LGC Geotechnical, 2020). The recommended setback for habitable structures is presented on the Geotechnical Map (Sheets 1 & 2). See LGC Geotechnical, 2020, for complete details regarding the onsite faulting.

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 and shallow ground rupture, soil liquefaction, and dynamic settlement. 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. The nearby major active faults that could produce these secondary effects include the Chino Fault, Whittier-Elsinore Fault Zone, and San Andreas Faults, among others. A discussion of these secondary effects 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 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. Studies indicate that saturated, loose to medium dense, 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. 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.

Based on our review of the State of California Seismic Hazard Zone Map for the Prado Dam 7.5 Minute Quadrangle, the site is located within an “Area Not Evaluated” for potential for liquefaction (CDMG, 2001). At completion of rough grading the development will be underlain by compacted fill over sufficiently dense alluvium or formational bedrock, therefore the potential for liquefaction is considered to be low.

2.5.2 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 low potential for liquefaction, the potential for lateral spreading is also considered to be low.

2.6 Seismic Design Parameters

The site seismic characteristics were evaluated per the guidelines set forth in Chapter 16, Section 1613 of the 2019 California Building Code (CBC) and applicable portions of ASCE 7-16 which has been adopted by the CBC. **Please note that the following seismic parameters are only applicable for code-based acceleration response spectra and are not applicable for where site-specific ground motion procedures are required by ASCE 7-16.** Representative site coordinates of latitude 33.9182 degrees north and longitude -117.6556 degrees west were utilized in our analyses. Please note that these coordinates are considered representative of the site for preliminary planning purposes, however their applicability must be verified with respect to a desired specific location within the site. 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 C are provided in Table 2. The structural designer should contact the geotechnical consultant if structural conditions (e.g., number of stories, seismically isolated structures, etc.) require site-specific ground motions.

TABLE 2

Seismic Design Parameters

Selected Parameters from 2019 CBC, Section 1613 - Earthquake Loads	Seismic Design Values	Notes/Exceptions
Distance to applicable faults classifies the site as a "Near-Fault" site.		Section 11.4.1 of ASCE 7
Site Class	C	Chapter 20 of ASCE 7
Ss (Risk-Targeted Spectral Acceleration for Short Periods)	2.008g	From SEAOC, 2020
S_1 (Risk-Targeted Spectral Accelerations for 1-Second Periods)	0.712g	From SEAOC, 2020
F_a (per Table 1613.2.3(1))	1.2	For Simplified Design Procedure of Section 12.14 of ASCE 7, F_a shall be taken as 1.4 (Section 12.14.8.1)
F_v (per Table 1613.2.3(2))	1.4	-
S_{MS} for Site Class C [Note: $S_{MS} = F_a S_S$]	2.410g	-
S_{M1} for Site Class C [Note: $S_{M1} = F_v S_1$]	0.997g	-
S_{DS} for Site Class C [Note: $S_{DS} = (^2/_3)\ S_{MS}$]	1.607g	-
S_{D1} for Site Class C [Note: $S_{D1} = (^2/_3)\ S_{M1}$]	0.664g	-

C_{RS} (Mapped Risk Coefficient at 0.2 sec)	0.911	ASCE 7 Chapter 22
C_{R1} (Mapped Risk Coefficient at 1 sec)	0.909	ASCE 7 Chapter 22

Section 1803.5.12 of the 2019 CBC (per Section 11.8.3 of ASCE 7) states that the maximum considered earthquake geometric mean (MCE_G) Peak Ground Acceleration (PGA) should be used for liquefaction potential. The PGA_M for the site is equal to 1.019g (SEAOC, 2020). The design PGA may be taken as 0.679g (2/3 of PGA_M).

A deaggregation of the PGA based on a 2,475-year average return period (MCE) indicates that an earthquake magnitude of 6.75 at a distance of approximately 5.47 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 (Design Earthquake) indicates that an earthquake magnitude of 6.66 at a distance of approximately 9.85 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, published shear strength data (CDMG, 2000) and our professional judgment. The soil shear strength for the landslide rupture surface is based on results of a residual torsional ring shear test from landslide rupture surface clay materials obtained during downhole logging of B-7. Where applicable, soil shear strength parameters were increased for seismic loading conditions. Laboratory test results are provided in Appendix C.

TABLE 3

Soil Shear Strength Parameters for Slope Stability Analysis

Soil Type	ϕ (Degrees)	Cohesion (psf)
Compacted Fill (Afc)	30	200
Landslide Basal Rupture Surface	14	0
Bedrock (Tpsc)	26	300
Alluvium (Qal)	28	100
Older Alluvium (Qoa)	26	200
Landslide Material (Qls)	26	200

2.8 *Global Slope Stability Analyses*

Global slope stability analyses were performed on critical cross sections (A-A' through D-D') 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. Where applicable, slope stability analysis was performed for static and seismic loading conditions. A minimum factor of safety of 1.5 is typically required for static loading conditions for the proposed development. Seismic slope stability analysis was performed based on a horizontal seismic coefficient (Kh) of 0.20 with a minimum required factor of safety of 1.1. For bedding planes less than 12 degrees from the horizontal, pseudo-static (seismic) slope stability was not performed.

For the up to approximately 30-foot high MSE walls located in the eastern portion of site, an increase in geogrid lengths on the order 5 to 10 feet beyond the requirements for local stability should be anticipated at this time. Once the proposed MSE Retaining Wall has been designed (including geogrid type, length, spacing, etc.), the global stability of the slopes shall be re-evaluated.

Slope stability analysis is provided in Appendix D.

3.0 CONCLUSIONS

Based on the results of our subsurface evaluation and geotechnical review of the preliminary grading plan for Tentative Tract No. 20317, it is our professional opinion that the proposed development is feasible from a geotechnical standpoint and will not adversely impact adjacent properties, 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:

- If constructed in general accordance with the geotechnical recommendations presented here in and in future geotechnical reports, it is our opinion that the proposed development is feasible from a geotechnical standpoint, and it is not anticipated to significantly impact surrounding development.
- The bedrock geologic unit mapped on the site is the Tertiary-aged Puente Formation, Sycamore Canyon Member. Quaternary Older alluvial deposits cap the low ridgelines at the southwest portion of the site and becomes thicker going east. More recent Quaternary Alluvial deposits are mapped within the active drainages, and Quaternary Landslides mantle the hillside southwest of the proposed residential development.
- A portion of the site is located within a State of California Earthquake Fault Zone (CDMG, 2003).-A thorough evaluation of potential for surface rupture and setbacks for habitable structures were presented in the referenced fault evaluation report (LGC Geotechnical, 2020). The findings and conclusions of the report were incorporated into the preliminary grading plan review herein.
- Based on our review of the State of California Seismic Hazard Zones for the Prado Dam 7.5 Minute Quadrangle, the site is within an Area Not Evaluated as having a potential for earthquake induced landslide. This potential for landslide impacting the development will be mitigated with design cut and fill grading and remedial grading measures presented herein.
- Based on our review of the State of California Seismic Hazard Zones for the Prado Dam 7.5 Minute Quadrangle, the site is located within an Area Not Evaluated for potential for liquefaction. At completion of rough grading the development will be underlain by compacted fill over sufficiently dense alluvium or formational bedrock, and the potential for liquefaction is considered to be low at completion of rough grading.
- 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.
- Proposed fill depths are estimated to be up to approximately 80 feet at completion of remedial grading and design fill placement. An increase of the minimum relative compaction criteria will be required for fill placed at depths greater than approximately 40 feet below proposed finish grade. Proposed fill depths greater than approximately 40 feet require surface settlement monitoring be performed after grading is completed to ensure long-term fill settlement is within tolerable limits prior to commencement of building construction.
- No significant groundwater was encountered during our subsurface field evaluation. The active drainage located mid-site may have interim (ephemeral) flow at the surface or at depth.
- Mitigation of offsite landslides to the southwest of the site will require significant grading as shown on maps and cross sections. A landslide up to 40 feet thick mantles the hillside and is recommended to be temporarily removed and recompacted.

- Global slope stability analysis for the proposed development indicates a safety of at least 1.5 and 1.1 for static and pseudo-static (seismic) loading conditions, respectively.
- Native slopes southwest of the central portion of the site have potential for erosion, sloughing, surficial instabilities, etc., which may be exacerbated by heavy rains or shaking events, etc. Repair of offsite surficial landslide failures may be required. Maintenance in the form of debris removal within the proposed debris basins will be required on a frequent basis.
- Based on the results of preliminary laboratory testing site soils are anticipated to have "very low" to "medium" expansion potential. Final expansion potential will be determined at the completion of rough grading. 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 adjacent to slopes will be impacted by slope creep.
- 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 portions of the site contain soils that are suitable for backfill of MSE and conventional retaining walls. However, select grading and/or stockpiling monitored by LGC Geotechnical will be required.
- For the approximately 30-foot high MSE retaining walls located in the eastern portion of site, an increase in geogrid lengths on the order 5 to 10 feet beyond the requirements for local stability should be anticipated at this time. Once the proposed MSE retaining wall has been designed (including geogrid type, length, spacing, etc.), the global stability of the retaining walls and slopes should be re-evaluated.
- Based on the results of infiltration testing and due to the hillside nature of the site, presence of fine-grained soils, it is our professional opinion that purposeful infiltration at the subject site is infeasible from a geotechnical and regulatory standpoint and therefore should not be performed. It is our understanding that no purposeful infiltration is proposed due to the hillside nature of the site and water quality will be achieved by installation of modular wetlands or similar (Hunsaker, 2020a).
- Additional borings may be necessary in order to confirm assumption or finalize design of MSE Walls. especially at the southeastern MSE Wall where potentially deep alluvial removals required for preparation of wall subgrade are constrained by the property boundary.

4.0 PRELIMINARY 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.

It should be noted that the following geotechnical recommendations are intended to provide sufficient information to develop the site in general accordance with the 2019 CBC requirements. With regard to the possible occurrence of potentially catastrophic geotechnical hazards such as seismic shaking, earthquake-induced landslides, liquefaction, etc. the following geotechnical recommendations should provide adequate protection for the proposed development to the extent required to reduce seismic risk to an "acceptable level." The "acceptable level" of risk is defined by the California Code of Regulations as "that level that provides reasonable protection of the public safety, though it does not necessarily ensure continued structural integrity and functionality of the project" [Section 3721(a)]. Therefore, repair and remedial work of the proposed improvements may be required after a significant seismic event. With regards to the potential for less significant geologic hazards to the proposed development such as expansive soils, fill settlement, slope creep, groundwater seepage, etc., the recommendations contained herein are intended as a reasonable protection against potential damaging effects. It should be understood, however, that our recommendations are intended to maintain the structural integrity of the proposed development and structures given the site geotechnical conditions, but cannot preclude the potential for some cosmetic distress or nuisance issues to develop as a result of the site geotechnical conditions.

The geotechnical recommendations contained herein must be confirmed to be suitable or modified based on the actual as-graded conditions.

4.1 Site Earthwork

We anticipate that earthwork at the site will consist of rough grading including design cuts and fills, excavation of one large buttress keyway along the western boundary of the edge of the site, remedial grading of near surface soils, installation of a subdrain system, and construction of Mechanically Stabilized Earth (MSE) walls and conventional retaining walls. In general, rough grading will be followed by the installation of utilities and foundations, and asphalt paving of interior streets. We recommend that earthwork onsite be performed in accordance with the following recommendations, the City of Chino Hills/2019 CBC requirements, and the General Earthwork and Grading Specifications for Rough Grading included in Appendix E. In case of conflict, the following recommendations shall supersede all previous recommendations and those included as part of Appendix E. The following recommendations should be considered preliminary and may be revised by the geotechnical consultant based on the actual conditions encountered during site grading.

4.1.1 Site Preparation

Prior to grading of areas to receive structural fill or engineered structures, the areas should be cleared of surface obstructions, vegetation, and debris. Vegetation and debris should be removed and properly disposed of offsite. Holes resulting from the removal of buried obstructions, which extend below proposed removal bottoms, should be replaced with properly compacted fill material.

4.1.2 *Buttress Keyway Excavation*

The Geotechnical Map (Sheets 1 & 2) depicts the approximate location and depth of the buttress keyways along the western/central boundary of the proposed residential development. The buttress keyway is shown in cross-sectional view on Cross Sections B-B', C-C', and D-D' (Sheets 3 & 4). The existing landslide depicted on Cross Sections C-C' and D-D' is recommended to be temporarily removed and recompacted with the proposed backcut excavation.

Subdrains will be required for the buttress keyway at the lowest elevations available and along the buttress backcut at regular intervals. Refer to the General Earthwork & Grading Specifications (Appendix E) for details regarding keyway construction and subdrains.

4.1.3 *Stabilization Fill Keyways*

Numerous proposed cut slopes are recommended to be replaced with stabilization fill keyways, as depicted on the maps and cross sections as keyways between 15 to 25 feet wide. Stabilization fill keyways should be constructed in general accordance with the recommendations of Appendix E. Keyways should be tilted into-slope and fill placed in thin lifts, with subdrains constructed along backcuts within areas that have at least 10 feet of compacted fill above the subdrain.

4.1.4 *Removals and Over-Excavation*

Due to presence of variable near surface bedrock at completion of design cut grading, we recommend design cut pads or cut/fill transition pads be over-excavated a minimum of 5 feet below finish pad grade, or a minimum of 2 feet below planned footings, whichever is greater. Additionally, selected areas of design cut where the cut is anticipated to exposed near-vertical bedding of the Puente Formation, are recommended to be over-excavated a minimum of 7 or 10 feet below pad grades depending on the anticipated weathering of the bedrock, potential differential settlement, or potential differential heave/uplift due to localized expansion of interbedded silty or clayey bedrock. Approximate locations of the pads anticipated to require over-excavation greater than 5 feet are noted on the Geotechnical Map.

In all design fill areas, we recommend the upper 5 feet from existing grade as a minimum be removed and recompacted as fill. Deeper removals are recommended for potentially compressible alluvium and thick deposits of colluvium mantling slopes within the proposed development. Design cut lots and sliver fill slopes at the daylight edges of the development should be provided with a daylight edge keyway in order to ensure the lot boundary is supported at a 1:1 (horizontal to vertical) inclination to competent materials. The actual depth and lateral extents of removals and lot over-excavation bottoms should be determined by the geotechnical consultant, based on subsurface conditions encountered during grading.

Removal bottoms and lot over-excavation bottoms must be accepted and mapped by the geotechnical consultant prior to subsequent fill placement.

4.1.5 Geologic Mapping

Removals, backcuts, and keyway excavations must be geologically mapped by the geotechnical consultant during earthwork construction to confirm the anticipated conditions. If unanticipated adverse joints, fractures, and/or bedding are exposed, additional analysis and/or remediation measures may be required. The grading contractor must trim the backcuts with a slope board to remove loose material to allow for confirmation geologic mapping. Updated and/or revised geotechnical recommendations may be required based on observed conditions.

4.1.6 Material for Fill

From a geotechnical perspective, the onsite soils are generally considered suitable for use as general compacted fill, provided they are relatively free of organic materials and construction debris. Based on preliminary results, it would appear that a portion of the onsite native soils may be considered generally suitable for placement within Mechanically Stabilized Earth (MSE) wall structural backfill zone. This preliminary finding will need to be further evaluated. Refer to Section 4.1.7, Select Material for MSE Wall Backfill, for required fill materials within the Backfill Zone.

Any encountered oversized material (material larger than 8 inches in maximum dimension) must be appropriately handled as outlined in Appendix E.

Conventional (masonry) 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 in maximum dimension. The site contains soils that are not suitable for retaining wall backfill due to their clay content and expansion potential; therefore, select grading (and potentially stockpiling) or import will be required by the contractor for obtaining suitable backfill soil.

If any import is required for general fill (i.e., not the select fill for MSE/retaining wall backfill), it should consist of clean, relatively granular soils of Low expansion potential (expansion index 50 or less based on ASTM D4829) and no particles larger than 3 inches in greatest dimension.

Aggregate base (crushed aggregate base or crushed miscellaneous base) should conform to the latest 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.

4.1.7 Select Material for MSE Wall Backfill

The proposed MSE Walls will require select backfill materials to be placed within the geogrid "reinforced" zone. The select fill shall consist of sandy materials with a maximum of 35 percent passing the No. 200 sieve and a Plasticity Index (PI) not exceeding 20. Due to the variable materials on site, select rough grading and potentially stockpiling will be required to provide an onsite source of MSE Wall backfill, or, imported soil will be needed to meet these criteria.

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

4.1.8 Subgrade Preparation Prior to Fill Placement

In general, removal bottom areas and areas to receive compacted fill should be scarified to a minimum depth of 6 to 8 inches, brought to a near-optimum moisture condition, and re-compacted in place. Removal bottoms and areas to receive fill should be observed and accepted by the geotechnical consultant prior to subsequent fill placement.

Removal bottoms within the active drainages on site should be observed for uniformity and tested to the satisfaction of the geotechnical consultant.

4.1.9 Fill Placement and Compaction

Material to be placed as fill should be brought to near optimum moisture content (generally near optimum to about 2 percent above optimum moisture content) and recompacted to at least 90 percent relative compaction (per ASTM D1557). Deep fills throughout the site are subject to the increased compaction criteria described in Section 4.1.9.1 below. Moisture conditioning (either adding water or drying back) of soils is anticipated to 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 under the observation and testing performed by the geotechnical consultant. Any encountered oversized material as previously defined must be appropriately handled (Appendix E).

Fill placed on 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. During backfill of excavations, the fill should be properly benched into firm and competent soils of temporary backcut slopes as it is placed in lifts.

Fill slope faces should also be compacted to minimum project recommendations. This may require overbuilding of the slope face and trimming back to design grades. To improve surficial stability, vegetation specified by the landscape architect should be established on the slope face as soon as it is practical, refer to Section 4.3.3.

For MSE Wall backfill, it is imperative that adequate compaction meeting project recommendations be obtained in the zone immediately behind the block units where compaction is typically achieved using hand equipment (e.g. whackers, etc.) in lieu of rubber-tired construction equipment.

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

If gap-graded ¾-inch rock is used for backfill (around storm drain storage chambers, retaining wall backfill, etc.) it will require compaction. Rock shall be placed in relatively thin lifts (typically not exceeding 6 inches) and mechanically compacted with observation by geotechnical consultant. Backfill rock shall meet the requirements of ASTM D2321. Gap-graded rock is required to be wrapped in filter fabric to prevent the migration of fines into the rock backfill.

4.1.9.1 Deep Fill Compaction Criteria

To reduce the amount of fill settlement and duration of settlement, fill placed at depths greater than 40 feet below proposed finish grade should be compacted to a minimum of 93 percent relative compaction at near optimum to about 2 percent above optimum moisture content per ASTM D1557.

4.1.9.2 Oversized Placement

Oversized material (material larger than 8 inches in maximum dimension) may be generated during site grading. Recommendations are provided for appropriate handling of oversized materials in General Earthwork & Grading Specifications, Appendix E. Oversize material should not be placed in deep fill areas where an increased minimum relative compaction is required. If feasible, crushing oversized materials or exporting to an offsite location may be considered.

4.1.10 Trench and Conventional Retaining Wall Backfill and Compaction

The onsite soils may generally be suitable as trench backfill, provided the soils are screened of rocks and other material greater than 3 inches in diameter and organic matter. 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 (per CTM 217) 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 tamped to ensure adequate compaction. Subsequent trench backfill should be compacted in uniform thin lifts by mechanical means to at least the recommended minimum relative compaction (per ASTM D1557).

Conventional (masonry) retaining wall backfill should consist of sandy soils as outlined in Section 4.1.6. 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 2 (rear of text). Retaining wall backfill soils should be compacted in relatively uniform thin lifts to at least 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.

4.1.11 Temporary Excavations

In general, we anticipate temporary slopes required for localized removals, over-excavations and haul roads to be grossly stable at 1:1 (horizontal: vertical) or flatter; however, excavations must be made in accordance with Cal OSHA and OSHA requirements. Vehicular traffic, stockpiles, and equipment storage should be set back from the perimeter of excavations a minimum distance equivalent to a 1:1 projection from the bottom of the excavation.

It should be noted that excavations into formational material (landslide or bedrock) are subject to localized failure along bedding. It has been our experience that any side of a trench with "out of slope bedding" is subject to failure, especially with the presence of groundwater. Ultimately, 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 fill them prior to weekends, holidays, or forecasted rain.

4.1.12 Preliminary Shrinkage and Bulking

Volumetric changes in earth quantities will occur when excavated onsite earth materials are replaced as properly compacted fill. The following is an ESTIMATE of shrinkage factors for the various geologic units found onsite. **Please note that these values are preliminary and will be updated based on additional laboratory testing in progress at this time.** These estimates are based on in-place densities of the various materials and on the estimated average degree of relative compaction achieved during grading.

Due to the combined variability in topographic surveys, inability to precisely model the removals and variability in on-site near-surface conditions, it is our opinion that the site will not balance at the end of grading. If importing/exporting a large volume of soils is not considered feasible or economical, we recommend a balance area be designated onsite that can fluctuate up or down based on the actual volume of soil. We recommend a "balance" area that can accommodate on the order of 5 percent of the total grading volume be considered.

TABLE 4

Estimated Shrinkage

Soil Type	Allowance	Estimated Range
Alluvium in Active Drainages	Shrinkage	0% to 15%
Older Alluvium/ Landslide Material	Neutral	0%
Bedrock	Bulkage	5% to 10%

Subsidence due to earthwork equipment is expected to be on the order of 0.1 feet. It should be stressed that these values are only estimates and that actual shrinkage factors are extremely difficult to predict. The effective shrinkage of onsite soils will depend primarily on the type of compaction equipment and method of compaction used onsite by the contractor. Additionally, the onsite geology is very complex; the above estimates are generalized groupings of similar lithologies and should be expected to vary across the site and with depth.

The above shrinkage estimates are intended as an aid for others in determining preliminary earthwork quantities. However, these estimates should be used with some caution since they are not absolute values. Contingencies should be made for balancing earthwork quantities based on actual shrinkage and subsidence that occurs during grading. Shrinkage and bulking are also expected to vary with variations in survey accuracy during rough grading.

Due to the combined variability in topographic surveys, inability to precisely model the removals and variability in on-site near-surface conditions, it is our opinion that the site will not balance at the end of grading. If importing/exporting a large volume of soils is not considered feasible or economical, we recommend a balance area be designated onsite that can fluctuate up or down based on the actual volume of soil. We recommend a "balance" area that can accommodate on the order of 5 percent of the total grading volume be considered.

4.1.13 Rippability and Oversize Material

Based on observations during our subsurface investigation, we anticipate the bedrock and alluvial soils will be rippable with conventional earth-moving equipment in good condition. However, it should be noted that locally cemented beds or concretion nodules may be generated that do not break down and must be handled as "oversize" material during fill placement. Recommendations for handling of oversize is presented in Appendix E, General Earthwork and Grading Specifications.

4.2 Settlement Monitoring

Fill soils are subject to post-grading settlement. This occurs even to properly compacted fill soils with properly constructed subdrains. Areas of development above deep fill areas should not be released for construction until settlement monitoring indicates future settlement has diminished to within the tolerable limits for the proposed foundations and other structural improvements.

Settlement monuments are to be installed promptly after grading in order to monitor settlement of deeper fills (approximately 40 feet or deeper). Surface settlement monuments are recommended at the approximate locations depicted on the Geotechnical Maps (Sheets 1 & 2), to be installed in accordance with the details provided in Appendix E. We estimate that approximately 10 to 12 settlement monuments will be recommended to be installed at completion of grading. While preliminary locations of settlement monuments are shown on the Geotechnical Map, the final locations should be determined during grading based on the actual as-graded fill depths. At the completion of their construction, the monuments should be surveyed by a licensed surveyor to a fixed offsite benchmark (read at three separate locations) to an accuracy of at least 0.01-foot with provided measurements to 0.001-foot. When site grading is completed, the monuments should then be monitored periodically at regular intervals (weekly in the first two months, twice in the third month, and then once a month until informed otherwise by the geotechnical consultant). The readings should be provided to this office and continue until total settlements are within tolerable limits. The frequency of the readings may be adjusted during the monitoring period based on results of the readings.

4.2.1 Settlement Waiting Period

Where overlying deep fills (typically 40 vertical feet or greater), shallow footings and slab-on-grade foundations should be constructed after a waiting period to verify that total settlements are within tolerable limits. Post-construction settlement monitoring is anticipated to need at least three months of data (depending on material type, depth of compacted fill, etc.) before sufficient information is available to confirm settlement is within tolerable limits. Our experience indicates that areas of deeper fill could have a settlement waiting period on the order of 6 months.

4.3 Slope Stability

Global slope stability for the site has been evaluated utilizing cross sections A-A' through D-D'. Buttress keyway limits and dimensions are presented on the Geotechnical Map (Sheets 1 & 2) and Cross Sections (Sheets 3 & 4). Recommendations for construction of slopes are presented below.

4.3.1 Fill Slopes

Design fill slopes at the site are anticipated to be both grossly and surficially stable as designed provided they are constructed in accordance with the Standard Earthwork and Grading Specifications (Appendix E) and proper irrigation, landscaping and maintenance is implemented. Fill slopes should be constructed with a maximum slope ratio of 2:1 (horizontal to vertical). Slope faces should also be compacted to minimum project recommendations. This may require overbuilding of the slope face and trimming back to design grades. To improve surficial stability, vegetation specified by the landscape architect should be established on the slope face as soon as it is practical, refer to Section 4.3.3.

4.3.2 __Natural Slopes__

The moderately steep native slopes along the southwestern boundary of the proposed development may be susceptible to "natural" phenomena such as erosion, sloughing and surficial instabilities such as debris flows. Debris flows occur during periods of prolonged intense precipitation. Slumps or failures within the soil mantle overlying bedrock may produce debris flows. Generally, there are three contributing factors that determine the susceptibility of debris flow potential of a hillside. These factors include the thickness of the colluvial soil mantle, steepness of the slope, and moisture content of the soil. Debris flows are common on slopes with inclinations between 3:1 to 1:1 (horizontal to vertical). During prolonged intense rainfall periods, a perched groundwater table may form within the soil when rainfall exceeds the infiltration rate of the underlying formational material. As the perched groundwater table forms, it reduces the available shear strength of the soil and the soil moves as a viscous fluid. Debris flows commonly follow existing drainages down slope, scarifying surficial soils and accumulating soil and debris that is deposited at the toes of slopes or into site debris basins on a seasonal basis. Isolated debris flows and slumps should be anticipated at the steeper portions of the native slopes around the site.

Native slopes will be left in their existing condition above portions of the site. These slopes will be subject to "natural" phenomena such as erosion, sloughing and surficial instabilities. It is impossible to predict where or when this may happen. Should erosion or slippage occur on a native slope that may impact the site, it should be promptly repaired.

4.3.3 __Slope Maintenance Guidelines__

It is recommended that any graded slopes be planted with ground cover vegetation as soon as practical to protect against erosion by reducing runoff velocity. Deep-rooted vegetation that requires little water and is able to survive local climate conditions should also be established to protect against surficial slumping. Under no circumstances should slopes be allowed to be bare of vegetation. Landscape vegetation should not be "trimmed" to root structures leaving no protection of the slopes. Irrigation levels should be kept to the minimum level necessary to establish healthy plant growth. Slopes must not be overwatered. If automatic sprinklers are used, they must be adjusted during periods of rainfall. A landscape professional should be consulted for landscape recommendations.

A program for the elimination of burrowing animals in both native and graded slope areas should be established to protect slope stability by reducing the potential for surface water to penetrate into the slope. Continuous erosion control, rodent control, and maintenance are essential to the long-term stability of all slopes. Trenches excavated on a slope face for utility or irrigation lines and/or for any purpose should be properly backfilled and compacted to project recommendations (refer to Section 4.1.10) to the slope face. Observation/testing and acceptance by the geotechnical consultant during trench backfill are recommended. V-ditches should be inspected and cleared of loose soil and/or debris on a routine basis, especially prior to and during the rainy season.

4.4 Provisional Foundation Recommendations

Based on the site geotechnical conditions and assuming the remedial recommendations provided herein are implemented, the site may be considered suitable for the support of the proposed residential structures using a post-tensioned slab-on-grade foundation system. Foundations must be designed to resist the impacts of expansive soils and estimated fill settlement. Based on preliminary laboratory testing, soils with "Very Low" to "Medium" expansion potential are anticipated to be encountered. At the completion of grading, if soils with "High" expansion potential are encountered, supplemental geotechnical foundation recommendations will be provided.

4.4.1 Preliminary Post-Tensioned Foundation Design Parameters

The geotechnical parameters provided herein may be used for post-tensioned slab foundations. These parameters have been determined in general accordance with the Post-Tensioning Institute (PTI) Standard Requirements for Design of Shallow Post-Tensioned Concrete Foundations on Expansive Soils, referenced in Chapter 18 of the 2019 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.

Our design parameters are based on our experience with similar projects, test results performed by others, 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 homeowners not properly maintain the areas surrounding the foundation, for example by overwatering and/or incorrect landscape design, 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/higher expansion potential are anticipated to show less/more movement.

TABLE 5

Preliminary Geotechnical Post-Tensioned Foundation Parameters for Medium Expansion

Parameter	PT Slab with Perimeter Footing	PT Mat with Thickened Edge
Expansion Index	Medium[1]	Medium[1]
Center Lift Edge moisture variation distance, e_m Center lift, y_m	 9.0 feet 0.50-inch	 9.0 feet 0.60-inch
Edge Lift Edge moisture variation distance, e_m Edge lift, y_m	 4.7 feet 1.10-inch	 4.7 feet 1.32-inch
Minimum Perimeter footing/thickened edge embedment below finish grade	18 inches[2]	6 inches[2]
1. Assumed for preliminary design purposes. Further evaluation is needed at the completion of grading. 2. Deepened footings may be required in certain areas due to slope setback criteria. 3. Moisture condition to 120% of optimum moisture content to a minimum depth of 18 inches prior to trenching.		

4.4.2 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 5. 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 beam should be expected even with properly constructed planters.

In addition to the factors mentioned above, future homeowners/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 homeowner's responsibility to perform periodic maintenance during hot and dry periods to ensure that adequate watering has been provided to keep soil from separating or pulling back from the foundation. Future homeowners 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 homeowners and property management personnel.

4.4.3 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 and thereby not the purview of the geotechnical consultant. 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.

4.5 Foundation Setback from Top-of-Slope and Bottom-of-Slope

Foundations should have adequate setback from top and bottom of slopes. Per the 2019 CBC, the minimum top-of-slope setback is H/3, with a maximum required setback of 40 feet, where H is the total height of the slope which also includes any MSE wall. This distance is measured horizontally from the outside bottom edge of the footing to the slope face. As an alternative to moving the building footprint, setback requirements may be accomplished by deepened footings or deep foundations.

The minimum bottom-of-slope setback is H/2, with a maximum required setback of 15 feet. Refer to Chapter 18 of the 2019 CBC. Foundation setback criteria should be reviewed based on the precise grading plans.

4.6 Soil Bearing and Lateral Resistance

Provided our earthwork recommendations are implemented, an allowable soil bearing pressure of 1,500 pounds per square foot (psf) may be used for the design of footings having a minimum width of 18 inches and minimum embedment of 12 inches below lowest adjacent ground surface. This value may be increased by 400 psf for each additional foot of embedment or 250 psf for each additional foot of foundation width to a maximum value of 3,000 psf. A mat slab foundation a minimum of 6 inches below lowest adjacent grade may be designed for an allowable soil bearing pressure of 1,000 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-mentioned 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).

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 0.30 may be assumed with dead-load forces. An allowable passive lateral earth pressure of 240 psf per foot of depth (or pcf) to a maximum of 2,400 psf may be used for lateral resistance. Allowable passive pressure may be increased to 325 pcf to a maximum of 3,250 psf for short duration seismic loading. These passive pressures are applicable for level (ground slope equal to or flatter than 5H:1V) conditions only. For a 2:1 (horizontal to vertical) downward sloping condition, a reduced passive lateral earth pressure of 90 pcf to a maximum of 900 psf may be used. This allowable passive pressure may be increased to 120 pcf to a maximum of 1,200 psf for short duration seismic loading. We recommend that the upper foot of passive resistance be neglected if finished grade will not be covered with concrete or asphalt. Frictional resistance and passive pressure may be used in combination without reduction. The provided allowable passive pressures are based on a factor of safety of 1.5 and 1.1 for static and seismic loading conditions, respectively. The structural designer should incorporate appropriate factors of safety and/or load factors in their design.

4.7 Lateral Earth Pressures for Conventional Retaining Walls

Lateral earth pressures for typical retaining wall backfill are presented in Table 6 for approved 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. Retaining wall backfill should also be limited to fill material not exceeding 3 inches in greatest dimension. Please note that the site contains soils that are not suitable for retaining wall backfill based on their expansion potential and fines content, therefore select grading/stockpiling of suitable soils will be required. The retaining wall designer should clearly indicate on the retaining wall plans the required sandy soil backfill.

Lateral earth pressures are provided as equivalent fluid unit weights, in psf/ft 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 retaining wall designer should clearly indicate on the retaining wall plans the required sandy backfill.

TABLE 6

Lateral Earth Pressures – Sandy Backfill

Condition	Equivalent Fluid Unit Weight (pcf)	
	Level Backfill	**2:1 Slope Above**
Active	35	55
At Rest	55	88

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 earth pressure will be higher. This would include 90-degree corners of retaining walls. Such walls should be designed for "at-rest." 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 consultant.

Surcharge loading effects from any 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 basement/retaining wall footing will surcharge the proposed retaining structure. In addition to the recommended earth pressure, retaining walls adjacent to streets/parking areas should be designed to resist vehicle traffic if applicable. Uniform surcharges may be estimated using the applicable coefficient of lateral earth pressure using a rectangular distribution. For a level backfill a factor of 0.33 and 0.50 may be used for at-rest and active conditions, respectively. The vertical traffic surcharge may be determined by the structural designer. The structural designer should contact the geotechnical engineer for any required geotechnical input in estimating any applicable surcharge loads. 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 10 pcf for a level backfill condition. 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 2019 CBC, the seismic earth pressure is applicable to "structures assigned to Seismic Design Category D, E, or F in accordance with Section 1613." While not anticipated at this time, if a retaining wall with a sloping backfill condition is proposed, the retaining wall designer should contact the geotechnical consultant for specific seismic lateral earth pressure increments based on the configuration of the planned retaining wall structures. This seismic lateral earth pressure is estimated using the procedure outlined by the Structural Engineers Association of California (Lew, et al, 2010).

Retaining wall structures should be provided with appropriate drainage and appropriately waterproofed. To reduce, but not eliminate, saturation of near surface (upper approximate 1-foot) soils in front of the retaining walls, the perforated subdrain pipe should be located as low as possible behind the retaining wall. The outlet pipe should be sloped to drain to a suitable outlet. In general, we do not recommend retaining wall outlet pipes be connected to area drains. If subdrains are connected to area drains, special care and information should be provided to homeowners to maintain these drains. Typical retaining wall drainage is illustrated in Figure 2. It should be noted that the recommended subdrain does not provide protection against seepage through the face of the wall and/or efflorescence. Efflorescence is generally a white crystalline powder (discoloration) that results when water containing soluble salts migrates over a period of time through the face of a retaining wall and evaporates. If such seepage or efflorescence is undesirable, retaining walls should be waterproofed to reduce this potential.

Soil bearing and lateral resistance (friction coefficient and passive resistance) are provided in Section 4.6. 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.

4.8 MSE Walls

The following soil shear strength parameters may be used for local stability design of site MSE walls. A soil unit weight of 120 pcf may be used for reinforced, retained and foundation soils.

TABLE 7

Soil Shear Strength Parameters for MSE Wall Design

Soil Zone	**ϕ (Degrees)**	**Cohesion (psf)**
Foundation Soils	30	200
Reinforced Select Sandy Soils*	30	0
Retained Soils	30	0

*Per local stability limits determined by MSE wall designer

The above design soil shear strength parameters are contingent on the proper backfill material being used in the construction of the MSE walls, refer to Section 4.1.7. The MSE wall designer should also incorporate any applicable surcharge loading into the local stability design.

Once the proposed MSE Retaining Wall has been designed (including geogrid type, length, spacing, etc.), the global stability of the slopes shall be re-evaluated. There is potential that these geogrid layers may need to be extended beyond the requirements for local stability. For the up to approximately 30-foot high MSE wall located in the eastern portion of site, an increase in geogrid lengths on the order 5 to 10 feet beyond the requirements for local stability should be anticipated at this time.

Heel drains are recommended for MSE walls constructed where the backcut adjacent to the end of the geogrid is bedrock materials.

The geogrid area behind the wall should be considered a restricted use area where no future excavations should be made into the geogrid zone, in order to protect the integrity of the wall. Damaging the geogrid reinforcement may result in negative consequences with respect to MSE wall stability and long-term durability. Project designers, construction contractors and future homeowners should be made aware of this recommended restricted use area.

Prior to construction, LGC Geotechnical should review proposed MSE wall plans prior to construction to verify that our geotechnical recommendations are implemented.

4.9 ***Subsurface Water Infiltration***

Recent regulatory changes in some jurisdictions have recommended that low flow runoff be infiltrated rather than discharged via conventional storm drainage systems. Typically, a combination of methods is implemented to reduce surface water runoff and increase infiltration including; permeable pavements/pavers for roadways and walkways and directing surface water runoff to grass-lined swales, retention areas, and/or drywells. It should be noted that intentionally infiltrating storm water conflicts with the geotechnical engineering objective of directing surface water away from structures and improvements. The geotechnical stability and integrity of the project site is reliant upon appropriately handling all surface water. 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.

In consideration of the hillside nature of the site and given the site will be underlain by compacted fill, dense older alluvium and/or bedrock, we strongly recommend against the intentional infiltration of storm water into subsurface soils. It is our understanding that no subsurface infiltration is proposed at the site by the project civil engineer.

4.10 ***Preliminary Asphalt Concrete Pavement Sections***

Provisional minimum street sections are provided below for Traffic Indices from 5.0 to 6.0 and an assumed R-value of 20. These recommendations must 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. Final street sections should be confirmed by the project civil engineer based upon the design Traffic Index. Determination of the Traffic Index (TI) is not the purview of the geotechnical consultant. If requested, LGC Geotechnical will provide sections for alternate TI values.

TABLE 8

Paving Section Options

Assumed Traffic Index	5.0	5.5	6.0
R-Value Subgrade	20	20	20
AC Thickness	4.0 inches	4.0 inches	4.0 inches
Aggregate Base Thickness	5.0 inches	7.0 inches	8.5 inches

The thicknesses shown are for <u>minimum</u> thicknesses. Increasing the thickness of any or all of the above layers will reduce the likelihood of the pavement experiencing distress during its service life. The above recommendations are based on the assumption that proper maintenance and irrigation of areas adjacent to the roadway 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 Section 4.1 ("Site Earthwork") and the related sub-sections of this report.

4.11 Soil Corrosivity to Concrete and Metal

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 our testing with regard to corrosion for the use of the client and other consultants, as they determine necessary.

Corrosion testing indicated soluble sulfate content less than approximately 0.01 percent, chloride content of approximately 64 parts per million (ppm), pH value of 6.92, and minimum resistivity of 2,114 ohm-cm. Based on Caltrans Corrosion Guidelines (2018), 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 1,500 ppm (0.15 percent) or greater. The test results indicate the tested soils are not considered corrosive using Caltrans criteria.

Based on laboratory sulfate test results, the near surface soils are designated to a class "S0" per ACI 318, Table 19.3.1.1 with respect to sulfates. Concrete in direct contact with the onsite soils can be designed according to ACI 318, Table 19.3.2.1 using the "S0" sulfate classification. This must be verified based on as-graded conditions.

4.12 Control of Surface Water and Drainage Control

From a geotechnical perspective, we recommend that compacted finished grade soils adjacent to the proposed structure be sloped away from the proposed structure and towards an approved drainage device or unobstructed swale. Drainage swales, wherever feasible, should not be constructed within 5 feet of building structures. Drainage should be designed by the project civil engineer <u>so that a properly constructed and maintained system will prevent</u>

ponding within 5 feet of the foundation. Code compliance of grades is not the purview of the geotechnical consultant.

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.

4.13 Nonstructural Concrete Flatwork

Nonstructural concrete flatwork (such as walkways, patios, bicycle trails, etc.) has a high 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 minimum guidelines outlined in Table 9. These guidelines will 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. Please note that these are preliminary recommendations that will need to be confirmed and/or modified based on as-graded conditions at the completion of grading.

TABLE 9

Minimal Guidelines for Nonstructural Concrete Flatwork for Medium Expansion Potential

	Homeowner Sidewalks	Private Drives	Patios/Entryways	City Sidewalk Curb and Gutters
Minimum Thickness (in.)	4 (nominal)	5 (full)	5 (full)	City/Agency Standard
Presoaking	Wet down prior to placing	Presoak to 12 inches	Presoak to 12 inches	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	—	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 driveways to separate from the garage slab, the builder may elect to install dowels to tie these two elements together. Similarly, future homeowners should consider the use of dowels to connect flatwork to the foundation.

4.14 Additional Geotechnical Field Exploration and Plan Review

When available, any updated rough, precise grading, MSE and conventional retaining wall and foundation plans should be reviewed by LGC Geotechnical in order to verify our geotechnical recommendations are implemented. Updated recommendations and/or additional field work may be necessary especially for verification of suitability of site soils for MSE Wall backfill and verification and recommendations for conditions in alluvial removals below MSE Walls. Additional analysis may be needed for further evaluation for excavation sequencing/temporary stability, MSE wall global stability, settlement potential in design fill areas, etc. At completion of rough grading, additional laboratory testing and post-grading recommendations will be required for site development.

4.15 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 1705 of the 2019 California Building Code (CBC).

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

- During rough grading (removal/over-excavation bottoms, fill placement, buttress keyway excavation, etc.);
- Geologic mapping of temporary backcuts and buttress keyway backcut;
- During MSE and retaining wall backfill and compaction;
- During utility trench backfill and compaction;
- During precise grading;
- 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 building and wall footing excavation and prior to placement of steel reinforcement and/or concrete; and
- When any unusual soil conditions are encountered during any construction operation subsequent to issuance of this report.

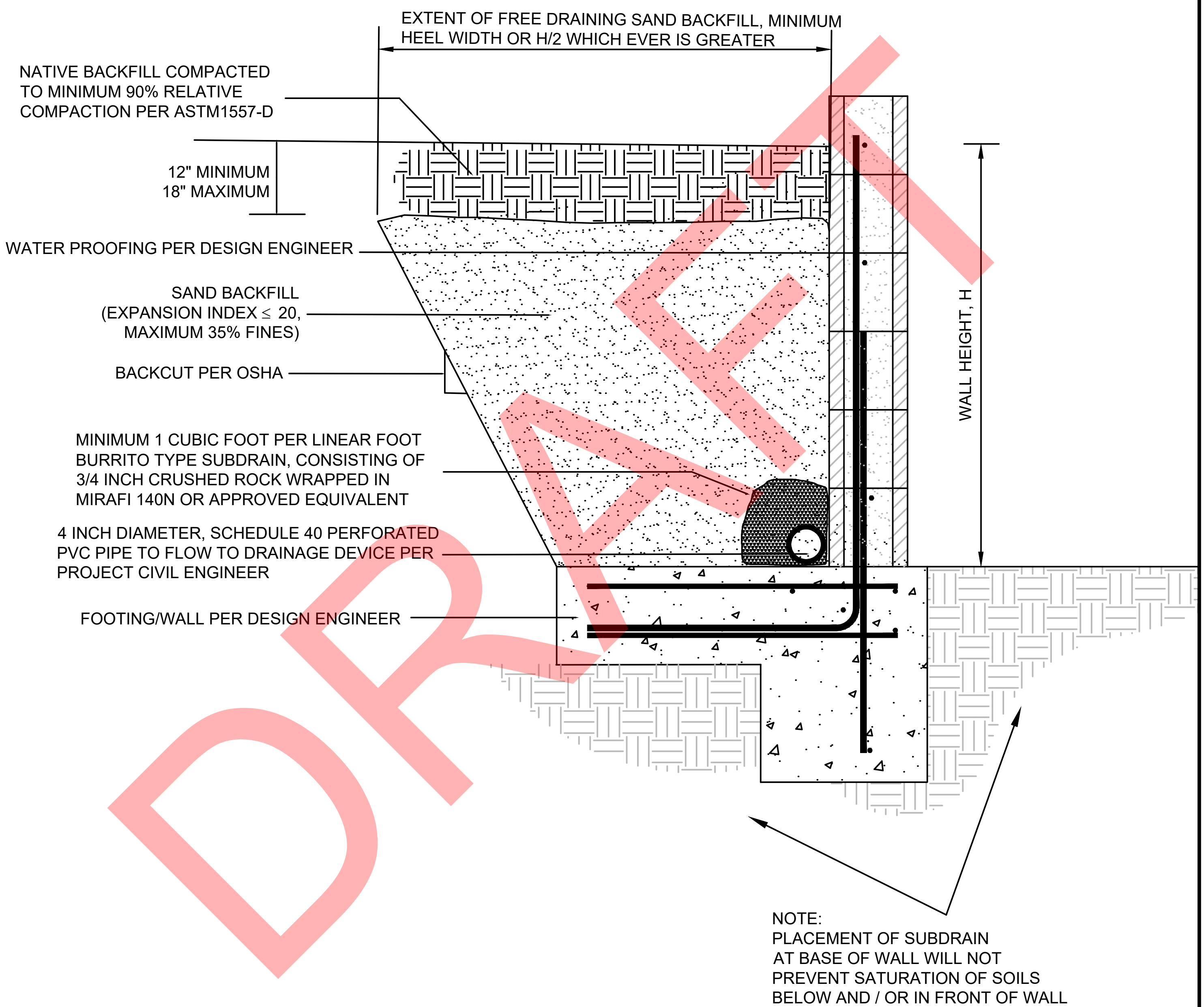
5.0 LIMITATIONS

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.

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.



NOTE:
This Figure is not applicable to retaining walls constructed at the top of slopes

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FIGURE 2
Retaining Wall Backfill Detail

PROJECT NAME	Trumark - Shady View
PROJECT NO.	18117-01
ENG. / GEOL.	DJB / KTM
SCALE	Not to Scale
DATE	April 2020

Appendix A
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APPENDIX A

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Appendix B
Logs of Exploratory Excavations and Infiltration Test Results

Appendix C
Laboratory Test Results

Appendix D
Slope Stability Analyses

Appendix E
General Earthwork Specifications for Rough Grading

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Geotechnical Map

PROJECT NAME	Trumark - Shady View
PROJECT NO.	18117-01
ENG. / GEOL.	DJB / KTM
SCALE	1" = 60'
DATE	April 2020

SHEET 1 of 4

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PROJECT NAME	Trumark - Shady View
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SCALE	1" = 60'
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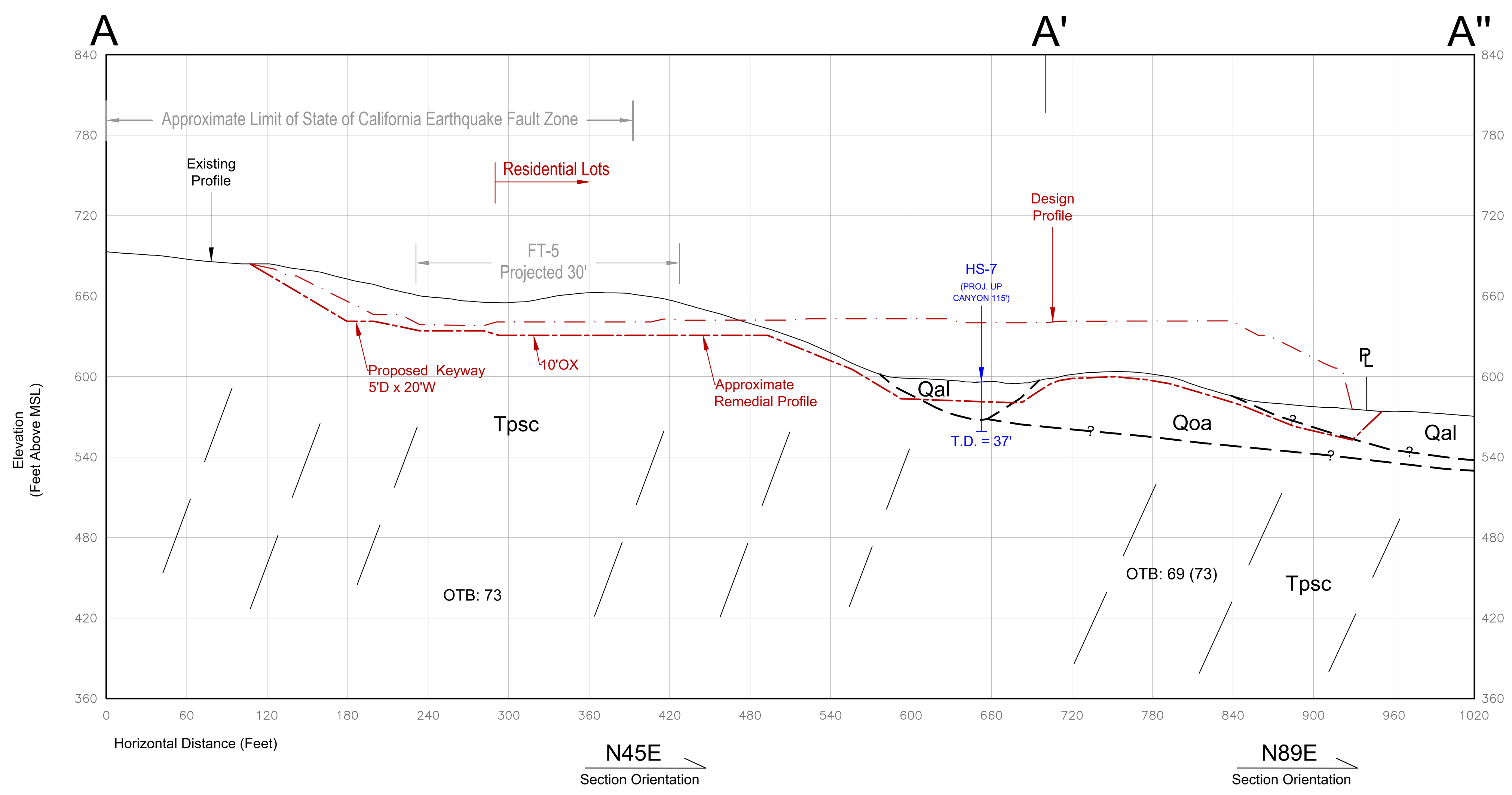
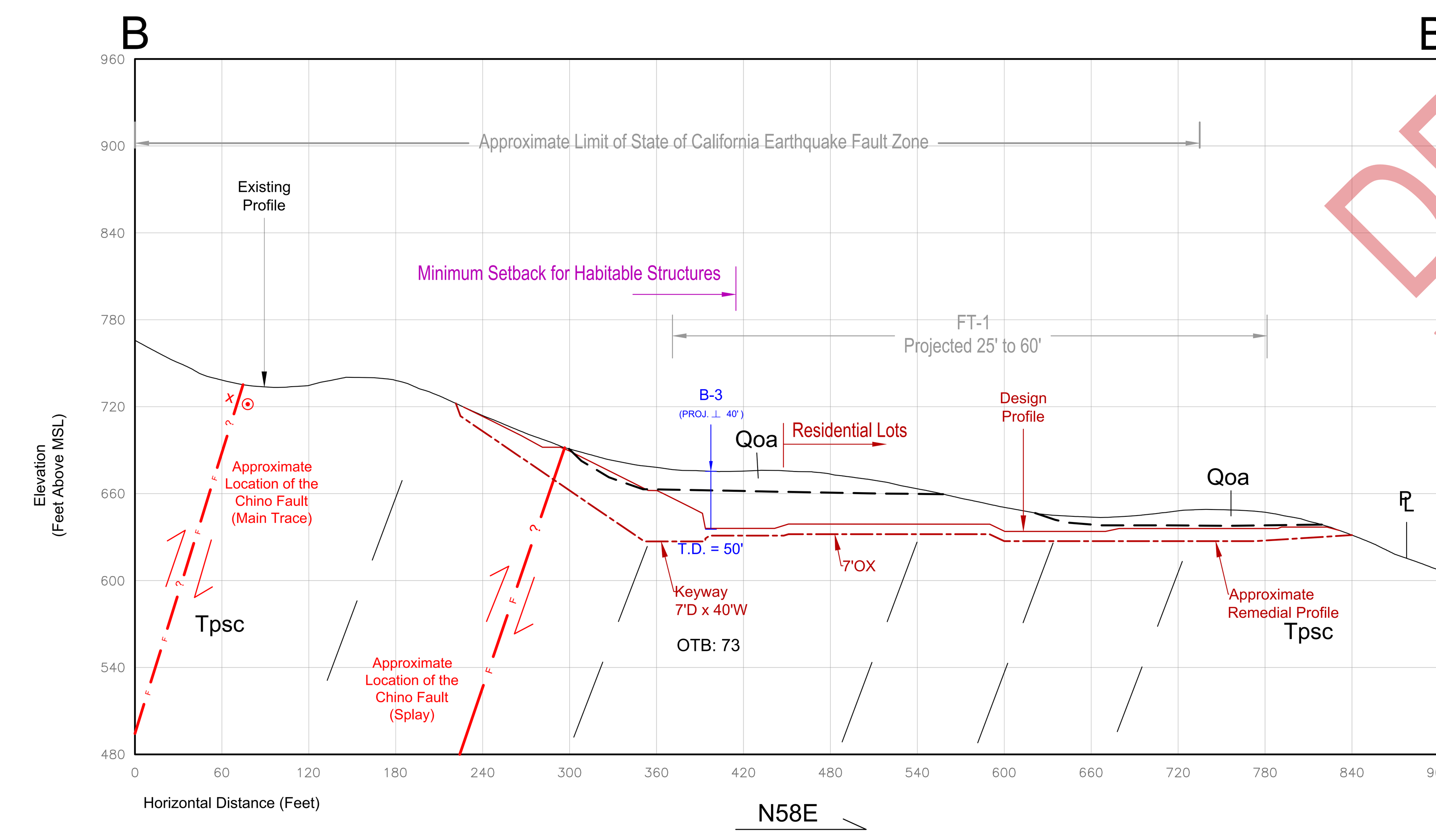
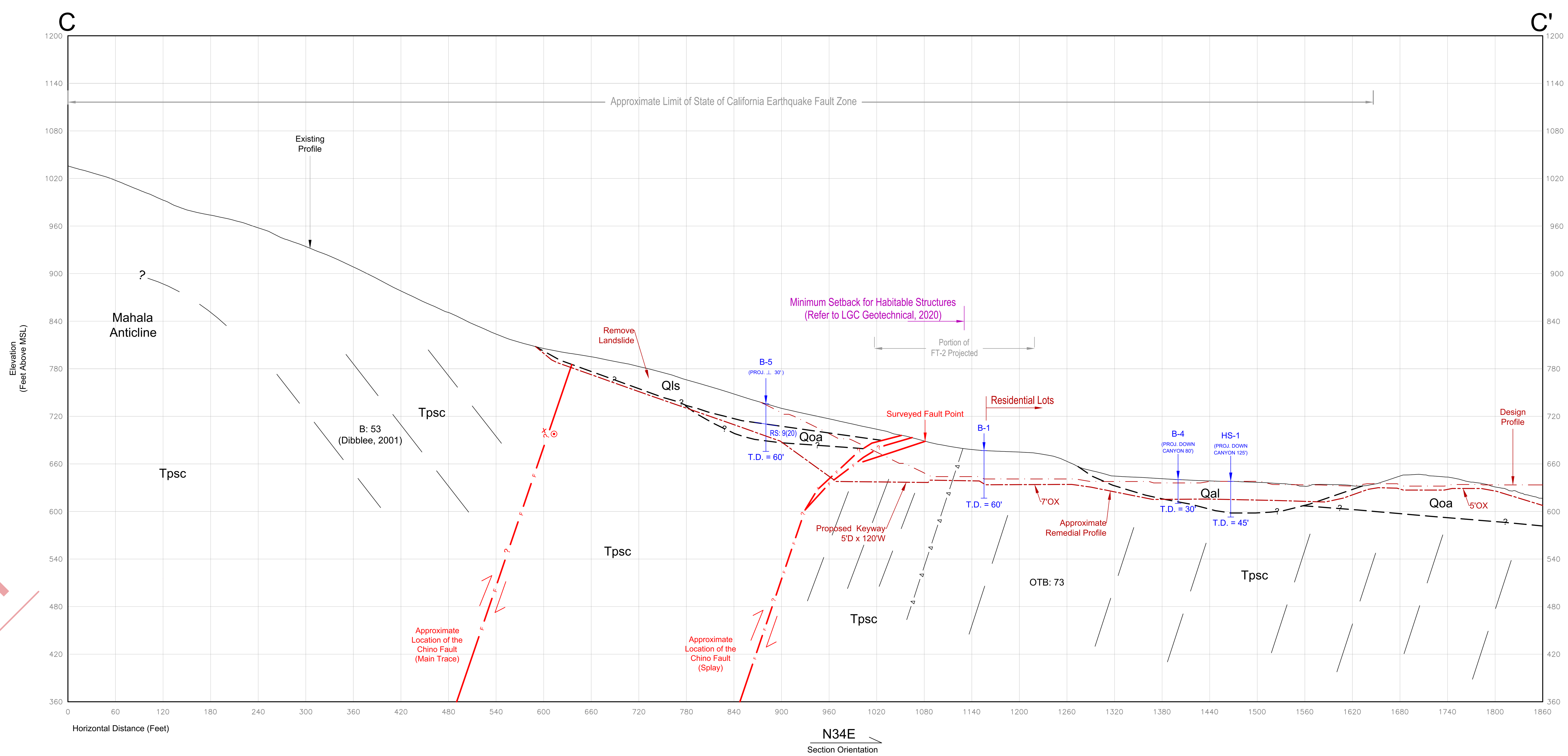
SHEET
2 of 4







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60' 0 60' 120'

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Geotechnical Cross Sections A-A" through C-C'

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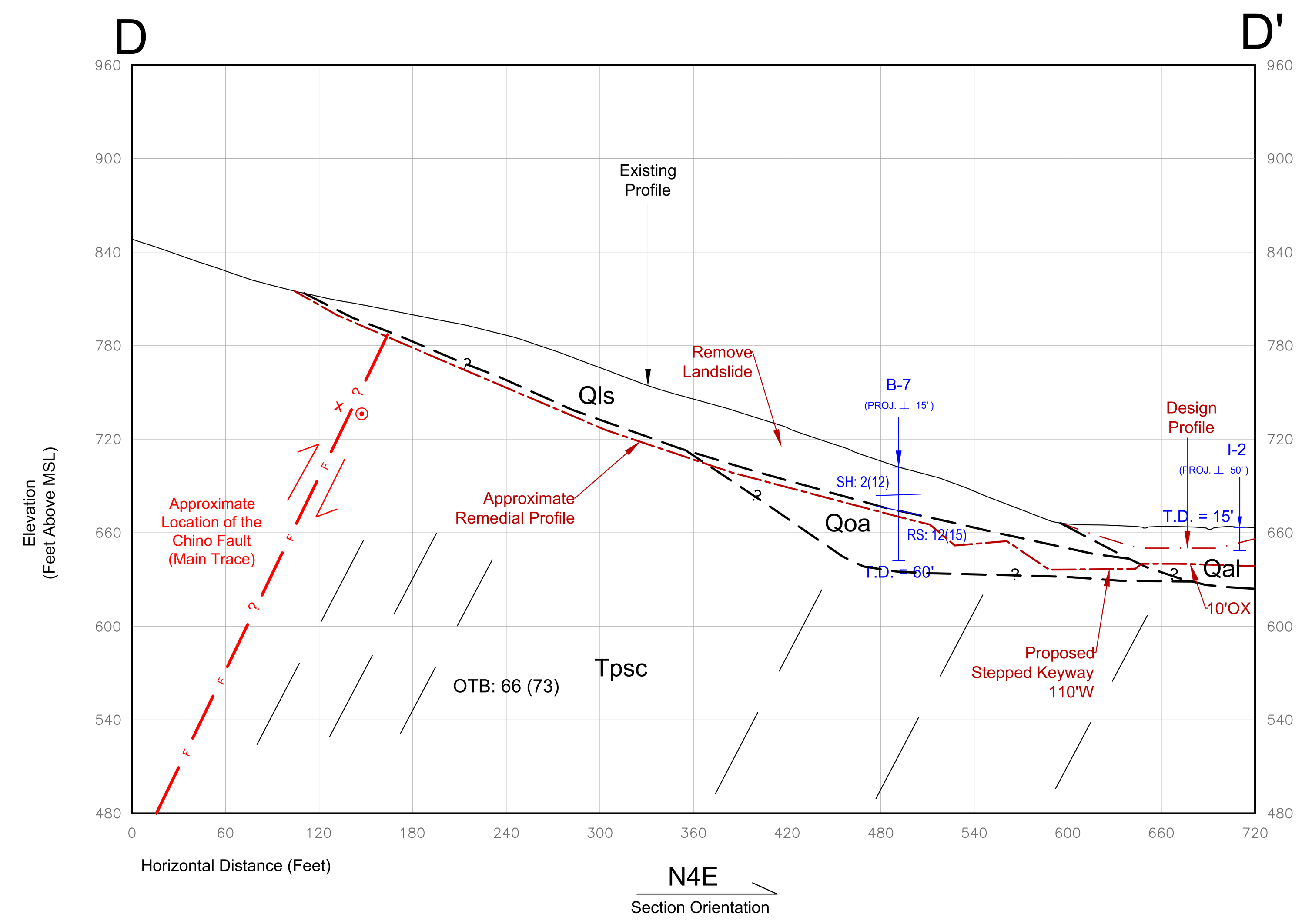
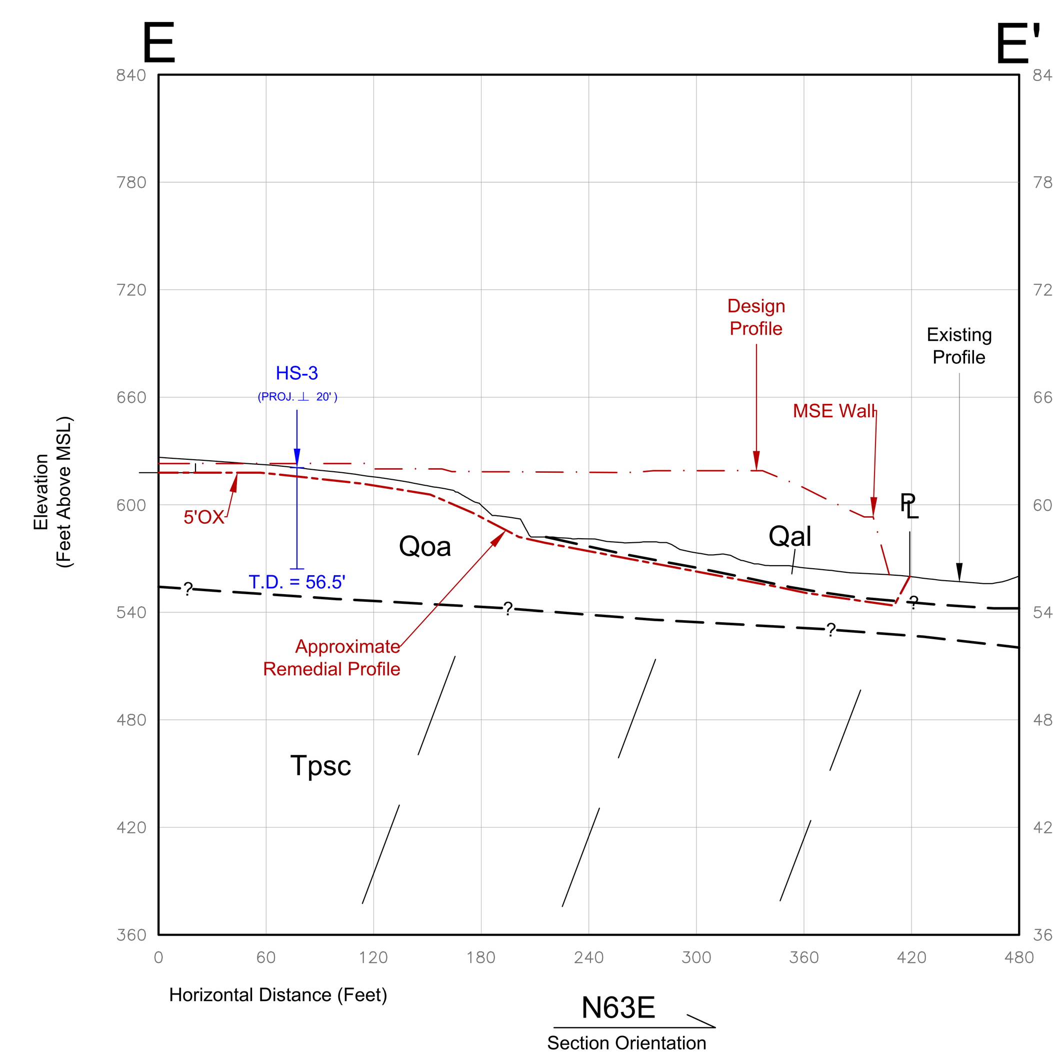
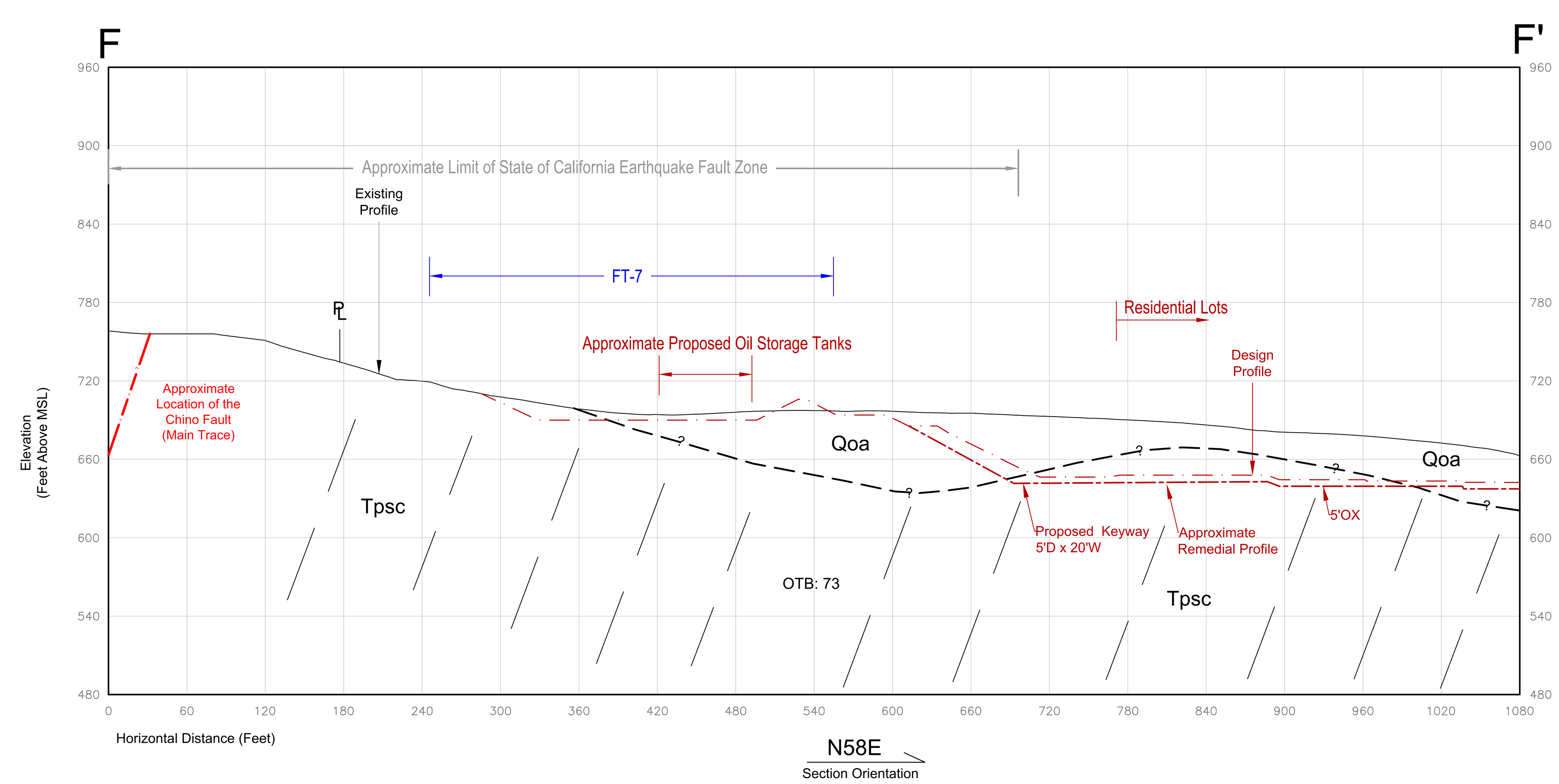
PROJECT NO.	18117-01
ENG. / GEOL.	DJB / KTM
SCALE	1" = 60'
DATE	April 2020







SCALE: 1"=60'

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Geotechnical Cross Sections D-D' through F-F'

PROJECT NAME	Trumark - Shady View
PROJECT NO.	18117-01
ENG. / GEOL.	DJB / KTM
SCALE	1" = 60'
DATE	April 2020