GEOTECHNICAL INVESTIGATION
TENTATIVE TRACT NO. 70583
APPROXIMATELY 300-ACRE SITE
SAN DIMAS FOOTHILLS
SAN DIMAS, CALIFORNIA
PREPARED FOR
NJD, LTD.
JOB NO. 09355-3

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August 20, 2009

NJD, LTD.
3300 East 1st Avenue, Suite 500
Denver, Colorado 80206
Attention: Mr. John Scott

Job No. 09355-3

Dear Mr. Scott:

Attached herewith is the Geotechnical Investigation report, prepared for the proposed Tentative Tract no. 70583, approximately 300-acre site, San Dimas, California.

This report was based upon a scope of services generally outlined in our proposal dated June 23, 2009, and other written and verbal communications.

We appreciate this opportunity to provide geotechnical services for this project. If you have questions or comments concerning this report, please contact this firm at your convenience.

Respectfully submitted,
C.H.J., INCORPORATED

John S. McKeown

John S. McKeown, E.G.
Project Geologist

JSM:ndt

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INTRODUCTION

During July and August of 2009, a geotechnical investigation for the proposed residential development designated as Tentative Tract No. 70583 and located in the foothills of the San Gabriel Mountains in the northwest portion of the City of San Dimas was performed by this firm. The purpose of this investigation was to explore and evaluate the geotechnical engineering conditions at the subject site and to provide appropriate geotechnical engineering recommendations for design and construction of the proposed structures and site improvements.

To orient our investigation at the site, a 60-scale Tentative Tract Map No. 70583 & Conceptual Grading Plan, prepared by Fuscoe Engineering, dated June 15, 2009, was furnished for our use. The plans depict a proposed development scheme including tentative building pad and street elevations. References made to lot numbers within this report reflect lot numbers shown on the Tentative Tract Map. The approximate location of the site is shown on the attached Index Map (Appendix "A").

The results of our investigation, together with our conclusions and recommendations, are presented in this report. This report incorporates the information presented in our previous letter report titled, "Report of Engineering Geologic Investigation, Tentative Tract No. 70583, Approximately 300-Acre Site, San Dimas Foothills, San Dimas, California", dated June 24, 2009.

SCOPE OF SERVICES

The scope of services provided during this geotechnical investigation included the following:

- Incorporation and update of the engineering geologic data presented in our letter, dated June 24, 2009, based on the geotechnical explorations and analysis performed for this investigation
- Additional geologic field reconnaissance of the site and surrounding area

- Update of the geologic mapping of the site at a scale of 1 inch equal to 100 feet
- Logging and sampling of exploratory test pits on the site
- Logging and sampling of hollow-stem auger borings for testing and evaluation
- Down-hole logging of large-diameter bucket-auger borings
- Laboratory testing on selected samples
- Seismic refraction survey lines at selected locations within the site
- Evaluation of the geotechnical engineering/geologic data to develop site-specific recommendations for site grading, conventional static and seismic foundation design/pile foundations, slope stability, preliminary pavement structural section design, and mitigation of potential geologic constraints

PROJECT CONSIDERATIONS

As we understand it, the project is to consist of development of approximately 125 acres of the 300-acre site. The developed portion of the site will include 61 single-family residential lots and associated infrastructure. It is proposed to construct a main access road to the planned residences from the northern terminus of existing Cataract Avenue along the west fork of Shuler Canyon. Roads within the development will provide access to the various clusters of building lots.

A water tank pad is proposed for Lot F that occupies a ridge in the northeast portion of the development area. The development is proposed to be connected to a sewer system; therefore, percolation testing was not within the scope of this investigation.

The conceptual grading plan indicates maximum cut depths of 64 feet and 40 feet for residential pads and the water tank pad, respectively. Maximum fill depths are proposed as 75 feet, 60 feet, 55 feet, and 50 feet for residential pads, an embankment, a retention basin, and a roadway embankment, respectively. We have assumed, for the purposes of this investigation, that all the proposed retention basins will be lined with an appropriate impermeable material. If this is not the case, additional evaluation of the basin berms for slope stability may be necessary.

The final project grading plan should be reviewed by the geotechnical engineer.

SITE DESCRIPTION

The site consists of approximately 300 acres of hillside land that is primarily undeveloped. Dirt roads provide access to limited areas of the site. Elevations range from 1,860 feet above mean sea level (amsl) to 1,025 feet amsl in the northeastern and southwestern portions of the site, respectively. The majority of the site is heavily vegetated with grasses and annual plants. Chaparral-type plants are common in drainage bottoms and on slopes. North- and northwest-facing slopes and canyon bottoms include areas of dense vegetation including trees and tall shrubs. South-facing slopes and ridge tops tend to be less heavily vegetated. Large patches of cactus are present within the site and occur primarily within the volcanic bedrock areas of the site.

Prior development within the site is limited to a former "corral/stables" area in the west-central portion of the site where an abandoned building and several paddock remnants are located and an existing structure in the east-central portion of the site. A series of abandoned power poles are present near the western site boundary. It is anticipated that a septic-type disposal system was utilized in the area of the corral/stables building. A large steel water tank and buried steel water line is present on a ridge west of the former corral/stables. Various fences and gates are located within and around the site perimeter.

Metal and glass debris including appliance shells and an automobile were observed in a canyon west of Lot 49. An accumulation of building materials and tools was observed in the area of Lot 27. Concrete debris was observed at the head of West Shuler Canyon. Other areas of minor fill debris were also observed.

The topography of the northern portion of the site is characterized by steep topographic relief in a series of northeast/southwest trending ridges separated by narrow ravines formed in bedrock materials of the Glendora Volcanics. The southern portion of the site is characterized by a broad, generally east-west trending upland formed in Puente Formation siltstone dissected by east-west oriented and southwest draining, steep-sided ravines.

A series of historic aerial photographs at various scales were reviewed for this project. The dates of the photographs span the time period from 1928 through 1998. In the earliest photographs dated circa 1928, the site appears as undeveloped land vegetated with grasses and trees are limited to north- and northwest-facing slopes. 1934-era photographs show numerous debris flow scars in the southeast portion of the site associated with steep slopes and limited road building. The 1936-era photographs show the existing fire access roads within the site and the former corral/stable in the valley area of the site. Cattle trails are visible traversing the site slopes in the 1936-era photographs. Subsequent photographs show progressive vegetation growth in the canyon bottoms. Photographs dated 1998 show the former corral/stable area in the western valley area of the site in use, together with an apparently-related development just west of the western site boundary. Internet-based photographs suggest abandonment of the corral/stable area subsequent to 1994.

Several landslides and landslide-related features were observed on the aerial photographs. These include soil-slip debris flows and deep-seated bedrock landslides. The locations of these features is shown on the attached Geologic Map and Site Plan, and further discussion is provided in a following section. Faults and/or fault-related features indicative of active faulting, as defined by the California Geologic Survey, were not observed on the aerial photographs reviewed.

FIELD INVESTIGATION

The soil conditions underlying the subject site were explored by means of 40 exploratory test pits (TP) excavated with a tire-mounted backhoe to a maximum depth of approximately 16 feet below the existing ground surface (bgs), 7 hollow-stem auger borings (HSA) drilled to a maximum depth of approximately 53-1/2 feet bgs using a truck-mounted CME 75 drill rig equipped for soil sampling, and 7 bucket-auger borings (BH) drilled to a maximum depth of approximately 60 feet bgs using a 24"-diameter bucket-auger rig. The bucket-auger holes were down-hole logged. The approximate locations of our test pits and borings are indicated on the enclosed Geologic Map and Site Plan (Appendix "A").

For the test pit excavations, continuous logs of the subsurface conditions, as encountered within the explorations, were recorded at the time of excavation by a staff geologist from this firm. Bulk samples of typical soil types obtained were returned to the laboratory in sealed containers for testing and evaluation.

For the hollow-stem auger borings, continuous logs of the subsurface conditions, as encountered within the exploratory borings, were recorded at the time of drilling by a staff geologist from this firm. A ring sampler (3.25-inch outer diameter and 2.42-inch inner diameter) was utilized in our investigation. The penetration resistance was recorded on the boring logs as the number of hammer blows used to advance the sampler in 6-inch increments (or less if noted). The samplers were driven with an automatic hammer dropping a 140-pound weight 30 inches for each blow. After the required seating, samplers were advanced up to 18 inches, providing up to three sets of blowcounts at each sampling interval. The recorded blows are raw numbers without corrections for hammer type (automatic vs. manual cathead) or sampler size (ring sampler vs. SPT sampler). Relatively undisturbed as well as bulk samples of typical soil types obtained were returned to the laboratory in sealed containers for testing and evaluation.

For the bucket-auger borings, continuous logs of the subsurface conditions, as encountered within the exploratory borings, were recorded at the time of drilling by a staff geologist from this firm. In selected bucket-auger borings, bulk samples and relatively undisturbed ring samples (obtained with the 3.25-inch outer diameter and 2.42-inch inner diameter ring sampler) were collected. The borings were then entered and logged down-hole by an engineering geologist from this firm.

Our exploratory logs, including the uncorrected blowcount data for the hollow-stem auger and bucket-auger borings, are presented in Appendix "B". The boundaries between strata presented on the logs represent approximate boundaries between soil types which may include gradual transitions.

LABORATORY INVESTIGATION

Included in our laboratory testing program were field moisture content tests on all samples returned to the laboratory and field dry density tests on all relatively undisturbed samples. The results are included on the exploratory boring logs. Optimum moisture content - maximum dry density relationships were established for typical soil types in order that the relative compaction of the subsoils might be evaluated. Relatively undisturbed direct shear and consolidation tests, as well as remolded direct shear tests, were performed on selected samples in order to provide shear strength and consolidation parameters for slope stability, bearing capacity, earth pressure and settlement evaluations. Expansion index tests were performed on selected samples of clay-bearing soil to evaluate their expansion potential. Sieve analyses and Atterberg limits were performed on selected samples of soil for classification purposes. Selected samples of material were delivered to Schiff Associates for preliminary corrosivity analysis.

Laboratory test results appear in Appendix "C". Soil classifications provided in our geotechnical investigation are generally as per the Unified Soil Classification System (USCS).

SITE GEOLOGY AND SUBSURFACE SOIL CONDITIONS

The geologic materials and structure within the site were explored by field mapping of accessible outcrops, excavation of backhoe test pits, placement of hollow-stem auger borings, down-hole logging of large diameter bucket-auger borings, and seismic refraction surveys at selected locations within the site. Streitz (1966), Dibblee (2002), and Morton and Miller (2003) included the area of the site in geologic mapping. The site is underlain by two bedrock units separated by an inactive fault that trends approximately east-west through the central portion of the site. The bedrock units include sedimentary rocks of the Puente Formation and extrusive rocks of the Glendora Volcanics. The bedrock units are mantled by fill, young alluvium, colluvium, debris flow deposits, older terrace deposits, and landslide debris of variable thickness.

Fill (f)

Fill (unit symbol f) is present locally within the site and is associated with grading of roads and development of the southern portion of the flat-lying corral area located in the west-central portion of the site. Roadway fill is derived from local materials including the bedrock units and colluvium and is limited to the area immediately adjacent to roads within the site. A larger area of roadway fill is located at the topographic saddle between Lots 19 and 21. Fill in the corral area includes abundant concrete and asphalt debris with lesser metal, glass, and wood debris. The corral area fill was encountered to a depth of 17 feet in Exploratory Boring No. HSA-1, where refusal occurred on large debris. The corral area fill is estimated to be up to 40 feet thick at the head of the west branch of Shuler Canyon and thins toward a daylight line in the central portion of the corral area based on topographic relations and data from site explorations. A smaller area of fill debris, including concrete fragments, was encountered in the southern portion of Lot 21. Refusal was encountered at a depth of 9 feet bgs in Test Pit No. TP-31 within this area. The undocumented fill generally consists of silty sands, clayey sands, and silts and range from loose to medium dense states. The undocumented fill and debris is considered unsuitable for support of structures.

A bathroom was observed in a portion of the existing building in the corral area. Therefore, a septic-type disposal system was likely utilized in the area of the corral. Leach lines and/or seepage pits may be encountered during demolition and grading of the building. Complete removal and abandonment of leach lines and/or seepage pits, if encountered, should be performed.

Alluvium (Qya)

Alluvium (unit symbol Qya) consisting of unconsolidated silty sand and clayey sand in loose to medium dense states was encountered at the mouth of Shuler Canyon. The lower portion of this unit contained gravel to 3 inches in size.

Colluvium (Qcol)

Colluvium (unit symbol Qcol) consisting of sandy silty clay and silty clayey sand in loose to medium dense states, derived from weathering and gravity transport of the bedrock materials, is present on ridge tops, on slopes, and in ravines and drainages within the site. This is the most widely distributed surficial material within the site; however, due to its limited thickness, is mapped only in areas where thicker accumulations were observed. Thick accumulations of colluvium occur locally within the narrow drainage/ravine bottoms in the site. The unconsolidated colluvium is subject to gravity creep on slopes and is considered a typical source for debris flow-type slope failures triggered by heavy precipitation events. The colluvium is generally considered unsuitable for support of structures.

Older Terrace Deposits (Qt)

Older terrace deposits (unit symbol Qt) were observed in two areally-limited ridge top locations capping Puente Formation bedrock. These deposits consist of weathered, well-rounded gravel and cobbles of volcanic and gneissic lithologies in an orange-brown sandy matrix. The elevation difference between these two areas suggests deposition by a west-flowing drainage system.

Landslide Deposits (Qls)

Landslide deposits (unit symbol Qls) were observed in the northwest portion of the site and locally within the west-central and southwestern portions of the site. The presence of landside deposits is based on evaluation of the topographic map of the site, review of aerial photographs, prior mapping by others, and observation of rock outcrops and geomorphology at the site. Landslide debris derived from the Glendora Volcanics unit is anticipated to consist of materials ranging in composition from pervasively crushed and sheared rock material that is highly weathered to relatively intact, hard and durable angular boulder-size clasts that are difficult to excavate. Landslide debris derived from the Puente Formation unit is anticipated to consist of a mixture of angular siltstone and sandstone fragments in a matrix of sandy clayey silt. Further discussion of landslides is provided in the section titled "SLOPE STABILITY."

Debris Flow Deposits (Qdf)

Debris flow deposits of variable age (based on degree of cementation) were observed in a road cut and in test pits located in the Wildwood Canyon area. These materials consist of clast-supported, poorly-sorted, subrounded to angular gravel and cobble clasts in a silty sand matrix. The debris flow deposits exposed in the road cut include boulder size clasts of Glendora Volcanics up to 5 feet in size. These materials are typically mantled by colluvium (unit Qcol).

Puente Formation (Tp)

The Puente Formation (unit symbol Tp) consists primarily of whitish to tan, thinly- and well-bedded siltstone and more thickly-bedded sandstone. The siltstone and sandstone weather to a dark gray clay-bearing soil mantle and erode to form a more subdued topography relative to the volcanic materials. Bedding attitudes within the Puente Formation strike primarily along an east-west axis and dip toward the north and south. Bedding is locally folded or overturned near faults and fold structures. Dibblee (2002) mapped a series of folds within the southern portion of the site that are expressed as alternately north- and south-dipping beds. Hard, cemented siliceous beds were observed in limited areas of the site and in bucket-auger borings. Jointing formed oblique to bedding is common within the siltstone units. Gypsum fillings along beds and fractures were observed locally in the Puente Formation.

Glendora Volcanics (Tgv)

The Glendora Volcanics (unit symbol Tgv) as designated for this investigation includes andesite flows and tuff breccia and locally a fine-grained basalt member that together comprise a steep-weathering formation exposed in road cut and limited natural outcrops. The andesite is fine-grained and locally porphyritic with primarily discontinuous, pervasive joints. The tuff breccia contains abundant rounded cobbles and angular andesite rock fragments and forms durable outcrops that stand at steep angles. The basalt member is limited in areal extent in natural exposures and appeared more regularly jointed than the other volcanic units. For the purposes of this investigation, the volcanic units are mapped as a single unit. The Glendora Volcanics weather to clay-bearing, slope-mantling sandy sediments that form the soil cover within the northern portion of the site. Large boulders (up to 5 feet in size) derived from the volcanic units were noted locally as rock fall below steep outcrops within the site. Outcrop patterns within the volcanics are less predictable than in the siltstone, due to the mechanics of extrusion and emplacement. Geologic structure in the volcanics is generally massive with pervasive small-scale jointing predominating over less common well-developed joints formed in fine-grained exposures. No consistent orientation of structure was observed.

Engineering Considerations of Geologic Materials

Groundwater was encountered in Exploratory Boring No. HSA-4 at a depth of 42 feet bgs. Groundwater was not encountered in any other exploration within the site.

The results of Expansion Index (E.I.) testing (ASTM D 4829) range from 18 to 101, indicating a "very low" to "high" potential expansion classification according to the 2007 California Building Code (CBC). Soils with an E.I. of 21 or greater require special consideration for foundation design to mitigate potential detrimental effects of expansive soils. Soils with an E.I. of 21 or greater will be encountered during grading. Selected grading or specialized foundations will be necessary should the expansive soils be encountered within the building pad areas.

Bedrock was encountered in multiple explorations within the site and underlies the majority of the site beneath a thin soil mantle. Refusal was experienced within all of the bucket-auger borings drilled in Glendora Volcanics. Refusal was not encountered in any of the exploratory borings drilled in the Puente Formation.

More detailed descriptions of the subsurface soil conditions encountered are included within our exploratory boring logs (Appendix "B").

FAULTING

The site does not lie within or immediately adjacent to an Alquist-Priolo Earthquake Fault Zone, designated by the State of California to include traces of suspected active faulting. In addition, mapped active faults and/or evidence of active faulting within, or projecting toward the proposed residential development portions of the site, was not observed on the geologic maps and aerial photographs reviewed for this investigation.

The tectonics of the Southern California area are dominated by the interaction of the North American plate and the Pacific plate, which are sliding past each other in a transform motion. Although some of the motion may be accommodated by rotation of crustal blocks such as the western Transverse Ranges (Dickinson, 1996), the San Andreas fault zone (SAFZ) is thought to represent the major surface expression of the tectonic boundary and to be accommodating most of the transform motion between the Pacific Plate and North American Plate. Some of the plate motion is accommodated along other northwest-trending, strike-slip faults that are related to the San Andreas system, such as the Newport-Inglewood, San Jacinto, and Elsinore faults. Local convergence related to a bend in the overall trend of SAFZ is accommodated along buried thrust faults within the Los Angeles basin such as the Puente Hills Blind-Thrust (PHT) system and the Northridge Thrust, and exposed faults including the Sierra Madre-Cucamonga fault system.

A bedrock fault that separates Puente Formation (Tp) from Glendora Volcanics (Tgv) trends roughly east-west through the site (Enclosure "A-2.2"). This fault was measured to strike N42W and dips to the northeast at an angle of 52 degrees near Test Pit No. 12. Prior work by Leighton & Associates (2000) included trenching across this fault trace in the area of Lot No. 59 where a thick cumilic soil profile was observed to be developed over the fault zone. This fault is considered inactive for planning purposes.

The Sierra Madre fault is mapped along the southern margin of the foothills within and near the southern boundary of the site. This area of the site does not include proposed occupancy structures. The Sierra Madre fault consists of several arcuate splays that characterize a system of frontal thrust faults that extend from the Santa Monica Mountains in the west to the Cucamonga fault zone and eastern San Gabriel Mountains known as the Transverse Ranges Frontal Fault system (TRFFS). The M_L 5.8 Sierra Madre earthquake on June 28, 1991 occurred on the Clamshell-Sawpit Canyon fault; an offshoot of the Sierra Madre fault located approximately 7 miles west of the site. The February 9, 1971 San Fernando (Sylmar) earthquake (M_W 6.6) occurred on the San Fernando fault; a member of the TRFFS.

The Cucamonga fault is located approximately 4 miles east of the site. The Cucamonga fault is part of a series of east-west trending, predominantly reverse and thrust faults coincident with the southern margin of the San Gabriel Mountains known as the Transverse Ranges frontal fault system. The San Fernando fault of this system ruptured during the 1971 moment magnitude (**M**) 6.7 San Fernando earthquake. Evidence of recent activity on the Cucamonga fault includes fresh scarps, sag ponds, and disrupted Holocene alluvium (Dutcher and Garrett, 1963; Yerkes, 1985; Morton and Yerkes, 1987).

The San Jose fault is located approximately 5 miles south of the site and trends from the southwestern San Jose Hills northeastward to the Upland-Claremont region. The San Jose fault is the source of a M_L

4.7 earthquake on June 26, 1988 and a larger M_L 5.4 earthquake on February 28, 1990 called the Upland earthquake, both epicentered northeast of the site.

The Mojave segment of the San Andreas fault zone is located along the northeast margin of the San Gabriel Mountains, approximately 18 miles northeast of the site. The San Andreas fault is characterized by youthful fault scarps, vegetational lineaments, springs, and offset drainages. The 1857 Fort Tejon earthquake of approximate 7.9 M_W occurred on the Mojave segment of the San Andreas fault. The Working Group on California Earthquake Probabilities (1995) tentatively assigned a 28 percent (± 13 percent) probability to a major earthquake occurring on the San Bernardino Mountains segment of the San Andreas fault between 1994 and 2024.

Other faults in the southern California region with a potential for producing seismic shaking at the site include the Chino-Central Avenue, Puente Hills blind thrust, Raymond, and Whittier faults, located 8 miles southeast, 9-1/2 miles southwest, 10-1/2 miles west, and 13-1/2 miles south of the site, respectively.

HISTORICAL EARTHQUAKES

The site, like most areas of southern California, is located within a seismically-active region. A map of recorded earthquake epicenters is included as Enclosure "A-6" (Epi Software, 2000). This map includes the California Institute of Technology database for earthquakes with magnitudes of 4.0 or greater from 1932 through 2008. Mitigation of the potential for damage due to seismic shaking is primarily through proper design and construction according to the current CBC.

2007 CALIFORNIA BUILDING CODE - SEISMIC PARAMETERS

Based on the geologic setting and anticipated earthwork for construction of the proposed project, the bedrock profile underlying the site is classified as Type *Sc*, "very dense soil and soft rock", according to the 2007 CBC. For areas of the site with fill and/or competent native soils having an aggregate thickness of 50 feet or greater over the bedrock profile, the site is classified as Type S_D, "stiff soil profile", according to the 2007 CBC.

The seismic parameters according to the 2007 CBC are summarized in the following table.

2007 CBC - Seismic Parameters		
Site Class	S_C	S_D
Mapped Spectral Acceleration Parameters	$S_s = 2.59$ and $S_1 = 0.97$	$S_s = 2.59$ and $S_1 = 0.97$
Site Coefficients	$F_a = 1.0$ and $F_v = 1.3$	$F_a = 1.0$ and $F_v = 1.5$
Adjusted Maximum Considered Earthquake (MCE) Spectral Response Parameters	$S_{MS} = 2.59$ and $S_{M1} = 1.26$	$S_{MS} = 2.59$ and $S_{M1} = 1.45$
Design Spectral Acceleration Parameters	$S_{DS} = 1.72$ and $S_{D1} = 0.84$	$S_{DS} = 1.72$ and $S_{D1} = 0.97$

The corresponding value of peak ground acceleration (PGA) from the design response spectra according to the 2007 CBC is 0.69g. The value of PGA is independent of site class.

SEISMIC REFRACTION INVESTIGATION

Seismic refraction surveys were performed at three locations within the site to estimate the rippability of the Glendora Volcanics bedrock material in the areas where proposed cuts exceed approximately 40 feet in depth. The locations of the seismic refraction survey lines are indicated on Enclosure "A-2" by the designations "SRL-1" through "SRL-3". A detailed description of the methodology and results is included in Appendix "D". Seismic refraction survey lines were limited to areas of the site underlain by the Glendora Volcanics as the Puente Formation bedrock material is anticipated to be rippable to the proposed excavation depths using conventional grading equipment.

The seismic velocity can be utilized to estimate the rippability of subsurface materials by heavy grading equipment. A chart included in the Caterpillar Performance Handbook (1992) correlates seismic velocity of different rock types to rippability by a D9R bulldozer utilizing single- or multi-shank rippers. This chart indicates that basalt rock, similar to the Glendora Volcanics, is rippable to a velocity of up to 7,500 feet per second (fps). Basalt rock exhibiting velocities between 7,500 and 8,700 fps is considered to be marginally rippable, and basalt rock exhibiting velocities greater than 8,700 fps is considered to be non-rippable. Our experience indicates that the velocities in the Caterpillar Performance Handbook (1992) are approximately 1,500 fps too fast for reasonable production rates; therefore, we have utilized a velocity of 6,000 fps to estimate the limit of rippable materials.

Measured seismic velocities at the site varied according to location. In general, rock exhibiting velocities suggesting marginally- to non-rippable conditions was encountered at a minimum depth of 54 feet bgs at the location of SRL-1. Based on the proposed maximum cut depth of approximately 65 feet in the area of Lot 36, approximately 10 feet of marginally- to non-rippable rock may be encountered. Rock exhibiting velocities suggesting marginally- to non-rippable conditions was encountered at depths ranging from 29 to 41 feet bgs at the location of SRL-2. Based on the proposed maximum cut depth of approximately 36 feet in the area of Lot F, approximately 7 feet of marginally- to non-rippable rock may be encountered. Rock exhibiting velocities suggesting marginally- to non-rippable conditions was encountered at depths ranging from 25 to 40 feet bgs at the location of SRL-3. Based on the proposed maximum cut depth of approximately 42 feet in the area of Lot 42, approximately 17 feet of marginally- to non-rippable rock may be encountered.

These data are intended as a guideline in estimating the depth of rippable materials in areas to be graded. The interface between weathered and fresh rock is anticipated to vary laterally and with depth. In addition, corestones that exhibit velocities indicative of marginally- to non-rippable conditions may be present locally at shallower depths, as seen in the tomographic profile for SRL-3 at Station 160. These corestones are anticipated to potentially generate oversize rock material, but should be removable by excavation around their margins.

SLOPE STABILITY

Stability of Existing Slopes

The natural topography of the northern portion of the site is characterized by steep topographic relief in a series of northeast/southwest trending ridges separated by narrow ravines formed in bedrock materials of the Glendora Volcanics. These areas include cliff-type slopes with rock blocks that have tumbled downslope as rock falls and more gently-sloping terrain mantled by accumulations of soil derived from in-situ weathering of the underlying rock. The volcanic bedrock is locally jointed and fractured and generally lacks continuous adversely oriented discontinuities; however, several landslides are identified with the volcanic rock units. The volcanic rock weathers to a potentially expansive, clay-bearing colluvial soil mantle.

The southern portion of the site is characterized by a broad, generally east-west trending upland formed in Puente Formation siltstone/sandstone dissected by east-west oriented and southwest draining, steep-sided ravines. The Tp is generally well bedded and thinly bedded where siltstone predominates and more thickly bedded where sandstone is the dominant component. The siltstone bedrock is locally siliceous (cemented) and hard and weathers to a potentially-expansive, clay-bearing soil mantle. Jointing is formed generally oblique to bedding; however, continuous or prominent joints were not observed. Bedrock folds were observed locally within borings, in road cut exposures, or in the pattern of mapped bedding orientations.

The majority of natural slopes exhibit gradients of 2 horizontal to 1 vertical [2(h):1(v)] to 1.5(h):1(v). Natural slopes also include areas as steep as 1.25(h):1(v) and locally steeper in areas of erosion and debris flow formation underlain by steep-standing bedrock. Several landslides, suspected landslides, and debris flow scars are present within the project area. The locations of these features are indicated on Enclosures "A-2.1" through "A-2.3".

The term "landslide", as used in this report, refers to deep-seated slope failures at least 10 feet deep that involve movement of primarily bedrock materials. Landslides are typically related to the underlying structure of the parent material. Debris flows are a type of surficial shallow failure that affects the upper weathered soil horizon (colluvium) formed from the parent material. Most debris flows occur during winter seasons with above normal rainfall, and the potential for occurrence can be enhanced in areas of recent land wildfires. Debris flow scars tend to become "absorbed" by the ambient topography within a relatively short time period from their occurrence compared with deep-seated landslides.

The susceptibility of a geologic unit to landsliding is dependent upon various factors, primarily: 1) the presence and orientation of weak structures, such as fractures, faults or joints; 2) the height and gradient of the natural or cut slope; 3) the presence and quantity of groundwater; and 4) the occurrence of strong seismic shaking. Portions of the site are identified as being susceptible to soil slip (Enclosure "A-4", Morton et al., 2003) or having a potential for seismically-induced landsliding or slope failure (Enclosure "A-5", CGS, 1999).

Mitigation measures for landslides include avoidance of construction in areas of known landslides, removal of landslide debris and replacement as engineered fill, use of stabilization (buttress) fills or structural retention systems at the toe of engineered slopes or landslide debris area, removal of material from the "head" area of individual landslides, reducing the height of a slope, and flattening or reducing slope angle. Management of surface irrigation is integral to any mitigation plan to avoid introduction of water into slopes. In some cases, excavation of a large slide mass may lead to further destabilization of ground upslope or may not be practical from an economic viewpoint.

Landslides

The area of Lots 49 through 52 are underlain by mapped landslide deposits or suspected landslide deposits. The landslides consist of west and southwest failing rock masses that exhibit steepened, arcuate source scarps and debris areas of moderately sloping topography that exhibit "hummocky" morphology. Portions of these deposits are visible in road-cut exposures at the northern boundary of Lots 51 and 52 and at the western boundary of Lot 49.

Loose rock blocks and a low scarp area were observed at the top of the slope above a landslide located north of and adjacent to Lots 51 and 52, suggesting continued creep of rock in the head scarp. This landslide is visible on aerial photographs of the site taken in multiple years. Roadcuts formed through the slide mass expose pervasively sheared and crushed rock material that is highly weathered and lacks intact rock material or blocks. It is anticipated that this feature will be exposed in the proposed cut slope located northeast of Lot No. 52 and may present a hazard to Lot No. 52. Mitigation may require removal of landslide debris or construction of a stabilization fill. A determination of the potential hazard and method of mitigation may be made at the time of grading.

Exploratory Boring No. BH-6, drilled within the area of a previously suspected landslide in the area Lots 51, 52, and 53, encountered refusal in hard rock at a depth of 8 feet bgs. A prior core boring performed in this area by Leighton & Associates (2000) encountered apparently intact bedrock of the Glendora Volcanics based on the logs presented in their report. This boring was terminated at a depth of 51 feet. This area is depicted on Enclosure "A-2.1" as underlain by intact bedrock based on the available subsurface data for this area of the site.

The toe of a large, previously mapped landslide is located in the northwest corner of the site. This landslide extends offsite toward the northeast, approximately 1 mile to its source area. This landslide is not located within the development area and is not anticipated to affect the proposed improvements.

A landslide underlying the area of Lot No. 49 and a portion of Lot No. 50 is identified based on aerial photographs and topographic relations. This landslide is visible on aerial photographs of the site taken in multiple years. Road cuts formed through the slide mass expose apparently intact rock material that is overlain by a thick colluvium/soil profile and locally by debris flow deposits. Cross Section B-B' (Enclosure "F-1") depicts the existing and proposed topographic profile in the area of Lot No. 49 and an estimate of the thickness of this landslide mass based on topographic relations. Mitigation may entail removal of landslide debris or construction of a stabilization fill. Based on the estimated thickness of the landslide deposit, up to approximately 35 feet of material (including the overlying colluvium) would need to be removed for mitigation by removal and replacement. Alternatively, construction of the proposed fill slope located west of Lot No. 49 as a stabilization fill could mitigate the potential for future movement of this slide mass. Further evaluation of this landslide is presented in the section titled, "SLOPE STABILITY ANALYSIS".

Two landslides, identified on aerial photographs and by geomorphic and topographic relations, "toe" into the west branch of Shuler Canyon within the alignment of a proposed roadway in the southwestern portion of the site. Based on the orientation of bedding measured within Test Pit TP-39, it appears that a shallow out-of-slope component of dip may contribute to instability of slopes in this area. Cross Section E-E' (Enclosure "F-2") depicts the existing and proposed topographic profile in the area of the roadway and an estimate of the thickness of this landslide mass based on topographic relations. Mitigation may entail removal of landslide debris or construction of a stabilization fill. Based on the estimated thickness of the landslide deposit, up to approximately 20 feet of material (including the overlying colluvium) would need to be removed for mitigation by removal and replacement. The landslide debris could be used in site fills. Alternatively, the proposed 50-foot deep canyon fill for construction of the roadway as a stabilization fill could mitigate the potential for future movement of this slide mass. Further evaluation of this landslide is presented in the section titled, "SLOPE STABILITY ANALYSIS".

In general, design and construction of the roadway in the west branch of Shuler Canyon will depend on conditions identified during grading or at such time as access for exploration equipment will allow investigation of the road alignment. Landslides and/or potentially unstable materials identified during grading will require mitigation according to the guidelines presented in the section titled, "RECOMMENDATIONS". The portion of the road alignment located at the head of west Shuler Canyon will encounter deep fill and debris west of Lot No. 1 that will require removal. The proximity of the alignment to the western site boundary may require use of retaining walls on the up-slope side of the alignment if unstable materials are present that require mitigation by removal.

Additional landslides are present in the east branch of Shuler Canyon; however, development is not planned in this area of the site. Several landslides are mapped on the flanks of slopes in the area of Lots 19 and 20 and west of the ridge of Lots 54 through 56. The downslope toe areas of these landslides do not underlie or project toward areas proposed for development.

Enclosure "A-2.1" presents our current understanding of known landslide deposits within proposed building areas of the site. It should be noted that the current conclusions with regard to existing landslides within the site are based on geomorphic relations, review of aerial photographs, observations of existing roadcut exposures, and explorations limited to relatively shallow depths due to the hard character of the Glendora Volcanic unit and limited access to Shuler Canyon in the southern portion of the site. The depth of currently recognized landslide deposits was estimated using balanced cross sections and topographic relations and, during grading, may be found to be greater or less than estimated. In addition, previously unrecognized landslide deposits may be encountered during grading of the site that will require mitigation by removal or stabilization. In contrast, previously mapped landslide areas may be found to be underlain by intact bedrock during grading.

Debris Flow Potential

The site is located in an area identified as locally having a moderate to high potential for generation of debris flows (Enclosure "A-4", Morton et al., 2003). Debris flow scars are present within the site in areas of steep canyon topography, primarily in the "head" areas of drainages. Identification of individual debris flows is not practical with regard to mitigation; rather, planning and design of project improvements should consider the potential effects of debris flow activity to down-stream/down-slope improvements including building lots and roadways. Mitigation may include debris flow deflection walls or berms and debris basins in canyons. Proposed building lots that have a potential for debris flow hazard include those lots located downslope of ungraded native slopes including Lot Nos. 1-5, 47-50, and 58-61. The locations of debris flow scars identified during the engineering geologic investigation are shown on Enclosures "A-2.1" through "A-2.3".

Stability of Proposed Slopes

Engineered cut and fill slopes are proposed at inclinations of 2(h):1(v) or lesser gradient. In some locations, engineered slopes are proposed above or below existing natural slopes with gradients steeper than 2(h):1(v). A series of Geologic Cross Sections is presented in Appendix "F" to illustrate the proposed slope geometries for the project. Calculations and analysis for selected proposed and natural slopes is presented in the Section titled, "SLOPE STABILITY ANALYSIS".

SLOPE STABILITY ANALYSIS

The stability of slopes was analyzed under both static and seismic conditions for rotational and plane failures using Spencer's method or Modified Bishop's method for cases in which Spencer's method gives invalid results, as recommended by the Implementation Committee of DMG Special Publication 117 (Blake, T.F., et al, 2002). Slide verison 5.042 (Rocscience, 2008) computer program was utilized for our analysis. Our seismic calculations were performed using a lateral pseudostatic seismic coefficient (kH) of 0.20, considering the distance to active faults.

REPRESENTATIVE CROSS SECTION:

Cross Sections A-A', B-B', and E-E' were selected as the representative cross sections for slope stability analyses of existing slopes. To evaluate the stability of high fill slopes, an 80-foot high 2(h):1(v) fill slope was analyzed. Fills were assumed to be placed on a 5(h):1(v) native slope with a 12-foot wide and 6-foot wide terrace at heights of 30 and 60 feet, respectively.

STRENGTH PARAMETERS:

To obtain the shear strength parameters, direct shear tests were performed on selected samples of Qcol, Puente Formation (Tp), and Glendora Volcanics (Tgv). Due to the difficulties in obtaining relatively undisturbed samples, only two direct shear tests were performed on relatively undisturbed samples from Tp material. For other materials, samples remolded to a relative compaction of 90 percent were used. To model the shear strength of the material within suspected slide zones, a direct shear test was performed with repeated shearing to a shear displacement of approximately 1/2 inch, corresponding to approximately 100 percent shear strain. The final residual shear strength was used to model the materials of the sheared bedrock.

To obtain the undisturbed shear strength of Tgv materials, back calculations under static and seismic conditions were performed on an idealized 200-foot-high (or less) and 1.5(h):1(v) slope, which was observed within the site as the highest and steepest stable native slope in Puente Formation and Glendora Volcanics. The results of the back calculations indicate an internal friction angle of 39 degrees and a cohesive strength of 940 pound per square foot (psf). Cut slopes formed in native bedrock at gradients of 2(h):1(v) or less are anticipated to be stable under static and seismic conditions as modeled for this investigation.

The strength parameters utilized in our calculations are summarized in the following table.

Table 1: Material Strength Properties

Material	Friction Angle (○)	Cohesion (psf)	Unit Weight (lb./ft^3)	Note
Fill (Tgv)	31	155	117	derived from Tgv
Fill (Tp)	31*	150*	115	derived from Tp
Qcol	28	150	117	remolded
Qls (above sheared bedrock)	21	430	113	from remolded Tgv
Qls (sheared bedrock)	19	940	120	repeated shearing
Tgv	39	900	130	back calculation
Tp	24	740	121	relatively undisturbed
TP(J)†	29	640	115	BH-4@20'/CGS 1998

*Includes results of testing by L&A (2000) **adverse bedding condition †neutral bedding condition

EFFECT OF GROUNDWATER:
For evaluation of existing landslides (Cross Sections B-B' and E-E'), it was anticipated that the sheared bedrock zone will be saturated. As such, groundwater was modeled near the top surface of the sheared bedrock zone. For other cross sections, materials were assumed to be in an unsaturated condition.

RESULTS:

***Section A-A'*:**
Section A-A' is composited from a cut lot of Lot 51 and a cut and fill lot of Lot 52. The fill was assumed to be derived from Tgv material. The results are included in Enclosures "F-4" and "F-5" for static and seismic conditions, respectively. The results satisfy the required 1.5 for static factors of safety and 1.1 for seismic factors of safety.

***Section B-B'*:**
Section B-B' includes a suspected slide plane between Qls and Tgv. An approximate 1-foot thick layer was modeled to represent the sheared slide plane zone. Both static and seismic stabilities along

approximately the middle of the zone and within the zone were analyzed. The results are shown in Enclosures "F-6" through "F-9". The results satisfy the required 1.5 for static factors of safety and 1.1 for seismic factors of safety.

Potential rotational failures were also evaluated for section B-B'. A retaining wall was modeled as having infinite strength to ensure the local stability. For a wall founded approximately 9 feet into the fill, both static and seismic factors of safety did not satisfy the required values (Enclosures "F-10" and "F-11"). The anticipated seismic sliding surface (Enclosure"F-8") cuts through the fill and the underlying native Qcol and Qls, suggesting that these materials could not provide sufficient strength for stability of the slope under seismic conditions. Therefore, the Qcol and underlying Qls materials (anticipated to be disturbed by suspected sliding) should be removed and recompacted. The results of an analysis that models the removal and recompaction of the Qcol and Qls materials and increasing the depth of the retaining wall to 14 feet are included in Enclosures "F-12" and "F-13". The results indicate that the seismic factor of safety does not satisfy the required value of 1.1. A yield acceleration coefficient of 0.235g was obtained for this configuration (Enclosure "F-14").

To satisfy the required factor of safety under the seismic condition for Section B-B', the inverted length of retaining wall could be increased to 23 feet. The result is shown in (Enclosure "F-15"). Alternatively, increasing the cohesion of the fill material by mixing with other on-site cohesive material could be performed to increase the stability of the fill. The results of this analysis are shown in Enclosure "F-16".

Section E-E':

Section E-E' includes a suspected slide plane surface and a potentially daylighted component of apparent bedding dip. The potential for rotational failures and the potential for failure along the slide plane surface and bedding planes were analyzed. The results are included in (Enclosures "F-17" through "F-28"). These results indicate that with the proposed fill in the valley, both static and seismic stabilities satisfy the required factors of safety of 1.5 and 1.1, respectively.

Proposed Fill Slopes:

Two types of fill slopes, fill derived from Tgv over native Tgv and fill derived from Tp over native Tp, were evaluated. The results are included in (Enclosures "F-29" through "F-32"). These results indicate that proposed 2(h):1(v) slope configurations that include a 12-foot-wide lower bench and 6-foot-wide upper bench satisfy the required factors of safety of 1.5 and 1.1, respectively.

Section J-J':

Section J-J' was constructed through a slope formed in Puente Formation (Tp) from the location of an existing residence eastward into Shuler Canyon. The potential for rotational failures was analyzed under a static condition, and an infinite strength was modeled for the proposed retaining wall and roadway. The results indicate that the existing slope does not satisfy the required factor of safety of 1.5 for the static condition (Enclosure "F-33"). A buttress fill, modeled for Section J-J', results in a slope that meets the required factors of safety for both static and seismic conditions (Enclosures "F-34" and "F-35").

SETTLEMENT

STATIC SETTLEMENT:

Calculations were performed utilizing Terzaghi's method to estimate potential static settlements due to foundation and fill loading at Exploratory Boring No. HSA-4 to represent placement of fill over the older colluvium soil. We performed calculations for anticipated post-grading soil conditions. Utilizing the maximum anticipated fill and alluvial depths, our calculations indicated total maximum settlement of approximately 9 inches for the area of Lot Nos 1 through 5, 14, and 58 through 61.

Settlement monitoring of fills greater than 40 feet vertical should be performed as given in the section "SETTLEMENT MONITORING" in order to verify substantial completion of compression of the fill.

SEISMIC SETTLEMENT:

Based on the dense nature of the sediments and bedrock underlying the site, seismic settlement is not considered a hazard at the site.

HYDROCONSOLIDATION:

As shown in Appendix "B", density testing and blowcount data from our exploratory borings indicates that soils encountered were generally in medium dense to very dense or stiff to hard states and are considered to be of very low hydroconsolidation potential.

GROUNDWATER AND LIQUEFACTION

The site is located in portions of Sections 27 and 34 of Township 1 North, Range 9 West, north of the San Gabriel Valley groundwater basin. The area of the site is underlain by bedrock formations at shallow depths and is not considered a groundwater-production area. Springs or seeps were not noted within the site; however, a spring is shown on the U.S. Geological Survey 7.5-minute topographic map near the western boundary of the site within proposed Lot "E". Groundwater was encountered within clayey sand in Exploratory Boring No. HSA-4 at a depth of 42 feet bgs but not in Exploratory Boring Nos. HSA-2, HSA-3, or HSA-5, suggesting that this occurrence is localized. Groundwater may occur in the subsurface within bedrock fractures or as "perched" water on the bedrock/colluvium interface and/or bedrock/alluvium interface locally.

Liquefaction is a process in which strong ground shaking causes saturated soils to lose their strength and behave as a fluid (Matti and Carson, 1991). Ground failure associated with liquefaction can result in severe damage to structures. The geologic conditions for increased susceptibility to liquefaction are: 1) shallow groundwater (generally less than 50 feet in depth), 2) the presence of unconsolidated sandy alluvium, typically Holocene in age, and 3) strong ground shaking. All three of these conditions must be present for liquefaction to occur.

The site is not located within an area identified as having a potential for liquefaction by the California Geological Survey (p.k.a. State of California Division of Mines and Geology, 1999). Based on the anticipated groundwater conditions within the site and the dense nature of the sediments and bedrock underlying the site, liquefaction is not considered a hazard at the site.

FLOODING AND EROSION

The site is located in an area in which flood hazards are "undetermined but possible" (FEMA, 2008). It is anticipated that site improvements will be constructed according to accepted standards and practices generally utilized in design of similar improvements in the site region. Drainage and impound structures/improvements are included in the tentative tract map provided. The assessment and/or mitigation of flooding hazard to the site falls under the purview of others.

The native soils mantling the site are considered moderately susceptible to erosion. Positive drainage should be provided, and water should not be allowed to pond anywhere on the site. Water should not be allowed to flow over any graded or natural areas in such a way as to cause erosion. Finish graded areas should be protected from the effects of runoff.

EXPANSION POTENTIAL

The results of E.I. testing (ASTM D 4829) range from 18 to 101, indicating a "very low" to "high" potential expansion classification according to the 2007 CBC. Soils with an E.I. of 21 or greater require special consideration for foundation design to mitigate potential detrimental effects of expansive soils. Soils with an E.I. of 21 or greater will be encountered during grading. Selected grading or specialized foundations will be necessary should the expansive soils be encountered within the building pad areas.

CONCLUSIONS

On the basis of our field and laboratory investigations, it is the opinion of this firm that the proposed development is feasible from a geotechnical engineering and engineering geologic standpoint, provided the recommendations contained in this report are implemented during grading and construction.

Based upon our field investigation and test data, it is our opinion that the upper native soils and existing fills, will not, in their present condition, provide uniform or adequate support for the proposed structures. These conditions may cause unacceptable differential and/or overall settlement upon application of the anticipated foundation loads. Site clearing can be expected to further aggravate the settlement-prone conditions.

Because of site conditions, it will be necessary to remove the upper 24 inches of existing soil in all areas to be graded. Further sub-excavation may be necessary, depending on the density of the underlain soils or the presence of undocumented fill.

In the area of Exploratory Boring No. HSA-7, it appears that deeper removal of younger alluvium (Qya) soil may be necessary. Removal depth in this area may exceed 15 feet bgs.

In addition, the corral area fill is estimated to be up to 40 feet thick at the head of the west branch of Shuler Canyon and thins toward a daylight line in the central portion of the corral area based on topographic relations and data from site explorations. Removals in this area are anticipated to be in excess of 40 feet bgs.

Other smaller areas of road fill and loose alluvium exist throughout the site. Removals of all unsuitable material should be performed and the subsequent excavation bottom approved by the engineering geologist prior to placement of any fill.

Utilizing the maximum anticipated fill and alluvial depths, our calculations indicated settlement of approximately 9 inches for the area of Lot Nos. 1 through 5, 14, and 58 through 61.

Removal and replacement of the alluvial material in the area of Lot Nos. 1 through 5, 14, and 58 through 61 should be performed to remove the consolidatable material. Maximum removals on the order of 60 feet bgs should be anticipated. Observation and approval by the engineering geologist should be performed during the grading operation.

Settlement monitoring of fills greater than 40 feet vertical should be performed as given in the section "SETTLEMENT MONITORING" in order to verify substantial completion of compression of the fill.

To provide adequate support for the proposed structures, it is our recommendation that the building areas be further subexcavated as necessary and recompacted to provide a compacted fill mat beneath footings and slabs. A compacted fill mat will provide a dense, uniform, high-strength soil layer to distribute the foundation loads over the underlying soils. Conventional spread foundations, either individual spread footings and/or continuous wall footings, may be utilized in conjunction with a non-expansive compacted fill mat.

E.I. tests indicated that selected clay bearing soils at the site have a "very low" to "high" expansion potential (E.I. of 18 and 107) when tested as per ASTM D 4829. It is our recommendation that soils utilized beneath structures consist of granular, non-clay-bearing soils. Removal and replacement of the expansive soil or mixing of the expansive soil with on-site or imported non-expansive material to lower it's E.I. to less than 20 may be performed. The depth of removal and replacement or mixing of the expansive soil below the proposed foundation system should be sufficient to ensure a constant moisture content in the remaining fill. As this may not be feasible, we have included recommendations for construction utilizing existing clay-bearing soils. Additional evaluation of soils for expansion potential should be conducted by the geotechnical engineer during the grading operation.

Evidence of active faulting on or immediately adjacent to the building areas of site was not observed during the geologic field reconnaissance or on the aerial photographs reviewed. The site is not located within an Alquist-Priolo Earthquake Fault Zone.

Landslides and debris flows were mapped within the site that will require mitigation by design or during grading.

Moderate to severe seismic shaking of the site can be expected during the lifetime of the proposed structures.

Based on the historic depth of groundwater and dense nature of the sediments beneath the site, liquefaction is not a hazard to the site.

Evidence of historic flooding at the site was not observed.

Temporary excavations are anticipated to conform to local and State codes with regard to the geologic materials present at the site. Finished slope configurations are not anticipated to exceed 2(h):1(v); therefore, slope stability hazards are not anticipated.

Provided that recommendations which are provided in this report are implemented, construction of the proposed improvements appears to be feasible from geological and geotechnical standpoints without adversely affecting the adjacent properties or existing on-site improvements.

RECOMMENDATIONS

SEISMIC DESIGN CONSIDERATIONS:

Moderate to severe seismic shaking of the site can be expected during the lifetime of the proposed structures. Therefore, the proposed structures should be designed accordingly.

Based on the geologic setting and anticipated earthwork for construction of the proposed project, the bedrock profile underlying the site is classified as Type *Sc*, "very dense soil and soft rock", according to the 2007 CBC. For areas of the site with fill and/or competent native soils having an aggregate thickness of 50 feet or greater over the bedrock profile, the site is classified as Type S_D, "stiff soil profile", according to the 2007 CBC.

The seismic parameters according to the 2007 CBC are summarized in the following table.

2007 CBC - Seismic Parameters		
Site Class	S_C	S_D
Mapped Spectral Acceleration Parameters	S_s = 2.59 and S_1 = 0.97	S_s = 2.59 and S_1 = 0.97
Site Coefficients	F_a = 1.0 and F_v = 1.3	F_a = 1.0 and F_v = 1.5
Adjusted Maximum Considered Earthquake (MCE) Spectral Response Parameters	S_{MS} = 2.59 and S_{M1} = 1.26	S_{MS} = 2.59 and S_{M1} = 1.45
Design Spectral Acceleration Parameters	S_{DS} = 1.72 and S_{D1} = 0.84	S_{DS} = 1.72 and S_{D1} = 0.97

The corresponding value of PGA from the design response spectra according to the 2007 CBC is 0.69g. The value of PGA is independent of site class.

GENERAL SITE GRADING:

It is imperative that no clearing and/or grading operations be performed without the presence of a representative of the geotechnical engineer. An on-site pre-job meeting with the developer, the contractor and the geotechnical engineer should occur prior to all grading related operations. Operations undertaken at the site without the geotechnical engineer present may result in exclusions of affected areas from the final compaction report for the project.

Grading of the subject site should be performed, at a minimum, in accordance with these recommendations and with applicable portions of the CBC. The following recommendations are presented for your assistance in establishing proper grading criteria.

INITIAL SITE PREPARATION:

All areas to be graded should be stripped of significant vegetation and other deleterious materials. These materials should be removed from the site for disposal. Any existing utility lines should be traced, removed, and rerouted from the building area.

If necessary, the abandonment of seepage pits will require that any existing effluent and water be pumped from the pits. Following pumping, any loose and/or organic material that remains in the pits should be removed. The pits should then be backfilled with a one-sack sand slurry mixture to within approximately 6 feet of the finish grade elevation. Following the backfill, the area surrounding the seepage pits should be excavated to a depth of approximately 6 feet below finish grade elevation. The excavation should include all loose material surrounding the pit. In addition, the excavation should allow access for compaction equipment. The excavation should then be backfilled to finish grade elevation as properly compacted fill. Septic tanks, if encountered, should be removed and the excavation filled with properly compacted fill material.

MINIMUM MANDATORY REMOVAL AND RECOMPACTION OF EXISTING SOILS:
All areas to be graded should have at least the upper 24 inches of existing soils removed, and the open excavation bottoms thus created should be observed by our engineering geologist to verify and document in writing that suitable non-compressible sediments or bedrock are exposed prior to refilling with properly tested and documented compacted fill. Deeper removals, such as in the area of Exploratory Boring No. HSA-7, may be necessary. Native subgrade compaction tests can be taken on the removal bottom where appropriate to provide in-place moisture/density data for relative compaction evaluations and to help support and document the engineering geologist's decision. A minimum native subgrade relative compaction of 85 percent (ASTM D 1557) may be utilized as a preliminary basis for suitable removal bottom. The entire site should have any undocumented fills, topsoil, or other unsuitable materials removed, and the entire site should be covered with compacted fill or cuts exposing suitable bedrock.

Cavities created by removal of subsurface obstructions such as structures, individual effluent disposal systems, and trees, should be thoroughly cleaned of loose soil, organic matter and other deleterious materials, shaped to provide access for construction equipment, and backfilled as recommended for site fill.

PREPARATION OF FILL AREAS:
Prior to placing fill, and after the mandatory subexcavation operation and the undocumented fill and loose soils have been removed, the surfaces of all areas to receive fill should be scarified to a depth of 12 inches or more. The scarified soils should be brought to near optimum moisture content and recompacted to a minimum relative compaction of 95 percent in accordance with ASTM D 1557.

PREPARATION OF FOOTING AREAS:
All footings should rest upon at least 18 inches of properly compacted fill material or approved bedrock material. In areas where the required thickness of compacted fill is not accomplished by site rough grading, mandatory subexcavation operation and the undocumented fill and loose soil removal, or

suitable bedrock material is not exposed, the footing areas should be subexcavated to a depth of 18 inches or more below the proposed footing base grade, with the subexcavation extending at least 5 feet beyond the footing line. This subexcavation operation should include a minimum of the upper 24 inches of existing material even though planned filling will be sufficient to satisfy compacted fill thickness requirements. The removal of the upper 24 inches of soil, regardless, is to assist in fill and loose soil identification and revealing buried obstructions. The bottom of this excavation should then be scarified to a depth of at least 12 inches, brought to near optimum moisture content, and recompacted to a minimum of 95 percent relative compaction in accordance with ASTM D 1557 prior to refilling the excavation to grade as properly compacted fill.

In building pad areas where grading results in cuts greater than 4 feet into bedrock material (either Puente Formation or Glendora Volcanics) over the entire building pad area, subexcavation and recompaction may not be necessary, and the proposed structure foundation may be founded into the bedrock material. This should be confirmed by the Engineering Geologist prior to finish grading of the building pad surface.

Should grading result in fill thicknesses that vary by a significant amount, a potential for static differential settlement will exist. As such, it is our recommendation that the thickness of fill not be allowed to vary by more than 50 percent, 10 feet maximum, across any structure area. If fill thicknesses are to vary by more than this amount as a result of grading, it will be necessary to increase the removals in the cut portion of the building pad in order to construct a fill mat with a relatively uniform fill thickness. The "structure area" includes the structure footprint and the zone of influence consisting of a 1(h):1(v) downward projection from 5 feet outside the structure footing. A determination of specific structural pad areas that require additional subexcavation should be performed at the time of grading.

COMPACTED FILLS:

The on-site soils should provide adequate quality fill material, provided they are free from organic matter and other deleterious materials. Unless approved by the geotechnical engineer, rock or similar irreducible material with a maximum dimension greater than 8 inches should not be buried or placed in the upper 10 feet of fill. Rocks with a dimension greater than 8 inches should be placed in accordance with the section of this report titled, "OVERSIZED MATERIAL" below.

Import fill should be inorganic, non-expansive granular soils free from rocks or lumps greater than 8 inches in maximum dimension. Sources for import fill should be observed and approved by the geotechnical engineer prior to their use.

Fill should be spread in near-horizontal layers (lifts), approximately 8 inches thick. Thicker lifts may be approved by the geotechnical engineer if testing indicates that the grading procedures are adequate to achieve the required compaction. Each lift should be spread evenly, thoroughly mixed during spreading to attain uniformity of the material and moisture in each layer, brought to near optimum moisture content and compacted to a minimum relative compaction of 95 percent in accordance with ASTM D 1557.

Based upon the relative compaction of the native bedrock tested during this investigation and the relative compaction anticipated for compacted fill soils, we estimate a compaction shrinkage of approximately 0 to 5 percent. Therefore, 1.00 cubic yards to 1.05 cubic yards of in-place soil material would be necessary to yield one cubic yard of properly compacted fill material. Based upon the relative compaction of the native alluvial and colluvial soils tested during this investigation and the relative compaction anticipated for compacted fill soils, we estimate a compaction shrinkage of approximately 10 to 15 percent. Therefore, 1.10 cubic yards to 1.15 cubic yards of in-place soil material would be necessary to yield one cubic yard of properly compacted fill material. In addition, we would anticipate subsidence of approximately 0.1 feet. These values are exclusive of losses due to stripping, tree removal, or the removal of other subsurface obstructions, if encountered, and may vary due to differing conditions within the project boundaries and the limitations of this investigation.

Values presented for shrinkage and subsidence are estimates only. Final grades should be adjusted, and/or contingency plans to import or export material should be made to accommodate possible variations in actual quantities during site grading.

OVERSIZED MATERIAL:

It is anticipated that significant quantities of oversized material requiring special handling for disposal may be encountered during the grading operation. While site-specific recommendations may be developed during grading plan preparation or in the field during construction, we are providing general methods for disposing of oversized rock on site for preliminary consideration.

Rocks between approximately 8 and 24 inches in size may be placed in areas of fill depth greater than approximately 10 feet below finish grade. Rocks with a maximum dimension greater than 24 inches should not be buried or placed in fill.

The oversized rock should be placed in windrows and adequately spaced to prevent nesting. Sandy matrix material should then be flooded in between the rock to fill any void spaces. Continuous observation of the rock placement and flooding operation should be conducted by the geotechnical engineer. These recommendations for rock disposal are illustrated on the attached Rock Disposal Detail (Appendix "E").

Again, these recommendations are preliminary. Additional recommendations can be provided once the proposed grading is known. In any case, it is crucial that the geotechnical engineer be present to observe these operations. Further recommendations may be made in the field depending on the actual conditions encountered. In addition, applicable local building codes should be reviewed and incorporated into any rock disposal area designs.

SLOPE CONSTRUCTION:

Cut and fill slopes should be constructed no steeper than 2(h):1(v). Fill slopes should be overfilled during construction and then cut back to expose fully compacted soil. A suitable alternative would be to compact the slopes during construction and then roll the final slopes to provide dense, erosion-resistant surfaces.

Where fills are to be placed against existing slopes steeper than 5(h):1(v), the existing slopes should be benched into competent native materials to provide a series of level benches to seat the fill and to remove the compressive and permeable topsoils. The benches should be a minimum of 8 feet in width, constructed at approximately 4-foot vertical intervals. In addition, a shear key should be constructed across the toe of the slope. The shear key should be a minimum of 15 feet wide and should penetrate beneath the toe of the slope a minimum of 2 feet into approved bedrock material or approved firm competent soils.

Where fill over cut slope will occur, the cut portion should be overexcavated and replaced as compacted fill to a distance of at least 15 feet horizontally behind the slope face. Within the cut portion of the slope, the horizontal thickness should not be greater at the top than at the bottom.

A typical shear key and slope benching detail is contained in Appendix "E".

SLOPE CREEP:

The outer, upper portions of cut and fill slopes will be subject to potential long-term movements due to creep or erosion forces. All proposed improvements planned near or on the top of slopes, including garden walls, flatwork, and pools, should be designed and constructed to minimize the effects of this movement. Where possible, improvements should be designed as far from the top of slope as possible. At a minimum, footings should be designed so that there is a least a 5-foot separation from the face of slope to the face of the footing. This may necessitate deepened footings. The actual design of such walls will be based on the wall loading conditions and the earth pressure required to resist these loads. This will fall under the purview of the wall designer who should consult this firm if actual earth pressure information is required.

SLOPE PROTECTION:

Inasmuch as the native materials are susceptible to erosion by wind and running water, it is our recommendation that the slopes at the project be planted as soon as possible after completion. The use of succulent ground covers, such as iceplant or sedum is not recommended. If watering is necessary to sustain plant growth on slopes, then the watering operation should be monitored to assure proper operation of the water system and to prevent over watering.

Measures should be provided to prevent surface water from flowing over slope faces.

Rodent infestation can also be a serious issue with respect to slope stability. Rodent tunneling and burrowing alters the strength of the soil and can allow water to infiltrate the soil, resulting in ultimate slope failure. Rodent burrows can also provide direct access for surface water to the slope face, causing surficial slope "blowouts". Although a maintenance issue, we recommend that measures be taken to prevent rodent infestation in slopes.

SUBDRAINS:

Fill construction will involve placement of relatively permeable fill over bedrock. The result will be conditions conducive to ponding or perching of landscape irrigation water at the fill/bedrock interfaces. Additionally, cuts may also expose perched water, springs, and seeps. Subdrains may be recommended based on conditions observed by the engineering geologist at the time of grading. A typical subdrain design is included in Appendix "E" of this report.

If encountered, springs and seeps in cut areas or areas with a potential for springs and seeps will need evaluation on a case-by-case basis as to the most practical mitigation recommendations. The need for subdrains or alternative mitigation recommendations should be made by the engineering geologist at the time of grading.

SETTLEMENT MONITORING:

All fills greater than 40 feet deep should be monitored. To verify substantial completion of compression of the fill, an initial reading of the settlement monitors should be taken immediately after construction. The fill should then be monitored at least four additional times at an interval determined by this firm for both horizontal and vertical movement. The criteria for a determination of the completion of significant settlement will be established by this firm after analysis of at least five readings. A typical settlement monitor detail is included as Enclosure "E-5". Locations of settlement monitors should be clearly marked and readily visible (red flagged). Clearance should be maintained from heavy equipment operations.

FOUNDATION DESIGN:

In areas where compacted fill is less than 20 feet, or approved bedrock material is exposed and the site is prepared as recommended, the proposed house structures may be safely founded on conventional spread foundations, either individual spread footings and/or continuous wall footings, bearing on a minimum of 18 inches of compacted fill or approved bedrock material. Footings should not be allowed to span from fill or native soil to bedrock material. Footings should be a minimum of 12 inches wide and should be established at a minimum depth of 12 inches below lowest adjacent final subgrade level. For the minimum width and depth, footings may be designed for a maximum safe soil bearing pressure of 1,500 pounds per square foot (psf) for dead plus live loads. This allowable bearing pressure may be increased by 100 psf for each additional foot of width and by 300 psf for each additional foot of depth, to a maximum safe soil bearing pressure of 2,500 psf for dead plus live loads. These bearing values may be increased by one-third for wind or seismic loading.

Footings should be set back from all slopes in accordance with information contained in Appendix "E".

For footings thus designed and constructed on a maximum fill thickness of less than 20 feet, we would anticipate a maximum static settlement of less than 1 inch. Differential settlement between similarly loaded adjacent footings is expected to be approximately half the total settlement.

Should grading result in fill thickness that varies by a significant amount, a potential for static differential settlement will exist. As such, it is our recommendation that the thickness of fill not be allowed to vary by more than 50 percent, 10 feet maximum, across any structure area. If fill thickness is to vary by more than this amount as a result of grading, it will be necessary to increase the removals in the cut portion of the building pad in order to construct a fill mat with a relatively uniform fill thickness. The "structure area" includes the structure footprint and the zone of influence consisting of a 1(h):1(v) downward projection from 5 feet outside the structure footing. A determination of specific structural pad areas that require additional subexcavation should be performed at the time of grading.

In areas where fill depth exceeds 20 feet below the pad elevation, post-tensioned slab or similar foundation system will be necessary for differential settlement.

POST-TENSIONED SLAB FOUNDATIONS (SETTLEMENT):

If the building pad area fill depth exceeds 20 feet below pad grade, the proposed structures may be safely founded on post-tensioned slab foundations. The compacted fill mat should not vary by more than 50 percent, 10 feet maximum, across any structure area. For preliminarily design purposes, post-tensioned slabs may be designed for a maximum differential settlement of 1 inches in 40 feet. Differential and total settlements may be different depending on fill thickness and locations. Further analysis may be performed after the grading and once the structure locations are known. The post-tensioned slab may be designed for a maximum safe soil bearing pressure of 1,500 psf for dead plus live loads.

Utility line connections should be flexible to allow for differential movement.

SLABS-ON-GRADE (non-expansive soil):

Except as note above, to provide adequate support, concrete slabs-on-grade should bear on a minimum of 12 inches of compacted soil. Concrete slabs-on-grade should be a minimum of 4 inches in thickness. The soil should be compacted to 95 percent relative compaction. The final pad surfaces should be rolled to provide smooth dense surfaces.

Slabs to receive moisture-sensitive coverings should be provided with a moisture vapor retarder. We recommend that a vapor retarder be designed and constructed according to the American Concrete Institute (ACI) 302.1R, Concrete Floor and Slab Construction, which addresses moisture vapor retarder construction. At a minimum, the vapor retarder should comply with ASTM E 1745 and have a nominal thickness of at least 10 mils. The vapor retarder should be properly sealed per the manufacturer's recommendations and protected from punctures and other damages. One inch of sand under the vapor retarder may assist in reducing punctures.

A modulus of vertical subgrade reaction of 150 pounds per cubic inch can be utilized in the preliminary design of slabs-on-grade for the proposed project. Further evaluation of the subgrade soils after completion of the grading could be performed.

EXPANSIVE SOILS:

E.I. tests (ASTM D 4829) indicate that selected soils encountered during this investigation have a "very low" to "high" expansion potential. The CBC specifies that structures constructed on soils with expansion indices greater than 20 require special design consideration.

The 2007 CBC presents two methods that may be used in design of slab-on-ground foundations. These methods are based on: 1) Design of Slab-On-Ground Foundations of the Wire Reinforcement Institute, Inc., and 2) Standard Requirements for Design of Shallow Post-Tensioned Concrete Foundations on Expansive Soils of the Post-Tensioning Institute. The Structural Engineer should provide appropriate designs of foundations and slabs-on-ground to accommodate soils with expansive characteristics. During and after the grading operation, additional expansion and Atterberg limit testing should be performed to provide final design recommendations for expansive soil on a lot-by-lot basis.

As an alternative, removal and replacement of the expansive soil or mixing of the expansive soil with on-site or imported non-expansive material to lower it's E.I. to less than 20 may be performed. The depth of removal and replacement or mixing of the expansive soil below the proposed foundation system should be sufficient to ensure a constant moisture content in the remaining fill. Additional evaluation of soils for expansion potential should be conducted by the geotechnical engineer during the grading operation.

It should be noted that due to the expansion of the clayey soils on the site, the ground will be subjected to shrinkage cracking. This shrinkage cracking tends to be more of an aesthetic condition than a structural problem, since below the building foundations moisture tends to be retained and kept relatively constant. However, sidewalks and other associated flatwork areas may be subjected to heave/shrinkage cycles if underlying soils and adjacent ground are not kept at a constant moisture.

CONCRETE FLATWORK:

Additional evaluation of soils for expansion potential should be conducted by the geotechnical engineer during the grading operation. Where expansive soils conditions are identified, they may adversely affect areas of portland cement concrete (PCC) flatwork such as sidewalks, driveways, curbs, and other non structural pavement areas. PCC flatwork on expansive soil may require special geotechnical or structural design considerations to accommodate the effects of expansion.

For structural building slabs in areas with expansive soils, we have provided recommendations for post-tensioned slab or slab-on-ground foundations design; however, post-tensioned slab or slab-on-ground foundations are not practical for concrete flatwork. As such, we are including the following general recommendations for concrete flatwork on expansive soil.

Geotechnical Methods of Mitigation:

> If a granular non-expansive soil is to be imported, the weighted E.I. method outlined in the CBC could be utilized to incrementally decrease the potential effects of expansive soils. The expansive effects can be reduced to a level of insignificance by supporting the flatwork on a minimum of 36 inches of granular non-expansive material.

> The expansive soils should be pre-saturated to a depth of 24 inches at least 7 days prior to placement of concrete. The pre-saturation should be to at least 5 percent above optimum moisture content.

> The expansive soils should be protected from moisture fluctuations to the extent practical. This may involve such factors as providing positive drainage away from the flatwork, avoidance of adjacent landscaping (especially trees) requiring irrigation, or perhaps placement of impermeable membranes. Irrigation pipes should not be placed near flatwork and must be properly maintained in order to avoid distress related to leaks and rupture. Landscape areas should slope away from the flatwork and structural areas by at least 3 percent. All surface water runoff must be diverted away from the margins of flatwork and structural areas and directed into paved roadways or appropriate drainage features.

Structural Methods of Mitigation:

> All flatwork should be designed to resist the effects of expansion. We are providing what we consider typical recommendations. The actual design including reinforcement should be provided by the structural or civil engineer.

All concrete flatwork subject to the effects of expansive soils should be a minimum of 4 inches in thickness and reinforced by utilizing a minimum of 6×6 W1.4/1.4 wire mesh or #3 Bars at 14 inches each way at mid-height. Curbing should contain at least one #4 Bar continuous top and bottom.

Where the flatwork abuts structures or adjacent flatwork, the flatwork should be doweled into the adjacent structure to avoid differential elevation. The dowels should be smooth and either wrapped or lubricated on one end to prevent bonding and allow for movement. In addition, felt or similar material should be placed between adjacent slab edges.

It should be cautioned that some distress to concrete flatwork may occur in spite of the measures taken to mitigate the effects. However, the distress will be lessened by incorporating as many of the above measures as practical into the design and construction of the flatwork. The costs of these preventative measures should be weighed against the costs of future repairs and maintenance.

LATERAL LOADING:

Resistance to lateral loads will be provided by passive earth pressure and base friction. For footings bearing against compacted fill, passive earth pressure may be considered to be developed at a rate of 250 psf per foot of depth. Base friction may be computed at 0.25 times the normal load. Base friction and passive earth pressure may be combined without reduction. These values may be increased by one-third for wind or seismic loading.

Other than conservative soil modeling, the lateral passive earth pressure and base friction values recommended do not include factors of safety. If the design is to be based on allowable lateral resistance values, we recommend that minimum factors of safety of 1.5 and 2.0 be applied to the friction coefficient and passive lateral earth pressure, respectively. The resulting allowable lateral resistance values are:

Base friction coefficient:	0.17
Passive lateral earth pressure:	125 psf per foot of depth

EARTH PRESSURES:

For preliminary retaining wall design, we recommend the following lateral earth pressure coefficients and equivalent fluid pressures for retaining wall design. These pressures are based upon the backfill soil having an E.I. of less than 20 and selected granular material with a shear angle of 34 degrees and cohesion of 0 psf within the area of influence created by a 1(h):1(v) slope starting from the bottom and continuing up from the heel portion of the retaining wall foundation.

At Rest:	level	-	$K_0 = 0.44$	$P_0 = 57$ pcf
	3:1	-	$K_0 = 0.60$	$P_0 = 78$ pcf
	2:1	-	$K_0 = 0.69$	$P_0 = 90$ pcf
Active:	level	-	$K_a = 0.28$	$P_a = 37$ pcf
	3:1	-	$K_a = 0.35$	$P_a = 46$ pcf
	2:1	-	$K_a = 0.42$	$P_a = 54$ pcf

The "at-rest" condition applies toward braced walls which are not free to tilt. The "active" condition applies toward unrestrained cantilevered walls where wall movement is anticipated. The structural designer should use judgement in determining the wall fixity and may utilize values interpolated between the "at-rest" and "active" conditions where appropriate.

These values should be verified prior to construction when the backfill materials and conditions have been determined. These values are applicable only to properly drained backfill with no additional surcharge loadings and do not include a factor of safety other than conservative modeling of the soil strength parameters. If import material is to be utilized for backfill, an engineer from this firm should verify the backfill has equivalent or superior strength values. Toe bearing pressure for walls on soils not bearing against compacted fill as described earlier in the section of this report titled, "PREPARATION OF FOOTING AREAS" should not exceed CBC values. Soils with an E.I. greater than 20 should not be utilized for wall backfill within the area of influence created by a 1(h):1(v) slope starting from the bottom and continuing up from the heel portion of the retaining wall foundation.

For walls with uniform surcharge loading and backfilled with selected granular material, the increase in active pressure can be calculated as the product of 0.28 and the surcharge load, q, (i.e., $0.28 \times q$) for level backfill. The increase in at-rest pressure can be calculated as the product of 0.44 and the surcharge load, q. The resulting additional surcharge pressure should be applied to the wall as a rectangular distribution, from top to bottom.

Backfill behind retaining walls should consist of a soil of sufficient granularity that the backfill will properly drain. The granular soil should be classified per the USCS as either a GW, GP, SW, SP, SW-SM, or SP-SM. Surface drainage should be provided to prevent ponding of water behind walls. A drainage system should be installed behind all retaining walls consisting of any of the following:

1. A 4-inch diameter perforated PVC (Schedule 40) pipe or equivalent at the base of the stem encased in 2 cubic feet of granular drain material per linear foot of pipe or

2. Synthetic drains such as Enkadrain, Miradrain, Hydraway 300, or equivalent.

Perforations in the PVC pipe should be 3/8-inch diameter. Granular drain material should be wrapped with filter cloth such as Mirafi 140 or equivalent to prevent clogging of the drains with fines. Walls should be waterproofed to prevent nuisance seepage. Water should outlet to an approved drain.

Foundation concrete should be placed in neat excavations with vertical sides, or the concrete should be formed and the excavations properly backfilled as recommended for site fill.

SEISMIC EARTH PRESSURE:

The active seismic earth pressure acting on a retaining wall backfilled with selected granular material was evaluated using the Mononobe-Okabe method. The pseudo-static horizontal acceleration coefficient (K_h) was assumed to be one-half of PGA (0.69g). The pseudo-static vertical acceleration coefficient (Kv) was taken as half of K_h. For retaining walls with selected granular material as backfill, a unit weight of 130 pcf and a friction angle of 34 degrees were used in the calculation. These values should be verified prior to construction when the backfill materials and conditions have been determined and are applicable only to level, properly drained backfill with no additional surcharge loadings.

An inverted triangular distribution of lateral earth pressure of the *seismic component only* was determined as 31 psf/ft for level backfill.

TRENCH EXCAVATION:

Unless specifically evaluated by our engineering geologist, all trench excavations should be performed following the recommendation of CAL/OSHA (California, State of, 2001) Type "C" soil in accordance with the CAL/OSHA (California, State of, 2001) excavation standards. Based upon a soil classification of Type "C", the temporary excavation should not be inclined steeper than 1.5(h):1(v) for maximum trench depth of less than 20 feet. For trench excavation deeper than 20 feet or for conditions that differ from those described for Type "C" in the CAL/OSHA excavation standards, this firm should be contacted.

TRENCH BEDDING AND BACKFILLS:

Trench Bedding - Pipe bedding material should meet and be placed according to the current edition of the Standard Specifications for Public Works Construction "Greenbook" or other project specifications. Pipe bedding should be uniform free-draining granular material with a sand equivalent of at least 30. Our investigation indicates that, for a majority of the soils encountered, the sand equivalent will be less than 30. Based upon these results, the on-site soils are generally not suitable for pipe bedding

The "Greenbook" specifies a minimum sand equivalent of 15 for trench wall materials with pipe bedding to be densified by jetting. Due to the density of the bedrock material, we do not recommend the use of jetting for densification of the bedding material. Water may accumulate in the trench due to the low permeability of the trench wall material, allowing the pipe to "float" and/or cause difficulty in attaining compaction in the soils placed on the bedding material.

As an alternative, a controlled low strength material (CLSM) could be considered for use as pipe bedding or any other areas which would be difficult to properly backfill, if encountered.

Backfill - The on-site soils should provide quality backfill material, provided they are free from organic matter and other deleterious materials. Unless approved by the geotechnical engineer, rock or similar irreducible material with a maximum dimension greater than 6 inches should not be buried or placed in backfills.

Fill to be compacted by heavy equipment should be spread in near-horizontal layers, approximately 8 inches in thickness. For fill to be compacted by hand-operated equipment, thinner lifts, 4 to 6 inches in thickness, should be utilized. Each lift should be spread evenly, brought to near optimum moisture content, and compacted to a minimum relative compaction of 95 percent in accordance with ASTM D 1557. To avoid pumping, backfill material should be mixed and moisture treated outside of the excavation prior to lift placement in the trench.

Soils required to be compacted to at least 95 percent relative compaction, such as street subgrade, should be moisture treated to near optimum moisture content not exceeding 1 percent above optimum.

A CLSM could be considered to fill any cavities, such as voids created by caving or undermining of soils beneath existing improvements or pavement to remain, or any other areas which would be difficult to properly backfill, if encountered. A suggested specification for CLSM is presented below.

Controlled Low Strength Material (CLSM):
CLSM shall consist of a fluid mixture of aggregate, cement, fly ash and water.

Aggregate shall comply with the requirements of ASTM C33. Recycled material or soil shall not be used. The maximum size of the aggregate shall be 3/8 inch.

Cement shall comply with the requirements of ASTM C150 and shall be Type II or V.

Fly ash shall comply with the requirements of ASTM C618 and shall be Class F fly ash.

Water shall be free from oil, salts and other impurities which would have an adverse effect on the quality of the CLSM.

The materials shall be proportioned by weight and mixed in accordance with ASTM C94. The CLSM mixture shall be sufficiently fluid to allow it to flow around piping and supports.

The CLSM mixture shall contain a minimum of 50 pounds and a maximum of 125 pounds of cement per cubic yard. The CLSM mixture shall contain a minimum of 225 pounds of combined cement and fly ash per cubic yard.

The CLSM shall be placed in a uniform manner that will prevent voids in, or segregation of CLSM and that will prevent any shifting or movement of the pipe and will not float the pipe. Pipe supports and holddowns may be required to keep the pipe in position. CLSM dams or earthplugs may be required to facilitate filling of the pipe excavation.

Backfilling over or placing any material over the CLSM shall not commence until the CLSM has sufficiently stiffened to support the backfilling operation without pumping or displacement, but in no case shall backfilling or material placement commence less than 24 hours after CLSM placement unless a preapproved accelerator is utilized.

SHORING DESIGN PARAMETERS:

Resistance to lateral shoring loads will be provided by "passive" earth pressure. Driving lateral loads will be exerted by "active" earth pressures for cantilevered or unbraced conditions and by "at-rest" earth pressures for braced conditions. These values do not include a factor of safety other than conservative modeling of the soil strength parameters. The shoring design engineer should use judgement to determine the appropriate pressure types and distributions. All three pressure distributions are given below as a function of depth (triangular).

Based upon a cohesion of 250 psf and a friction angle of 22 degrees, as conservatively modeled from our relatively undisturbed direct shear tests and based upon anticipated soil unit weights of 135 pounds per cubic foot (pcf), we recommend the following lateral earth pressure coefficients and distributions.

At Rest:	level	-	$K_0 = 0.63$	$P_0 = 81$ pcf
Active:	level	-	$K_a = 0.45$	$P_a = 59$ pcf
Passive:			$K_p = 2.20$	$P_p = 286$ pcf

A minimum rectangular pressure distribution of 125 psf should be applied over the entire depth of the trench to account for any traffic surcharge. Other surcharges may be treated as additional trench height by assuming 1 additional foot of height for each 125 psf of areal surcharge.

POTENTIAL EROSION:

The potential for erosion should be mitigated by proper drainage design. Water should not be allowed to flow over graded areas or natural areas so as to cause erosion. Graded areas should be planted or otherwise protected from erosion by wind or water.

CHEMICAL/CORROSIVITY TESTING:

Selected samples of materials were delivered to Schiff Associates, Inc. for soil corrosivity testing. Laboratory testing consisted of pH, resistivity, and major soluble salts commonly found in soils. The results of the laboratory tests performed by Schiff Associates, Inc. appear in Appendix "C".

These tests have been performed to screen the site for potentially corrosive soils. Although C.H.J., Incorporated does not practice corrosion engineering, values from the soil tested are considered potentially "mildly" corrosive to ferrous metals at as-received condition and "severely" corrosive at saturated condition. Specific corrosion control measures, such as coating of the pipe with non-corrosive material or alternative non-metallic pipe material, are considered to be needed if there is a potential for saturated soils.

Ammonium and nitrate levels did not indicate a concern as to corrosion of buried copper.

Results of the soluble sulfate testing indicate a "negligible" anticipated exposure to sulfate attack. Based upon the criteria from Table 4.3.1. of the American Concrete Institute Manual of Concrete Practice (2000), no special measures, such as specific cement types, water-cement ratios, etc., will be needed for this "negligible" exposure to sulfate attack. Since the Puente Formation on-site is known to include gypsum crystals, additional sulfate testing of building pads should be conducted prior to construction.

The soluble chloride content of the soils tested was not at levels high enough to be of concern with respect to corrosion of reinforcing steel. The results should be considered in combination with the soluble chloride content of the hardened concrete in determining the effect of chloride on the corrosion of reinforcing steel.

C.H.J., Incorporated does not practice corrosion engineering. If further information concerning the corrosion characteristics, or interpretation of the results submitted herein, are required, then a competent corrosion engineer could be consulted.

CONSTRUCTION OBSERVATION:

All grading operations, including site clearing and stripping, should be observed by a representative of the geotechnical engineer. The presence of the geotechnical engineer's field representative will be for the purpose of providing observation and field testing, and will not include any supervising or directing of the actual work of the contractor, his employees or agents. Neither the presence of the geotechnical engineer's field representative nor the observations and testing by the geotechnical engineer shall excuse the contractor in any way for defects discovered in his work. It is understood that the geotechnical engineer will not be responsible for job or site safety on this project, which will be the sole responsibility of the contractor.

LIMITATIONS

C.H.J., Incorporated has striven to perform our services within the limits prescribed by our client, and in a manner consistent with the usual thoroughness and competence of reputable geotechnical engineers and engineering geologists practicing under similar circumstances. No other representation, express or implied, and no warranty or guarantee is included or intended by virtue of the services performed or reports, opinion, documents, or otherwise supplied.

This report reflects the testing conducted on the site as the site existed during the investigation, which is the subject of this report. However, changes in the conditions of a property can occur with the passage of time, due to natural processes or the works of man on this or adjacent properties. Changes in applicable or appropriate standards may also occur whether as a result of legislation, application, or the broadening of knowledge. Therefore, this report is indicative of only those conditions tested at the time of the subject investigation, and the findings of this report may be invalidated fully or partially by changes outside of the control of C.H.J., Incorporated. This report is therefore subject to review and should not be relied upon after a period of one year.

The conclusions and recommendations in this report are based upon observations performed and data collected at separate locations, and interpolation between these locations, carried out for the project and the scope of services described. It is assumed and expected that the conditions between locations observed and/or sampled are similar to those encountered at the individual locations where observation and sampling was performed. However, conditions between these locations may vary significantly. Should conditions be encountered in the field, by the client or any firm performing services for the client or the client's assign, that appear different than those described herein, this firm should be contacted immediately in order that we might evaluate their effect.

If this report or portions thereof are provided to contractors or included in specifications, it should be understood by all parties that they are provided for information only and should be used as such.

The report and its contents resulting from this investigation are not intended or represented to be suitable for reuse on extensions or modifications of the project, or for use on any other project.

CLOSURE

We appreciate this opportunity to be of service and trust this report provides the information desired at this time. Should questions arise, please do not hesitate to contact this office.

Respectfully submitted,
C.H.J., INCORPORATED

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Fairchild photograph collection, Whittier College, California, March 20, 1933, flight no. C-2550, black and white aerial photograph nos. A:3-6, 28-31, 56-57, scale 1" = 1,320'.

Fairchild photograph collection, Whittier College, California, June 18, 1934, flight no. C-3064, black and white aerial photograph nos. 197-201, scale 1" = 800'.

Fairchild photograph collection, Whittier College, California, June 11, 1936, flight no. C-4061, black and white aerial photograph nos. 6, 43-44, scale 1" = 2,000'.

Fairchild photograph collection, Whittier College, California, February 1949, flight no. C-13373, black and white aerial photograph nos. 1:6-7, scale 1" = 2,000'.

Fairchild photograph collection, Whittier College, California, July 1949, flight no. C-13990, black and white aerial photograph nos. 3:44-46, scale 1" = 1,500'.

Fairchild photograph collection, Whittier College, California, June 1949, flight no. C-13990X, black and white aerial photograph nos. 1:183-185, scale 1" = 1,500'.

Fairchild photograph collection, Whittier College, California, June 4, 1950, flight no. C-15200, black and white aerial photograph nos. 4:110-112, scale 1" = 400'.

Fairchild photograph collection, Whittier College, California, August 15, 1952, flight no. C-18537, black and white aerial photograph nos. S:3, scale 1" = 3,333'.

Fairchild photograph collection, Whittier College, California, 1953, flight no. C-19400, black and white aerial photograph nos. 3:89-90, scale 1" = 5,280'.

Fairchild photograph collection, Whittier College, California, July 12, 1954, flight no. C-20645, black and white aerial photograph nos. 3:10-11, 14-15, 44-45, scale 1" = 800'.

Fairchild photograph collection, Whittier College, California, 1958, flight no. C-23023, black and white aerial photograph nos. LA:3:23-25; 4:28-29, scale 1" = 3,000'.