PRELIMINARY GEOTECHNICAL
INTERPRETIVE REPORT
PROPOSED COMMERCIAL DEVELOPMENT
ASSESSOR'S PARCEL NUMBER 377-160-014
SOUTH CORNER OF CHANEY & WEST MINTHORN STREETS
CITY OF LAKE ELSINORE
RIVERSIDE COUNTY, CALIFORNIA

PROJECT NO. 18679-10
FEBRUARY 27, 2019

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February 27, 2019

Project No. 18679-10

Mr. Rod Oshita
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Subject: **Preliminary Geotechnical Interpretive Report, Proposed Commercial Development, Assessor's Parcel Number 377-160-014, South Corner of Chaney & West Minthorn Streets, City of Lake Elsinore, Riverside County, California**

In accordance with your request, CW Soils is pleased to present our preliminary geotechnical interpretive report for the proposed Commercial Development, Assessor's Parcel Number 377-160-014, located on the south corner of Chaney Street and West Minthorn Street in the City of Lake Elsinore, Riverside County, California. Our services were completed in accordance with the scope of work described in our proposal, dated March 23, 2018. The purpose of our work was to evaluate the nature, distribution, and engineering properties of the geologic formations underlying the site with respect to the proposed improvements.

CW Soils appreciates the opportunity to offer our services on this project. If we can be of further assistance, please do not hesitate to contact the undersigned at your convenience.

Respectfully submitted,

CW Soils

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INTRODUCTION

This report prepared by CW Soils, presents the preliminary interpretive geotechnical evaluation for the proposed improvements. The purpose of our work was to evaluate the nature, distribution, and engineering properties of the geologic formations underlying the site with respect to the proposed improvements. Furthermore, we have included grading and foundation design recommendations based on the information you provided.

SITE DESCRIPTION

The site is located on the south corner of Chaney Street and West Minthorn Street in the City of Lake Elsinore, Riverside County, California. The subject property is surrounded by commercial developments and a school. The general location of the subject property is illustrated on Figure 1 – Vicinity Map.

The subject property consists of undeveloped land with relatively flat terrain. Topographic relief at the subject property is relatively low. Vegetation at the site includes sparse amounts of annual weeds/grasses, along with some small to large trees near the perimeters.

PROPOSED DEVELOPMENT

Based on our understanding of the proposed project, three (3) buildings positioned throughout the site are planned. The proposed commercial development is anticipated to consist of wood, concrete, or steel framed one- and/or two-story structures utilizing slab on grade construction with associated streets, landscape areas, and utilities.

Formal plans have not been prepared and await the conclusions and recommendations of this report.

FIELD EXPLORATION AND LABORATORY TESTING

Field Exploration

Subsurface exploration at the subject property was performed on April 12, 2018. A truck mounted hollow-stem-auger drill rig was mobilized to advance six (6) borings throughout the project area to a maximum depth of 50.5 feet.

Classification and logging of the soils encountered during the field exploration were performed in general accordance with the Standard Practice for Description and Identification of Soils (Visual-Manual Procedure) of ASTM D 2488. Earth material descriptions may have been reconciled to reflect laboratory test results with regard to ASTM D 2487 or re-examination in the laboratory. Descriptive logs and the Log Symbols & Terms explanation sheet are presented in Appendix B.

REFERENCE: Google Earth (Version 7.1.2.2041) [Software]. Mountain View, CA: Google Inc. (2013).

CW SOILS	VICINTY MAP	14396-10
		Not to Scale
		FIGURE 1

Associated with the subsurface exploration was the collection of disturbed bulk samples and/or relatively undisturbed samples of soils for laboratory testing and analysis. Relatively undisturbed samples were obtained with a 3.0 inch outside diameter (2.43 inch inside diameter) modified California split-spoon sampler lined with 1 inch high brass rings. A Standard Penetration Test (N) split-spoon sampler was utilized to obtain penetration resistance and samples as needed. Samples obtained using a hollow stem auger drill rig, were mechanically driven with successive 30 inch drops of a 140-pound automatic trip safety hammer. The blow counts required to drive the sampler the final 12 inches of an 18 inch drive were recorded in the boring logs. The deepest recovered portion of the driven samples were placed in sealed containers and transported to the laboratory for testing and analysis. The exploratory locations and geologic conditions at the subject property are illustrated on Plate 1 – Geotechnical Map.

Laboratory Testing

Maximum dry density/optimum moisture content, expansion potential, pH, resistivity, sulfate content, chloride content, and in-situ density/moisture content were determined for selected samples of soils, considered representative of those noted during the field exploration. The laboratory test results are reflected throughout the Conclusions and Recommendations of this report. Summaries of the test results and brief descriptions of laboratory test criteria are presented in Appendix C.

FINDINGS

Regional Geology

Regionally, the project is located in the Peninsular Ranges Geomorphic Province of California. The Peninsular Ranges are characterized by northwest trending sediment filled elongated valleys divided by steep mountain ranges. Associated with and subparallel to the northwest trending San Andreas Fault, are the San Jacinto Fault, Newport-Inglewood Fault, and the Whittier-Elsinore Fault zones. The northwest trend of the province has played a major role in shaping the dominant structural geologic features in the region as well. The Perris Block forms the eastern boundary of the Elsinore Fault, while the west side is comprised of the Santa Ana Mountains. The Perris Block is in turn bounded to the east by the San Jacinto Fault. The Peninsular Ranges Province and the Transverse Range Province are separated by the northern perimeter of the Los Angeles basin, which is formed by a northerly dipping blind thrust fault.

The low lying areas within the Peninsular Ranges Province are principally made up of Tertiary and Quaternary non-marine alluvial sediments consisting of alluvial deposits, sandstones, claystones, siltstones, conglomerates, and occasional volcanic units. The mountainous regions are primarily made up of Pre-Cretaceous, metasedimentary, and metavolcanic rocks along with Cretaceous plutonic rocks of the Southern California Batholith. A map illustrating the regional geology is presented on Figure 2 – Regional Geologic Map.

Local Geology

The most relevant local geologic units expected to be present at the site are summarized in this section. A general description of the dominant soils that form the geologic units is provided below:

- Artificial Fill, Undocumented (map symbol Afu): Undocumented artificial fill materials were encountered throughout the site within the upper 3 to 7 feet during exploration. These materials are typically locally derived from the native materials and consist generally of light brown to dark brown sandy clay and clayey sand in a slightly moist to moist, and soft to medium stiff state. These materials are generally inconsistent, poorly consolidated fills.

- Quaternary Alluvium (map symbol Qal): Quaternary alluvium was encountered to a maximum depth of 50 feet. These alluvial deposits consist predominately of interlayered moderate yellowish brown to grayish brown, fine to coarse grained silty sand, clayey sand, sandy clay, sandy silt, and occasional clay. These deposits were generally noted to be in a slightly moist to wet, loose to very dense state.

- Cretaceous Heterogeneous Granitic Rocks (map symbol Khg): Cretaceous age granitic rocks composed of a wide variety of compositions make up this unit. Rock types typically include monzogranite, granodiorite, tonalite and gabbro, with the most common being tonalite (Morton, 2004). This rock unit was encountered at depth below the site. These granitic rocks were observed to be yellowish brown and in a moderately hard to hard state.

Aerial Photographs

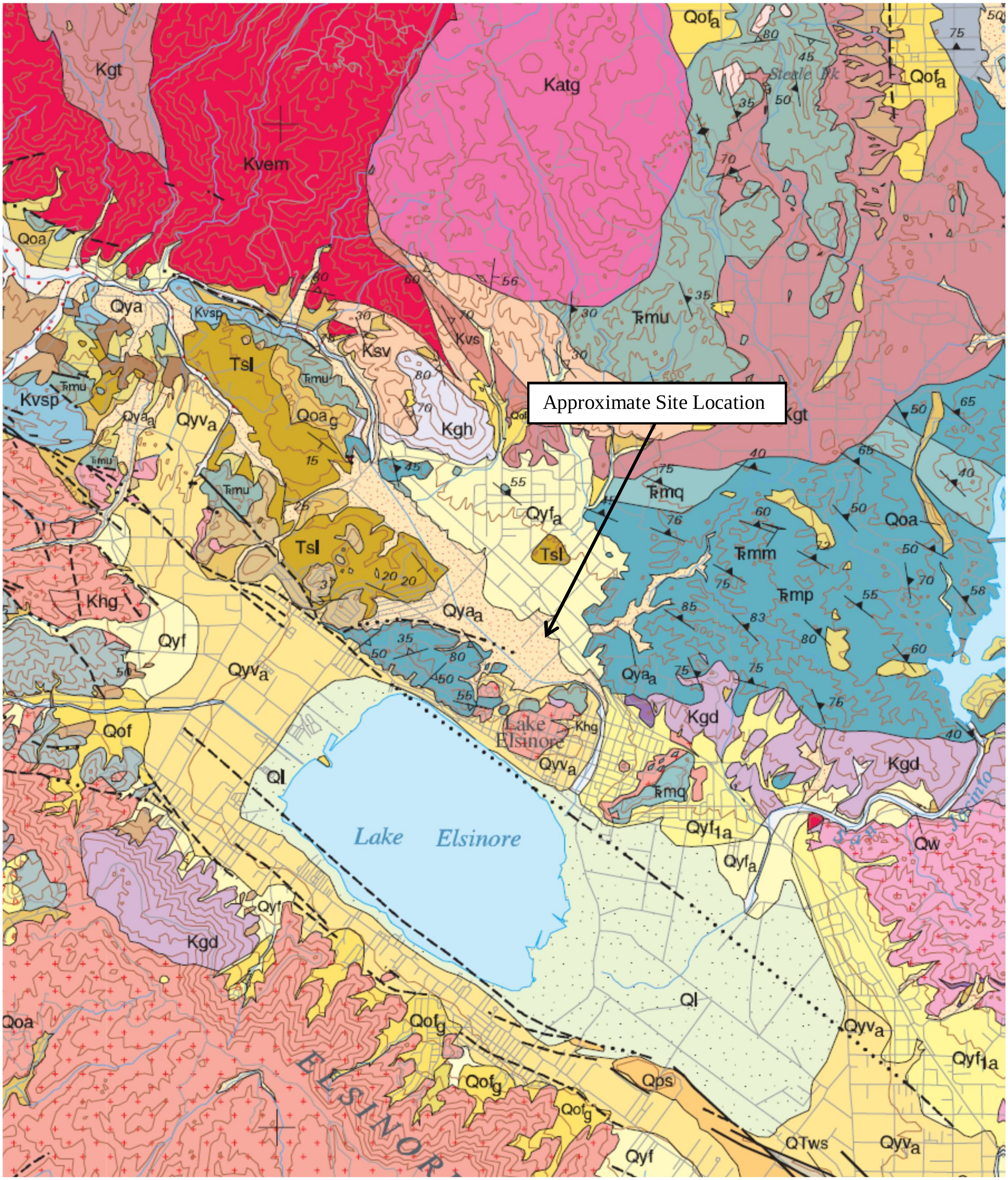
A review of aerial photographs was performed during our geotechnical evaluation. No strong geomorphic expressions suggestive of recent faulting, such as linear topography, offset streams/drainage courses, lines of natural springs, or fault scarps, were interpreted to project through the proposed project area during our review of the aerial photographs of the subject property. Aerial photographs from different time periods and various scales that were utilized in our geomorphic interpretations include the following from Google Earth dated May 1994, May 2002, October 2005, June 2009, June 2012, April 2014, and February 2018.

Faulting

Significant ground shaking will likely impact the site within the design life of the proposed project, due to the project being located in a seismically active region. The geologic structure of the entire southern California area is dominated by northwest-trending faults associated with the San Andreas Fault system. The San Andreas Fault system accommodates for most of the right lateral movement associated with the relative motion between the Pacific and North American tectonic plates.

The subject property is not located within an Alquist-Priolo Fault Rupture Hazard Study Zone, established by the State of California to restrict the construction of habitable structures across identifiable traces of known active faults. No active faults are known to project through the proposed project. As defined by the State of California, an active fault has undergone surface displacement within the past 11,700 years or during the Holocene epoch.

Based upon our understanding of the site and our analysis using the referenced software (USGS Earthquake Hazards Program, Unified Hazard Tool for Conterminous U.S. 2014 (v4.1.1) Deaggregation), the Elsinore Fault with an approximate source to site distance of 2.69 kilometers is the closest known active fault anticipated to produce the highest horizontal ground accelerations, having an estimated maximum modal magnitude of 7.36.



Reference: Morton, D.M., Hauser, Rachel M., and Ruppert, Kelly R., 2004, *Preliminary Digital Geologic Map of the Santa Ana 30' x 60' Quadrangle, Southern California, Version 2.0*: U.S. Geological Survey Open-File Report 99-0172

CW SOILS	REGIONAL GEOLOGIC MAP	14396-10
		Not to Scale
		FIGURE 2

CONCLUSIONS AND RECOMMENDATIONS

General

From a geotechnical point of view, the subject property is considered suitable for the proposed improvements, provided the design information and conclusions and recommendations herein are incorporated into the plans and are implemented during construction.

Geologic and soils engineering issues requiring special attention during planning and construction include temporary excavations near property lines. Recommended removals are on the order of 16 to 20 feet deep, and as a result the excavations should extend beyond the proposed building perimeters an equivalent horizontal distance. As such, we recommend the buildings be moved away from the property lines or costly shoring will be necessary.

Additionally, two layers of Tensar TX 160 or equivalent should be placed within the compacted fill at approximately 10 and 12 feet below grade. The conclusions and recommendations provided herein have been developed through diligent methodology to help mitigate potential geologic and soils engineering issues affecting the proposed improvements.

The site soils have significant clay content with MEDIUM expansion potential and therefore stormwater should not purposely or accidentally be introduced into subgrade soils proximate to building foundations or other surface improvements.

Earthwork

Grading Operations

Grading operations are subject to the provisions of the 2016 California Building Code (CBC), including Appendix J Grading, as well as all applicable grading codes and requirements of the appropriate reviewing agency. Grading operations should also be conducted in accordance with applicable requirements of our General Earthwork and Grading Specifications within the final appendix of this report, unless more conservative recommendations are provided herein.

Clearing and Grubbing

Areas undergoing grading operations should be stripped of vegetation including trees, grasses, weeds, brush, shrubs, or any other debris and properly disposed of offsite. Laborers should be employed to remove roots, branches, or other deleterious materials during grading operations.

CW Soils should be notified in a timely manner in order to provide observations during Clearing and Grubbing operations. Any buried foundations or unanticipated conditions should be brought to our immediate attention to consider whether adjustments are necessary.

Excavation Characteristics

Based on our experience with similar projects in similar settings, the near surface soils, will be readily excavated with conventional earth moving equipment appropriately selected for the task to be performed.

Groundwater

Groundwater was observed in Borings 1, 2, and 5 at depths ranging from 19.5 to 23 feet below existing grade. It should be noted that localized groundwater or variations in the level of groundwater could be discovered during grading due to the limited number of exploratory locations or other factors.

Based on experience working in the area of the subject project, which includes design and construction of the local Target Building (groundwater approximately 5-7 feet), Fairway commercial properties (groundwater approximately 7-10 feet), borings on a project (C.W. Soils, 2015) approximately 400 feet away (groundwater approximately 15.5-18 feet), and the current groundwater level at the site (groundwater approximately 19.5-23 feet). Our best interpretation of this data is that the historic groundwater at the site would have likely fluctuated into the less than 10-foot deep range.

Ground Preparation

Removal excavations should be verified by the project engineer, geologist or their representative. Prior to placing compacted fills, the exposed bottom should be scarified to a depth of 6 inches or more, watered or air dried as necessary to achieve near optimum moisture content and then compacted to a minimum of 90 percent of the maximum dry density as determined by ASTM D1557-12.

Remedial grading should extend horizontally beyond the perimeter of the proposed structures a distance equal to the depth of compacted fill below the proposed footing or a minimum of 5 feet, whichever is greater. The anticipated removal depths are shown on Plate 1 – Geotechnical Map. In general the anticipated removal depths should vary from 16 to 20 feet below existing grade.

Additionally, two layers of Tensar TX 160 or equivalent should be placed within the compacted fill at approximately 10 and 12 feet below grade. The conclusions and recommendations provided herein have been developed through diligent methodology to help mitigate potential geologic and soils engineering issues affecting the proposed improvements.

Wet Removals

Should removals of wet alluvial soils be required, special grading equipment and procedures can reduce overall costs. Careful planning by an experienced grading contractor can minimize the need for special equipment, such as swamp cats, draglines, excavators, pumps, and top loading earthmovers. Possible methods of obtaining bottom stabilization may include the placement of imported angular rock and/or geotextile ground reinforcement. Areas should be set aside for drying and mixing of wet materials with dry materials to reduce the moisture content prior to placing as compacted fill. Specific recommendations can be provided based on the actual conditions encountered.

Oversize Rock

Minor quantities of oversize rock (i.e., rock exceeding a maximum dimension of 12 inches) are expected to be encountered during grading. Oversize rock that is encountered should be disposed of offsite, dispersed throughout the site at the surface of natural grades, or stockpiled and crushed for future use. The disposal of oversize rock is discussed in greater detail in the last appendix of this report, General Earthwork and Grading Specifications.

Compacted Fill Placement

Well mixed soils should be placed in 6 to 8 inch maximum (uncompacted) lifts, watered or air dried as necessary to achieve uniform near optimum moisture content and then compacted to a minimum of 90 percent of the maximum dry density as determined by ASTM D1557-12.

Import Soils

If needed to achieve final design grades, all potential import materials should be non-expansive, free of deleterious/oversize materials, and approved by the project soils engineering consultant prior to delivery onsite.

Fill Slopes

Fill slopes higher than 5 feet and steeper than 5:1 (h:v) require a keyway at the toe. Keyways should be excavated 2 feet into competent earth materials, as measured on the downhill side and be a minimum of 10 feet wide. Backcuts for keyway excavations should be cut no steeper than 1:1 or as recommended by the soils engineer or engineering geologist. As compacted fill is placed, proper benching into competent earth materials should be maintained.

Temporary Backcuts

With regard to excavation safety, it is the responsibility of the grading contractor to follow all Cal-OSHA requirements. Adequate slope stability to protect adjacent developments must be maintained, temporary backcuts for canyon removals, stabilization fills, and/or keyways may be needed. It is imperative that grading schedules minimize the exposure time of the unsupported excavations. Temporary backcuts should be observed by the engineering geologist or his representative during grading/construction operations.

Temporary Shoring

Temporary vertical excavations removing support from adjacent properties will require the use of temporary shoring. Shoring recommendations may be provided upon request under a separate scope of work.

Geotechnical Observations

Clearing operations, removal of unsuitable materials, and general grading procedures should be observed by the project soils consultant or his representative. Compacted fill should not be placed without prior bottom observations being conducted by the soils consultant or his representative to verify the adequacy of the removals.

The project soils consultant or his representative should be present to observe grading operations and to check that the minimum compaction requirements are being obtained. In addition, verification of compliance with the other grading recommendations presented herein should be provided concurrently.

Post Grading Considerations

Slope Landscaping and Maintenance

Provided all drainage provisions are properly constructed and maintained, the gross stability of graded slopes should not be adversely affected. However, satisfactory slope and building pad drainage is essential for the long term performance of the site. Concentrated drainage should not be allowed to flow uncontrolled over any descending slope. As recommended by the project landscape architect, engineered slopes should be landscaped with deep rooted, drought tolerant maintenance free plant species.

Site Drainage

Maintaining control over drainage throughout the site is important for the long term performance of the proposed improvements. We recommend roof gutters or equivalent roof collection system for proposed structures. Pad and roof drainage should be routed in non-erosive drainage devices to driveways, adjacent streets, storm-drain facilities, or other locations approved by the building official. Drainage should not be allowed to pond on the building pad or near any foundations. Planters located within retaining wall backfill should be sealed to prevent moisture intrusion into the backfill. Planters located next to structures should be sealed to the depth of the footings. Drainage control devices require periodic cleaning, testing and maintenance to remain effective.

Building pad drainage should be designed to meet the minimum gradient requirements of the CBC, to divert water away from foundations.

Utility Trenches

All utility trench backfill should be compacted at near optimum moisture to a minimum of 90 percent of the maximum dry density as determined by ASTM D1557-12. Trench backfill should be placed in approximately 6 to 8 inch maximum loose lifts and then mechanically compacted with a hydro-hammer, a sheepsfoot, pneumatic tampers, or similar equipment. Within pavement areas, the upper 6 inches of subgrade materials for utility trench backfill should be compacted to 95 percent of the maximum dry density determined by ASTM D1557-12. The utility trench backfill should be observed and tested by the project soils engineer or their representative to verify that the minimum compaction requirements have been obtained.

Where utility trenches undercut perimeter foundations, all utility trenches should be backfilled with compacted fill, lean concrete, or concrete slurry. When practical, interior or exterior utility trenches that run parallel to structure footings should not be located within a 1:1 (h:v) plane projected downward from the outside bottom edge of the footing.

SEISMIC DESIGN PARAMETERS

Ground Motions

To resist the effects of design level seismic ground motions in order to prevent collapse (1% probability of collapse in 50 years), structures are required to be designed and constructed in accordance with the 2016 California Building Code Section 1613. The design is reliant on the site class, risk category (I, II, III, or IV), and mapped spectral accelerations for short periods (S_s) and a 1-second period (S_1).

Based on data and maps jointly compiled by the United States Geological Survey (USGS) and the California Geological Survey (CGS), spectral accelerations for the subject property were generated via a software application provided by the USGS website, *Earthquake Hazards Program*. The data summarized in the following table is based on the Maximum Considered Earthquake Geometric Mean (MCE_G) with 5% damped ground motions having a 2% probability of being exceeded in 50 years (2,475 year return period).

The seismic design parameters were determined by a combination of the site class, mapped spectral accelerations, on site soil/rock conditions, and risk category. The compilation of seismic design parameters found below are considered appropriate for implementation during structural design. The USGS Design Summary Report is included in Appendix D.

PARAMETER		FACTOR
Site Location		Latitude: 33.6832 Longitude: -117.3343
Site Class (1613.3.2 of 2016 CBC, Chapter 20 of ASCE 7)		D
Mapped Spectral Accelerations for short periods	S_s (g)	2.298
Mapped Spectral Accelerations for 1-Second Period	S_1 (g)	0.919
Maximum Considered Earthquake Spectral Response Acceleration for Short Periods	S_{ms} (g)	2.068
Maximum Considered Earthquake Spectral Response Acceleration for 1-Second Period	S_{m1} (g)	2.206
Design Spectral Response Acceleration for Short Periods	S_{DS} (g)	1.379
Design Spectral Response Acceleration for 1-Second Period	S_{D1} (g)	1.470
Seismic Design Category		E
Importance Factor Based on Occupancy Category		II

A probabilistic seismic hazard assessment for the site was conducted in accordance with the 2016 CBC, Section 1803.5.12. The probabilistic seismic hazard maps and data files were jointly prepared by the United States Geological Survey (USGS) and the California Geological Survey (CGS). Actual ground shaking intensities at the subject property may be substantially higher or lower based on complex variables such as the near source directivity effects, depth and consistency of soils, topography, geologic structure, direction of fault rupture, seismic wave reflection, refraction, and attenuation rates. The estimated probabilistic peak ground acceleration at the site is, PGA = 0.901. The anticipated horizontal ground acceleration for evaluating the potential for liquefaction at the site during the design earthquake event is 0.811 g ($PGA_M = F_{PGA}$ PGA, per the 2016 CBC Section 1803.5.12).

Secondary Seismic Hazards

Secondary effects of seismic shaking include several types of ground failure as well as induced flooding. Ground failure that could occur as a consequence of severe ground shaking, include landslides, ground lurching, shallow ground rupture, and liquefaction/lateral spreading. The likelihood of occurrence of each type of ground failure depends on the severity and distance from the earthquake epicenter, topography, geologic structure, groundwater conditions, and other factors. All of the secondary effects of seismic activity listed above are considered to be unlikely, based on our experience, subsurface exploration, and laboratory testing.

Seismically induced flooding is normally associated with a tsunami (seismic sea wave), a seiche (i.e., a wave-like oscillation of surface water in an enclosed basin that may be initiated by a strong earthquake) or failure of a major reservoir or retention system up gradient of the site. As a result of the site being at an elevation of more than 1,000 feet above mean sea level and being more than 20 miles inland from the nearest coastline of the Pacific Ocean, the potential for seismically induced flooding due to a tsunamis is considered remote. The likelihood of induced flooding due to a seiche overcoming a dam's freeboard is considered remote. In addition, it is considered remote that any major reservoir up gradient of the subject property would be compromised to a point of failure.

Liquefaction and Lateral Spreading

The three requirements for liquefaction to occur include seismic shaking, poorly consolidated cohesionless sands, and groundwater. Liquefaction results in a substantial loss of shear strength in loose, saturated, cohesionless soils subjected to earthquake induced ground shaking. Potential impacts from liquefaction include loss of bearing capacity, liquefaction related settlement, lateral movements, and surface manifestation in the form of sand boils. The potential for design level earthquake induced liquefaction and lateral spreading to occur beneath the proposed structures is considered very low to remote due to the recommended compacted fill and the dense nature of the deeper onsite soils.

We have provided liquefaction analyses that model the existing ungraded conditions and recommended graded conditions, using a groundwater level of 5 feet to represent a conservative historic high groundwater level. The analyses of the post graded conditions revealed that potentially liquefiable soils were encountered in boring B-2, from 14 to 19 feet. Our analyses were performed utilizing the guidelines of *Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction Hazards in California* (SCEC, 1999) and *Guidelines for Evaluating and Mitigating Seismic Hazards in California*, California Geological Survey, Special Publication 117A 2008. Based on our calculations, we estimate that dynamic settlement of sands due to liquefaction will be on the order of 1.7 inches in the vicinity of Boring B-2 prior to performing the recommended grading improvements. Upon completion of the recommended grading improvements we estimate that dynamic settlement of sands due to liquefaction will be on the order of 0 inches in the vicinity of Boring B-2. The liquefaction potential and dynamic settlement of sands calculations can be found in Appendix E.

Ground Subsidence

Groundwater or oil withdrawal from soils can cause a permanent collapse of pore space previously occupied by the fluid. The consolidation of subsurface sediments resulting from fluid withdrawal may cause the ground surface to subside, potentially resulting in differential subsidence which can significantly damage engineered structures. Since excessive withdrawal of fluids is not anticipated in the vicinity of the proposed project, the potential for subsidence is considered low to remote.

PRELIMINARY FOUNDATION DESIGN RECOMMENDATIONS

General

Shallow foundations are considered feasible for support of the proposed structures, provided grading and construction are performed in accordance with the recommendations of this report. Foundation recommendations are provided in the following sections. Graphic presentations of relevant information and recommendations are also included on Plate 1 – Geotechnical Map.

Allowable Bearing Values

An allowable bearing value of 1,500 pounds per square foot (psf) is recommended for design of 12 inch wide continuous footings founded at a minimum depth of 12 inches below the lowest adjacent final grade and 24 inch square pad footings. This value may be increased by 20 percent for each additional 1-foot of width and/or depth to a maximum value of 2,000 psf. Recommended allowable bearing values include both dead and frequently applied live loads and may be increased by one third when designing for short duration wind or seismic forces.

Settlement

We estimate that the maximum total settlement of the footings will be less than approximately ¾ inch, based on the anticipated loading and the settlement characteristics of the underling earth materials. Differential settlement is expected to be about ½ inch over a horizontal distance of approximately 20 feet, for an angular distortion ratio of 1:480. The majority of the settlement is anticipated to occur during construction or shortly after the initial application of loading.

The above settlement estimates are based on the assumption that the grading and construction are performed in accordance with the recommendations presented in this report. Additionally, the project soils consultant or his representative will be provided the opportunity to observe the foundation excavations.

Lateral Resistance

Passive earth pressure of 200 psf per foot of depth to a maximum value of 2,000 psf may be used to establish lateral bearing resistance for footings. A coefficient of friction of 0.30 times the dead load forces may be used between concrete and the supporting soils to determine lateral sliding resistance. When combining passive and friction for lateral resistance, the passive component should be reduced by one third. In no case shall the lateral sliding resistance exceed one-half the dead load for clay, sandy clay, sandy silty clay, silty clay, and clayey silt.

The above lateral resistance values are based on footings for an entire structure being placed directly against compacted fill.

Expansive Soil Considerations

The preliminary laboratory test results indicate that the onsite soils exhibit an expansion potential of **MEDIUM** as classified by the 2016 CBC Section 1803.5.3 and ASTM D4829-03.

Additional, testing for expansive soil conditions should be conducted upon completion of rough grading and prior to construction. The following recommendations should be considered the very minimum requirements, for the soils tested. It is common practice for the project architect or structural engineer to require additional slab thickness, footing sizes, and/or reinforcement.

Medium Expansion Potential (Expansion Index of 51 to 90)

Our laboratory test results indicate that the soils onsite exhibit a **MEDIUM** expansion potential as classified by the 2016 CBC Section 1803.5.3 and ASTM D4829-03. As such, the CBC specifies that slab on grade foundations (floor slabs) resting on soils with expansion indices greater than 20, require special design considerations per the 2016 CBC Sections 1808.6.1 and 1808.6.2. The design procedures incorporate the thickness and plasticity index of the various soils within the upper 15 feet of the proposed structure. We have assumed an effective plasticity index of 16, for preliminary design purposes.

Conventional Footings

- Exterior continuous footings should be founded at the minimum depths below the lowest adjacent final grade (i.e. minimum 18 inch depth for one-story and two-story, and minimum 24 inch depth for three-story construction). Interior continuous footings for one-, two-, and three-story construction may be founded at a minimum depth of 12 inches below the lowest adjacent final grade. In accordance with Table 1809.7 of the 2016 CBC, all continuous footings should have a minimum width of 12, 15, and 18 inches, for one-, two-, and three-story structures, respectively, and should be reinforced with a minimum of four (4) No. 4 bars, two (2) top and two (2) bottom.

- Exterior pad footings intended to support roof overhangs, such as second story decks, patio covers and similar construction should be a minimum of 24 inches square and founded at a minimum depth of 18 inches below the lowest adjacent final grade. The pad footings should be reinforced with a minimum of No. 4 bars spaced a maximum of 18 inches on center, each way, and should be placed near the bottom-third of the footings.

Building Floor Slabs

- Building floor slabs should be a minimum of 4 inches thick. All floor slabs should be reinforced with a minimum of No. 3 bars spaced a maximum of 18 inches on center, each way, supported by concrete chairs or bricks to ensure desired mid-depth placement. Based on an assumed effective plasticity index of 16, the project architect or structural engineer should evaluate minimum floor slab thickness and reinforcement in accordance with 2016 CBC Section 1808.6.2.

- Building floor slabs with moisture sensitive or occupied areas, should be underlain by a minimum 10-mil thick moisture barrier to help reduce the upward migration of moisture from the underlying soils. The moisture barrier should be properly installed using the guidelines of ACI publication 318-05 and meet the performance standards of ASTM E 1745 Class A material. Prior to placing concrete, it is the responsibility of the contractor to ensure that the moisture barrier is properly placed and free of openings, rips, or punctures. As an option for additional moisture protection and foundation strength, higher strength concrete, such as a minimum compressive strength of 5,000 pounds per square inch (psi) in 28-days may be used. In addition, a capillary break/vapor retarder for concrete slabs should be provided in accordance with CALGreen. Ultimately, the design of the moisture barrier system along with recommendations for concrete placement and curing are the purview of the foundation engineer, factoring in the project conditions provided by the architect and owner.

- Garage floor slabs should be a minimum of 5 inches thick and should be reinforced in a similar manner as living area floor slabs. Garage floor slabs should be placed separately from adjacent wall footings with a positive separation maintained with ⅜ inch minimum felt expansion joint materials and quartered with weakened plane joints. A 12 inch wide turn down founded at the same depth as adjacent footings should be provided across garage entrances. The turn down should be reinforced with a minimum of two (2) No. 4 bars, one (1) top and one (1) bottom.

- Prior to placing concrete, the subgrade soils below all floor slabs should be pre-watered to achieve a moisture content at least 1.1 times optimum. The moisture content should penetrate a minimum depth of 12 inches into the subgrade soils. The pre-watering should be verified and tested by CW Soils.

Post Tensioned Slab/Foundation Design Recommendations

In lieu of the proceeding foundation recommendations, post tensioned slabs may be used for the proposed structures. Post tension foundations are generally considered to be a better foundation system, but may be slightly higher in overall cost. The foundation engineer may design the post tensioned foundation system using the following Post Tensioned Foundation Slab Design table. These parameters have been provided in general accordance with Post Tensioned Design. Alternate designs addressing the effects of expansive soils are allowed per 2016 CBC Section 1808.6.2. When utilizing these parameters, the foundation engineer should design the foundation system in accordance with the allowable deflection criteria of applicable codes.

It should be noted that the post tensioned design methodology is partially based on the assumption that soils moisture changes around and underneath post tensioned slabs, are only influenced by climate conditions. With regard to expansive soils, moisture variations below slabs are the major factor in foundation damage. However, the design methodology does not take into account presaturation, owner irrigation, or other non-climate related influences on the moisture content of the subgrade soils. In recognition of these realities, we modified the soils parameters obtained from this methodology to help account for reasonable irrigation practices. Additionally, the slab subgrades should be presoaked to a depth of 12 inches and maintained at above optimum moisture until placing concrete. Furthermore, prior to placing concrete, the subgrade soils below all floor slabs and perimeter footings should be presoaked to achieve moisture contents at least 1.0, 1.1, 1.2, and 1.3 times optimum to depths of 6, 12, 18, and 24 inches for Low, Medium, High, and Very High expansion potential soils, respectively. The moisture content should penetrate to a minimum depth of 24 inches into the subgrade soils. The pre-watering should be verified and tested by CW Soils.

Ponding water near the foundation can significantly change the moisture content of the soils below the foundation, causing excessive foundation movement and detrimental effects. Our recommendations do not account for excessive irrigation and/or incorrect landscape designs. To prevent moisture infiltration below the foundation, planters placed adjacent to the foundation should be designed with an effective drainage system or liners. Some lifting of the perimeter foundation should be expected even with properly constructed planters.

Future owners should be informed and educated of the importance in maintaining a consistent level of moisture within the soils around structures. Potential negative consequences can result from either excessive watering or allowing expansive soils to become too dry. Expansive soils will shrink as they dry, followed by swelling during the rainy winter season or when irrigation is resumed, causing distress to site improvements.

Post Tensioned Foundation Slab Design

PARAMETER		VALUE
Expansion Index		Medium[1]
Percent Finer than 0.002 mm in the Fraction Passing the No. 200 Sieve		< 30 percent (assumed)
Clay Mineral Type		Montmorillonite (assumed)
Thornthwaite Moisture Index		-20
Depth to Constant Soil Suction		7 feet
Constant Soil Suction		P.F. 3.6
Moisture Velocity		0.7 inch/month
Center Lift	Edge moisture variation distance, e_m Center lift, y_m	5.5 feet 2.5 inches
Edge Lift	Edge moisture variation distance, e_m Edge lift, y_m	3.5 feet 1.0 inches
Soluble Sulfate Content for Design of Concrete Mixtures in Contact with Soils		Negligible
Modulus of Subgrade Reaction, k (assuming presaturation as indicated below)		120 pci
Minimum Perimeter Foundation Embedment		18
Perimeter Foundation Reinforcement		--
Under Slab Moisture Barrier and Sand Layer		10-mil thick moisture barrier meeting the requirements of a ASTM E 1745 Class A material

1. Assumed for design purposes or obtained by laboratory testing.
2. Recommendations for foundation reinforcement are ultimately the purview of the foundation/structural engineer based upon the soils criteria presented in this report and structural engineering considerations.

Structural Setbacks and Building Clearance

Structural setbacks are required by the 2016 California Building Code (CBC). No additional structural setbacks are required due to geologic or soils conditions within the site. Improvements constructed near natural or properly compacted engineered slopes can, over time, be affected by natural processes including gravity forces, shrink/swell processes, weathering, and long term secondary settlement. As a result, the CBC requires that structures be setback or footings deepened to resist the influence of these processes.

For structures that are planned near ascending and descending slopes, the footings should be embedded to satisfy the requirements presented in the 2016 CBC, Section 1808.7. Foundations are required to be founded in accordance with the Foundation Clearances from Slopes Detail (CBC, 2016), which is illustrated in the last Appendix of this report.

Foundation Observations

Prior to the placement of forms, concrete, or steel, all foundation excavations should be observed by the geologist, engineer, or his representative to verify that they have been excavated into competent bearing materials, in accordance with the 2016 CBC. The foundations should be excavated per the approved plans, moistened, cleaned of all loose materials, trimmed neat, level, and square. Moisture softened soils should be removed prior to steel or concrete placement. Soils from foundation excavations should be removed from slab on grade areas, unless they have been properly compacted and tested.

Corrosivity

Corrosion is defined by the National Association of Corrosion Engineers (NACE) as "a deterioration of a substance or its properties because of a reaction with its environment." From a soils engineering point of view, the "substances" are the reinforced concrete foundations or buried metallic elements (not surrounded by concrete) and the "environment" is the prevailing soils in contact with them. Many factors can contribute to corrosivity, including the presence of chlorides, sulfates, salts, organic materials, different oxygen levels, poor drainage, varying soils consistencies, and moisture content. It is not considered practical or realistic to test for all of the factors which may contribute to corrosivity.

The level of chlorides considered to be significantly detrimental to concrete is based upon the industry recognized Caltrans standard "Bridge Design Specifications". Under subsection 8.22.1 of that document, Caltrans established that "Corrosive water or soil contains more than 500 parts per million (ppm) of chlorides". Based on limited testing, the onsite soils tested have chloride contents *less* than 500 ppm. Therefore, specific requirements resulting from elevated chloride contents are not required.

When the soluble sulfate content of soils exceeds 0.1 percent by weight, specific guidelines for concrete mix design are provided in the 2016 CBC Section 1904 and in ACI 318, Section 4.3 Table 4.3.1. Based on limited testing, the onsite soils are classified as having a *negligible* sulfate exposure condition, in accordance with Table 4.3.1. Therefore, structural concrete in contact with onsite soils should utilize Type I or II.

The onsite soils in contact with buried steel should be considered *moderately corrosive*, based on our laboratory testing of resistivity. Additionally, pH values below 9.7 are recognized as being corrosive to most common metallic components including, copper, steel, iron, and aluminum. The pH values for the soils tested were *lower* than 9.7. Therefore, any steel or metallic materials that are exposed to the soils should be encased in concrete or other remedies applied to provide corrosion protection.

For structures utilizing post tensioned systems, the post tensioning cables should be encased in concrete and/or encapsulated in accordance with the Post Tensioning Institute Guide Specifications. If post tensioning cable end plate anchors and nuts are exposed, they should also be protected. If the anchor plates and nuts are recessed into the edge of the concrete slab, the recess should be filled in with a non-shrink, non-porous, moisture-insensitive epoxy grout so that the anchorage assembly and the end of the cable are completely encased and isolated from the soils. A standard non-shrink, non-metallic cementatious grout may be used only when the post tension anchoring assembly is polyethylene encapsulated, similar to that offered by Hayes Industries, LTD or O'Strand, Inc.

It should be noted that CW Soils are not corrosion engineers and the test results for corrosivity are based on limited samples thought to be representative. The grading operations may blend various soils together and/or unveil soils with higher corrosive properties. This blending or imported material could alter and increase the detrimental properties of the onsite soils. Thus, it is important that additional testing near final grades for chlorides and sulfates along with testing for pH and resistivity be performed upon completion of the grading operations. Laboratory test results are presented in Appendix C.

RETAINING WALLS

Active and At-Rest Earth Pressures

Retaining wall foundations may be designed in accordance with the recommendations provided in the Preliminary Foundation Design Recommendation section of this report. For design of retaining walls up to 6 feet high, the table below provides the minimum recommended equivalent fluid pressures.

The active earth pressure should be used for design of unrestrained retaining walls, which are free to tilt slightly. The at-rest earth pressure should be used for design of retaining walls that are restrained at the top, such as basement walls, curved walls with no joints, or walls restrained at corners. For curved walls, active pressure may be used if tilting is acceptable and construction joints are provided at each angle point and at a minimum of 15 foot intervals along the curved segments.

MINIMUM STATIC EQUIVALENT FLUID PRESSURE (pcf, ≤*6 feet high)		
PRESSURE TYPE	BACKSLOPE CONDITION	
	LEVEL	2:1 (h:v)
Active Earth Pressure	45	75
At-Rest Earth Pressure	68	110

Hydrostatic pressure behind the retaining walls has not been taken into account when calculating the parameters provided. Therefore, the subdrain system is a very important part of the design. If additional loads are being applied within a 1:1 plane projected up from the heel of the retaining wall footing, due to surcharge loads imposed by other nearby walls, structures, vehicles, etc., then additional pressure should be added to the above earth pressures to account for the expected surcharge loads. In order to minimize surcharge loads and the settlement potential of nearby structures, the footings for the structure can be deepened below the 1:1 plane projected up from the heel of the retaining wall footing.

Upon request and under a separate scope of work, more detailed analyses can be provided to address retaining wall designs with regard to value engineering, stepped retaining walls, actual retaining wall heights, actual backfill inclinations, specific backfill materials, higher retaining walls requiring earthquake design motions, etc.

Subdrain System

To prevent the buildup of hydrostatic pressure behind the proposed retaining walls, we recommend a perforated pipe and gravel subdrain system be provided behind all retaining walls. The subdrain system should consist of 4 inch minimum diameter Schedule 40 PVC or ABS SDR-35 perforated pipe, placed with the perforations facing down. The pipe should be surrounded by a minimum of 1 cubic foot per foot of ¾- or 1½ inch open graded gravel wrapped in Mirafi 140N or equivalent filter fabric, to prevent infiltration of fines and subsequent clogging of the subdrain system.

In addition, the retaining walls should be adequately coated on the backfilled side of the walls with a proven waterproofing compound by an experienced professional to inhibit infiltration of moisture through the walls.

Temporary Excavations

All excavations should be made in accordance with Cal-OSHA requirements. CW Soils is not responsible for job site safety.

Retaining Wall Backfill

Retaining wall backfill materials should be approved by the soils engineer or his representative prior to placement as compacted fill. Retaining wall backfill should be placed in lifts no greater than 6 to 8 inches, watered or air dried as necessary to achieve near optimum moisture contents. All retaining wall backfill should be compacted to a minimum of 90 percent of the maximum dry density as determined by ASTM D1557. When practical, retaining wall backfill should be capped with a paved surface drain.

EXTERIOR CONCRETE

Subgrade Preparation

Subgrade soils underlying concrete flatwork should be compacted at near optimum moisture to a minimum of 90 percent of the maximum dry density as determined by ASTM test method D1557-12. Prior to placing concrete, the subgrade soils should be moistened to at least optimum or slightly above optimum moisture content (see table below). Pre-watering of the soils prior to placing concrete will promote uniform curing of the concrete and minimize the development of shrinkage cracks. The higher the expansion potential of the onsite soils the longer it will take to achieve the recommended presaturation. Therefore, the procedure and timing should be planned in advance.

Flatwork Design

Cracking within concrete flatwork is often a result of factors such as the use of too high of a water to cement ratio and/or inadequate steps taken to prevent moisture loss during the curing of the concrete. However, minor cracking within concrete flatwork is normal and should be expected. It should be noted that the reduction of slab cracking is often a function of proper slab design, concrete mix design, placement, curing, and finishing practices. We recommend the adherence to the guidelines of the American Concrete Institute (ACI).

When placed over expansive soils, exterior concrete elements are susceptible to lifting and cracking. When this occurs with highly expansive soils, the detrimental impacts can be significant and may necessitate the removal and replacement of the affected improvements. In order to reduce the potential for unsightly cracking, we suggest a combination of presaturation of the subgrade soils, reinforcement, restraint, and a layer of granular materials. Although these measures may not completely eliminate distress to concrete improvements, the application of these measures can significantly reduce the distress caused by expansive soils. The degree and extent the measures recommended in the following table are applied depend on:

- The expansion potential of the subgrade soils.
- The practicality of implementing the measures (such as presaturation).
- The benefits verse the economics of the measures.

The project owner should perform a cost/benefit analysis on the factors to determine the extent the measures will be applied to each project. The expansive potential of the onsite soils should be considered **MEDIUM**.

CONCRETE FLATWORK					
CONSTRUCTION DESIGN	**EXPANSION INDEX**				
	VERY LOW	**LOW**	**MEDIUM**	**HIGH**	**VERY HIGH**
Slab Thickness, Minimum	3.5 inches	3.5 inches	4.5 inches	4.5 inches	5 inches
Subbase, Gravel Layer	NA	NA	Optional	3 inches	4 inches
Presaturation, Relative to Optimum Moisture Content	Pre-wet NA	Optimum 6 inches Deep	1.1 x Optimum 12 inches Deep	1.2 x Optimum 18 inches Deep	1.3 x Optimum 24 inches Deep
Joint, Maximum Spacing, (joint to extend ¼ slab)	10 feet or less	10 feet or less	8 feet or less	6 feet or less	6 feet or less
Reinforcement, Mid-Depth	NA	NA	Optional (WWF 6 x 6 W1.4 x W1.4)	No. 3 Rebar 24" On Center Both Ways	No. 3 Rebar 24" On Center Both Ways
Restraint, Slip Dowels Mid-Depth	NA	NA	Optional	Across Cold Joints	Across Cold Joints

The use of a granular layer for exterior slabs is primarily intended to facilitate presaturation and subsequent construction operations by providing a working surface over the saturated soils and to help retain the moisture. Where these factors are insignificant, the layer may be omitted.

PRELIMINARY PAVEMENT DESIGN

An assumed R-value of 10 may be used for preliminary pavement design. Calculated in accordance with the State of California design procedures using assumed Traffic Indices, the following table summarizes the minimum recommended asphalt concrete pavement sections. Final pavement design should be based on sampling and testing of post grading conditions. Alternative, but equivalent pavement sections and calculation sheets have been provided within the appendices of this report.

ASPHALT CONCRETE PAVEMENT DESIGN			
PARAMETERS	**AUTO PARKING**	**AUTO DRIVES**	**ENTRANCES/TRUCK DRIVES**
Assumed Traffic Index	4.0	5.0	6.0
Preliminary Design R-Value	10	10	10
AC Thickness (inches)	3	3	3½
AB Thickness (inches)	5	9	11.5

Note: AC – Asphalt Concrete
AB – Aggregate Base

The following table includes the minimum recommended Portland cement concrete pavement design sections calculated using the guidelines of the State of California design procedures.

PORTLAND CEMENT CONCRETE PAVEMENT DESIGN			
Street Type	**Preliminary Design R-Value**	**Traffic Index**	**Pavement Section (inches)**
ENTRANCES/TRUCK DRIVES	10	6.0	6 PCC over 10 AB

Note: PCC – Portland Cement Concrete
AB – Aggregate Base

The minimum requirements for the Portland cement concrete shall be a six sack mix and 3,500 pounds per square inch at 28 days.

The subgrade soils immediately below the aggregate base (base) should be compacted to a minimum of 90 percent of the maximum dry density as determined by ASTM D1557 to a minimum depth of 12 inches. Base materials should be compacted to a minimum of 95 percent of the maximum dry density as determined by ASTM D1557.

Base materials should consist of Class 2 aggregate base conforming to Section 26-1.02B of the State of California Standard Specifications or crushed aggregate base conforming to Section 200-2 of the Standard Specifications for Public Works Construction (Greenbook). Base materials should be compacted at or slightly below optimum moisture content. Asphalt concrete materials and construction operations should conform to Section 203 of the Greenbook.

GRADING PLAN REVIEW AND CONSTRUCTION SERVICES

This report has been prepared for the exclusive use of **Mr. Rod Oshita** and their authorized representative. It is unlikely to contain sufficient information for other parties or other uses. CW Soils should be provided the opportunity to review the final design plans and specifications prior to construction, in order to verify that the recommendations have been properly incorporated into the project plans and specifications. If CW Soils is not accorded the opportunity to review the project plans and specifications, we are not responsibility for misinterpretation of our recommendations.

We recommend that CW Soils be retained to provide soils engineering and engineering geologic services during the grading and foundation excavation phases of work, in order to allow for design changes in the event that the subsurface conditions differ from those anticipated prior to construction.

CW Soils should review any changes in the project and modify the conclusions and recommendations of this report in writing. This report along with the drawings contained within are intended for design input purposes only and are not intended to act as construction drawings or specifications. In the event that conditions during grading or construction operations appear to differ from those indicated in this report, our office should be notified immediately, as appropriate revisions may be required.

REPORT LIMITATIONS

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

Soils vary in type, strength, and other engineering properties between points of observation and exploration. Groundwater and moisture conditions can also vary due to natural processes or the works of man on this or adjacent properties. As a result, we do not and cannot have complete knowledge of the subsurface conditions beneath the proposed project. No practical study can completely eliminate uncertainty with regard to the anticipated geologic and soils engineering conditions in connection with a proposed project. The conclusions and recommendations within this report are based upon the findings at the points of observation and are subject to confirmation by CW Soils based on the conditions revealed during grading and construction operations.

This report was prepared with the understanding that it is the responsibility of the owner, to ensure that the conclusions and recommendations contained herein are brought to the attention of the other project consultants and are incorporated into the plans and specifications. The owners' contractor should implement the recommendations in this report and notify the owner as well as our office if they consider any of the recommendations presented herein to be unsafe or unsuitable.

APPENDIX A

REFERENCES

APPENDIX A

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APPENDIX B

FIELD EXPLORATION

LOG SYMBOLS & TERMS

The No. 200 Standard Sieve is about the smallest particle visible to the naked eye.					
Coarse-grained Soils >½ of materials larger than #200 sieve	**GRAVELS** Higher percentage of coarse fraction is larger than #4 sieve	Clean Gravels (less than 5% fines)		**GW**	Well-graded gravels, little or no fines
				GP	Poorly-graded gravels, little or no fines
		5 – 12% fines		**GW-GM**	Well-graded gravel with silt
				GW-GC	Well-graded gravel with clay
				GP-GM	Poorly-graded gravel with silt
				GP-GC	Poorly-graded gravel with clay
		Gravels with fines	PI < 4	**GM**	Silty Gravels
			PI > 7	**GC**	Clayey Gravels
	SANDS Higher percentage of coarse fraction is smaller than #4 sieve	Clean Sands (less than 5% fines)		**SW**	Well-graded sands, little or no fines
				SP	Poorly-graded sands, little or no fines
		5 – 12% fines		**SW-SM**	Well-graded sand with silt
				SW-SC	Well-graded sand with clay
				SP-SM	Poorly-graded sand with silt
				SP-SC	Poorly-graded sand with clay
		Sands with fines	PI < 4	**SM**	Silty Sands
			PI > 7	**SC**	Clayey Sands
			PI 4-7	**SC-SM**	Silty clayey sands
Fine-grained Soils ≥½ of materials smaller than #200 sieve	**SILTS & CLAYS** Liquid Limit Less Than 50		PI < 4	**ML**	Inorganic silts & sandy silts
			PI > 7	**CL**	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, lean clays
			PI 4-7	**ML-CL**	Silts & clays of low plasticity, sandy silty clay, silty clay
	SILTS & CLAYS Liquid Limit Greater Than 50			**MH**	Inorganic silts, micaceous or diatomaceous silt, sandy silt
				CH	Inorganic clays of high plasticity, fat clays, sandy clays, gravelly clays
				OH	Organic silts and clays of medium-to-high plasticity
Highly Organic Soils				**PT**	Peat, humus swamp soils with higher organic content

Symbols	
	Ring Sample
	SPT Sample
NR	No Recovery
▽	Groundwater

Grain Size				
Description		Sieve Size	Grain Size	Approximate Size
Boulders		>12”	>12”	Larger than basketball-sized
Cobbles		3-12”	3-12”	Fist-sized to basketball-sized
Gravel	Coarse	¾-3”	¾-3”	Thumb-sized to fist-sized
	Fine	#4-¾”	0.19-0.75”	Pea-sized to thumb-sized
Sand	Coarse	#10-#4	0.079-0.19”	Rock salt-sized to pea-sized
	Medium	#40-#10	0.017-0.079”	Sugar-sized to rock salt-sized
	Fine	#200-#40	0.0029-0.017”	Flour-sized to sugar-sized
Fines		Passing #200	<0.0029”	Flour-sized and smaller

Moisture Content
Slightly Moist
Moist
Very Moist
Wet

Consistency – Fine Grained Soils			
Apparent Density	SPT (# blows/foot)	Modified CA Sampler (# blows/foot)	Field Test
Very Soft	<1	<2	Easily penetrated by thumb; exudes between thumb and fingers when squeezed in hand
Soft	2-3	3-6	Easily penetrated one inch by thumb; molded by light finger pressure
Medium Stiff	4-6	7-12	Penetrated over ½ inch by thumb with moderate effort; molded by strong finger pressure
Stiff	7-10	13-15	Indented about ½ inch by thumb but penetrated only with great effort
Very Stiff	11-20	16-30	Readily indented thumbnail
Hard	>20	>30	Indented with difficulty by thumbnail

Relative Density – Coarse Grained Soils			
Apparent Density	SPT (# blows/foot)	Modified CA Sampler (# blows/foot)	Field Test
Very Loose	<2	<4	Easily penetrated with ½ inch reinforcing rod pushed by hand
Loose	3-5	4-10	Easily penetrated with ½ inch reinforcing rod pushed by hand
Medium Dense	6-15	11-30	Easily penetrated 1-foot with ½ inch reinforcing rod driven with a 5-lb hammer
Dense	16-25	31-50	Difficult to penetrate 1-foot with ½ inch reinforcing rod driven with a 5-lb hammer
Very Dense	>25	>50	Penetrated only a few inches with ½ inch reinforcing rod driven with a 5-lb hammer

Geotechnical Boring Log B-1

Date: March 24, 2015	**Project Name: Multi-Tenent Building** **Page: 1 of 1**
Project Number: 14396-10	**Logged By: CW**
Drilling Company: California Pacific	**Type of Rig: Mobile B53**
Drive Weight (lbs): 140	**Drop (in): 30** **Hole Diameter (in): 8**
Top of Hole Elevation (ft): 1270	**Hole Location: See Geotechnical Map**

Depth (ft)	Blow Count Per Foot	Sample Number	Dry Density (pcf)	Moisture (%)	Classification Symbol	MATERIAL DESCRIPTION
0						**Quaternary Young Alluvial Deposits (Qya):**
					SM	Silty SAND; light brown, slightly moist, loose, some gravel
5	13	R-1	-	2.6		medium dense
10	7	R-2	98.8	6.9		slightly moist to moist, loose
15	push	R-3	101.6	26.6	CL	CLAY with sand; medium brown, wet, very soft
					▽	groundwater at 15.5 feet
20	5	N-1	-	20.1	ML	Sandy SILT; medium brown, wet, medium stiff
						Total Depth: 21.5 Feet
						Groundwater at 15.5 Feet
25						
30						

Geotechnical Boring Log B-2

Date: March 24, 2015	**Project Name: Multi-Tenent Building** **Page: 1 of 2**
Project Number: 14396-10	**Logged By: CW**
Drilling Company: California Pacific	**Type of Rig: Mobile B53**
Drive Weight (lbs): 140	**Drop (in): 30** **Hole Diameter (in): 8**
Top of Hole Elevation (ft): 1271	**Hole Location: See Geotechnical Map**

Depth (ft)	Blow Count Per Foot	Sample Number	Dry Density (pcf)	Moisture (%)	Classification Symbol	MATERIAL DESCRIPTION
0						**Quaternary Young Alluvial Deposits (Qya):**
		Bag 1 @ 0-5'			SM	Silty SAND; light yellowish brown, slightly moist, loose
5	7	R-1	99.6	2.1		
					ML	Sandy SILT; moderate brown, moist, soft
	6	R-2	105.1	9.8		
10	7	R-3	99.9	8.7		medium stiff
15	4	R-4	91.6	29.1	CL	CLAY with sand; medium brown, wet, soft
				▽		groundwater at 17.5 feet
20	9	N-1	-	18.2		stiff
25	8				SC	Clayey SAND; medium brown, wet, loose
		R-5	113.3	17.9		
30						

Geotechnical Boring Log B-2

Date: March 24, 2015	**Project Name: Multi-Tenent Building** **Page: 2 of 2**
Project Number: 14396-10	**Logged By: CW**
Drilling Company: California Pacific	**Type of Rig: Mobile B53**
Drive Weight (lbs): 140	**Drop (in): 30** **Hole Diameter (in): 8**
Top of Hole Elevation (ft): 1271	**Hole Location: See Geotechnical Map**

Depth (ft)	Blow Count Per Foot	Sample Number	Dry Density (pcf)	Moisture (%)	Classification Symbol	MATERIAL DESCRIPTION
30	27	N-2	-	12.4	GM	Silty GRAVEL with sand; grayish brown, very moist to wet, very dense
35	40	N-3	-	9.3		moist
40	32	N-4	-	13.3		very moist to wet
45	50-2"	N-5	-	NR		**Practical Refusal at 45 Feet**
						Groundwater at 17.5 Feet
50						
55						
60						

CW SOILS

Geotechnical Boring Log B-3

Date: March 24, 2015	**Project Name: Multi-Tenent Building** **Page: 1 of 1**
Project Number: 14396-10	**Logged By: CW**
Drilling Company: California Pacific	**Type of Rig: Mobile B53**
Drive Weight (lbs): 140	**Drop (in): 30** **Hole Diameter (in): 8**
Top of Hole Elevation (ft): 1271.5	**Hole Location: See Geotechnical Map**

Depth (ft)	Blow Count Per Foot	Sample Number	Dry Density (pcf)	Moisture (%)	Classification Symbol	MATERIAL DESCRIPTION
0						**Quaternary Young Alluvial Deposits (Qya):**
					SM	Silty SAND; light brown, slightly moist, loose, some gravel
5	8	R-1	105.6	2.9		
10	9	R-2	103.0	6.3		slightly moist to moist
15	4	R-3	98.8	24.0	CL	CLAY with sand; medium brown, very moist to wet, soft
				▽		groundwater at 18 feet
20	3	N-1	-	NR		
						Total Depth: 21.5 Feet
						Groundwater at 18 Feet
25						
30						

Geotechnical Boring Log B-4

Date: March 24, 2015	**Project Name: Multi-Tenent Building** — **Page: 1 of 1**
Project Number: 14396-10	**Logged By: CW**
Drilling Company: California Pacific	**Type of Rig: Mobile B53**
Drive Weight (lbs): 140	**Drop (in): 30** — **Hole Diameter (in): 8**
Top of Hole Elevation (ft): 1272	**Hole Location: See Geotechnical Map**

Depth (ft)	Blow Count Per Foot	Sample Number	Dry Density (pcf)	Moisture (%)	Classification Symbol	MATERIAL DESCRIPTION
0						**Quaternary Young Alluvial Deposits (Qya):**
					SM	Silty SAND; light brown, slightly moist, loose, some gravel
					ML	Sandy SILT; medium brown, slightly moist to moist, medium stiff
5	8	R-1	100.4	6.8		
10	3	R-2	104.5	17.4		very moist, soft
15	9	R-3	96.1	23.6	CL	CLAY with sand; medium brown, wet, medium stiff, lenses of silty SAND
				▽		groundwater at 17 feet
					SM	Silty SAND; medium brown, wet, medium dense
20	7	N-1	-	NR		
						Total Depth: 21.5 Feet
						Groundwater at 17 Feet
25						
30						

CW SOILS

APPENDIX C

LABORATORY PROCEDURES AND TEST RESULTS

APPENDIX C

Laboratory Procedures and Test Results

Our laboratory testing has provided quantitative and qualitative data involving the relevant engineering properties of the representative soils selected for testing. Representative samples were tested using the guidelines of the American Society for Testing and Materials (ASTM) procedures or California Test Methods (CTM).

Soil Classification: The soils observed during exploration were classified and logged in general accordance with the Standard Practice for Description and Identification of Soils (Visual-Manual Procedure) of ASTM D 2488. Upon completion of laboratory testing, exploratory logs and sample descriptions may have been reconciled to reflect laboratory test results with regard to ASTM D 2487.

Moisture and Density Tests: For select samples, moisture content and dry density determinations were obtained using the guidelines of ASTM D 2216 and ASTM D 2937, respectively. These tests were performed on relatively undisturbed samples and the test results are presented on the exploratory logs.

Maximum Density Tests: The maximum dry density and optimum moisture content of representative samples were determined using the guidelines of ASTM D1557. The test results are presented in the table below.

SAMPLE LOCATION	MATERIAL DESCRIPTION	MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE CONTENT (%)
B-2 @ 0-5 feet	Sandy CLAY	130.5	9.5

Expansion Index: The expansion potential of representative samples was evaluated using the guidelines of ASTM D 4829. The test results are presented in the table below.

SAMPLE LOCATION	MATERIAL DESCRIPTION	EXPANSION INDEX	EXPANSION POTENTIAL
B-2 @ 0-5 feet	Sandy CLAY	53	MEDIUM

Minimum Resistivity and pH Tests: Minimum resistivity and pH tests of select samples were performed using the guidelines of CTM 643. The test results are presented in the table below.

SAMPLE LOCATION	MATERIAL DESCRIPTION	pH	MINIMUM RESISTIVITY (ohm-cm)
B-2 @ 0-5 feet	Sandy CLAY	7.5	1,380

Soluble Sulfate: The soluble sulfate content of select samples was determined using the guidelines of CTM 417. The test results are presented in the table below.

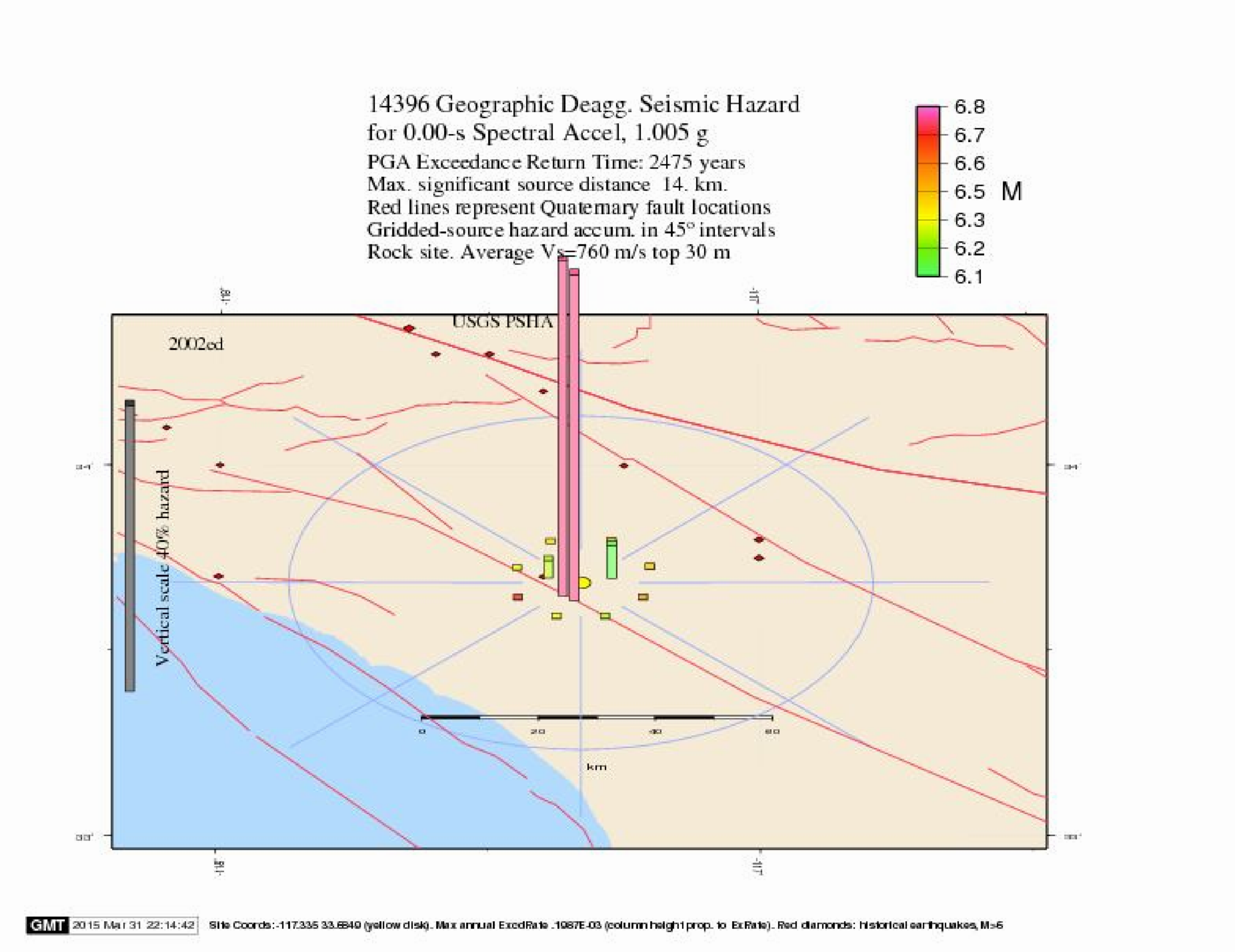
SAMPLE LOCATION	MATERIAL DESCRIPTION	SULFATE CONTENT (% by weight)	SULFATE EXPOSURE
B-2 @ 0-5 feet	Sandy CLAY	0.006	Negligible

Chloride Content: Chloride content of select samples was determined using the guidelines of CTM 422. The test results are presented in the table below.

SAMPLE LOCATION	MATERIAL DESCRIPTION	CHLORIDE CONTENT (ppm)
B-2 @ 0-5 feet	Sandy CLAY	50

APPENDIX D

SEISMICITY


14396 Geographic Deagg. Seismic Hazard
for 0.00-s Spectral Accel, 1.005 g
PGA Exceedance Return Time: 2475 years
Max. significant source distance 14. km.
Red lines represent Quaternary fault locations
Gridded-source hazard accum. in 45° intervals
Rock site. Average Vs=760 m/s top 30 m
6.8
6.7
6.6
6.5
M
6.3
6.2
6.1
USGS PSHA
2002ed
Vertical scale 40% hazard
-118°
-117°
0
20
40
60
km
GMT 2015 Mar 31 22:14:42
Site Coords:-117.336 33.6849 (yellow disk). Max annual ExcdRate .1987E-03 (column height prop. to ExRate). Red diamonds: historical earthquakes, M>6

*** Deaggregation of Seismic Hazard for PGA & 2 Periods of Spectral Accel. ***
*** Data from U.S.G.S. National Seismic Hazards Mapping Project, 2002 version ***
PSHA Deaggregation. %contributions. site: 14396 long: 117.335 W., lat: 33.684 N.
USGS 2002-03 update files and programs. dM=0.2. Site descr:ROCK
Return period: 2475 yrs. Exceedance PGA =1.0054 g.
#Pr[at least one eq with median motion>=PGA in 50 yrs]=0.00000

DIST(KM)	MAG(MW)	ALL_EPS	EPSILON>2	1<EPS<2	0<EPS<1	-1<EPS<0	-2<EPS<-1	EPS<-2
6.5	5.05	0.185	0.185	0.000	0.000	0.000	0.000	0.000
6.5	5.20	0.400	0.400	0.000	0.000	0.000	0.000	0.000
6.6	5.40	0.436	0.436	0.000	0.000	0.000	0.000	0.000
6.6	5.60	0.471	0.425	0.045	0.000	0.000	0.000	0.000
6.6	5.80	0.500	0.383	0.117	0.000	0.000	0.000	0.000
5.8	6.02	0.772	0.436	0.336	0.000	0.000	0.000	0.000
11.6	6.02	0.063	0.063	0.000	0.000	0.000	0.000	0.000
5.5	6.20	1.052	0.469	0.584	0.000	0.000	0.000	0.000
11.4	6.21	0.121	0.121	0.000	0.000	0.000	0.000	0.000
5.7	6.40	1.154	0.455	0.700	0.000	0.000	0.000	0.000
11.7	6.40	0.123	0.123	0.000	0.000	0.000	0.000	0.000
4.6	6.64	53.540	17.372	36.168	0.000	0.000	0.000	0.000
12.0	6.60	0.105	0.105	0.000	0.000	0.000	0.000	0.000
4.6	6.87	36.901	12.835	24.066	0.000	0.000	0.000	0.000
11.7	6.78	0.069	0.069	0.000	0.000	0.000	0.000	0.000
4.7	7.08	4.066	1.489	2.577	0.000	0.000	0.000	0.000

Summary statistics for above PSHA PGA deaggregation, R=distance, e=epsilon:
Mean src-site R= 4.7 km; M= 6.71; eps0= 1.44. Mean calculated for all sources.
Modal src-site R= 4.6 km; M= 6.64; eps0= 1.41 from peak (R,M) bin
Gridded source distance metrics: Rseis Rrup and Rjb
MODE R*= 4.5km; M*= 6.64; EPS.INTERVAL: 1 to 2 sigma % CONTRIB.= 36.168

Principal sources (faults, subduction, random seismicity having >10% contribution)

Source Category:	% contr.	R(km)	M	epsilon0 (mean values)
California SS faults	92.31	4.6	6.75	1.41
Individual fault hazard details if contrib.>1%:				
Elsinore-17	45.65	4.9	6.75	1.50
Elsinore-16	46.66	4.3	6.75	1.32

******************** Southern California **
PSHA Deaggregation. %contributions. ROCK site: 14396 long: 117.335 d W., lat: 33.684 N.
USGS 2002-2003 update files and programs. Analysis on DaMoYr:31/03/2015
Return period: 2475 yrs. 1.00 s. PSA =0.9106 g.
#Pr[at least one eq with median motion>=PSA in 50 yrs]=0.00000

DIST(km)	MAG(Mw)	ALL_EPS	EPSILON>2	1<EPS<2	0<EPS<1	-1<EPS<0	-2<EPS<-1	EPS<-2
6.6	5.81	0.072	0.072	0.000	0.000	0.000	0.000	0.000
5.3	6.03	0.216	0.210	0.006	0.000	0.000	0.000	0.000
4.8	6.20	0.440	0.380	0.060	0.000	0.000	0.000	0.000
11.5	6.21	0.061	0.061	0.000	0.000	0.000	0.000	0.000
5.0	6.41	0.711	0.486	0.225	0.000	0.000	0.000	0.000
12.4	6.41	0.116	0.116	0.000	0.000	0.000	0.000	0.000
4.6	6.66	46.548	20.521	26.026	0.000	0.000	0.000	0.000
12.8	6.61	0.175	0.175	0.000	0.000	0.000	0.000	0.000
4.7	6.88	44.684	15.387	29.297	0.000	0.000	0.000	0.000
12.8	6.79	0.180	0.179	0.001	0.000	0.000	0.000	0.000
4.7	7.08	6.115	1.540	4.575	0.000	0.000	0.000	0.000
13.7	6.95	0.053	0.053	0.000	0.000	0.000	0.000	0.000
4.3	7.15	0.254	0.033	0.210	0.011	0.000	0.000	0.000
51.5	7.81	0.060	0.060	0.000	0.000	0.000	0.000	0.000
51.5	8.04	0.120	0.120	0.000	0.000	0.000	0.000	0.000
51.5	8.22	0.134	0.134	0.000	0.000	0.000	0.000	0.000

Summary statistics for above 1.0s PSA deaggregation, R=distance, e=epsilon:
Mean src-site R= 4.9 km; M= 6.78; eps0= 1.52. Mean calculated for all sources.
Modal src-site R= 4.6 km; M= 6.66; eps0= 1.58 from peak (R,M) bin
Gridded source distance metrics: Rseis Rrup and Rjb

```
 MODE R*=   4.6km; M*= 6.88; EPS.INTERVAL: 1 to 2 sigma  % CONTRIB.= 29.297

Principal sources (faults, subduction, random seismicity having >10% contribution)
Source Category:                % contr.   R(km)    M   epsilon0 (mean values)
California SS faults              95.79     4.8   6.79    1.50
Individual fault hazard details if contrib.>1%:
Elsinore-17                       48.31     4.9   6.79    1.57
Elsinore-16                       47.16     4.4   6.78    1.43
******************** Southern California ****************************************
PSHA Deaggregation. %contributions. ROCK site: 14396  long: 117.335 d W., lat: 33.684 N.
USGS 2002-2003 update files and programs. Analysis on DaMoYr:31/03/2015
Return period: 2475 yrs. 0.20 s. PSA =2.4760   g.
#Pr[at least one eq with median motion>=PSA in 50 yrs]=0.00000
DIST(km) MAG(Mw) ALL_EPS EPSILON>2  1<EPS<2 0<EPS<1 -1<EPS<0 -2<EPS<-1 EPS<-2
    6.4    5.05    0.185    0.185    0.000    0.000    0.000    0.000    0.000
    6.5    5.20    0.407    0.407    0.000    0.000    0.000    0.000    0.000
    6.6    5.40    0.451    0.451    0.000    0.000    0.000    0.000    0.000
    6.6    5.60    0.488    0.446    0.043    0.000    0.000    0.000    0.000
    6.7    5.80    0.512    0.398    0.114    0.000    0.000    0.000    0.000
    5.9    6.02    0.755    0.436    0.319    0.000    0.000    0.000    0.000
   11.9    6.02    0.090    0.090    0.000    0.000    0.000    0.000    0.000
    5.5    6.20    1.010    0.472    0.539    0.000    0.000    0.000    0.000
   11.7    6.21    0.168    0.168    0.000    0.000    0.000    0.000    0.000
    5.7    6.40    1.112    0.466    0.646    0.000    0.000    0.000    0.000
   12.0    6.40    0.177    0.177    0.000    0.000    0.000    0.000    0.000
    4.6    6.64   53.304   16.549   36.755    0.000    0.000    0.000    0.000
   12.3    6.60    0.160    0.160    0.000    0.000    0.000    0.000    0.000
    4.6    6.87   36.971   11.765   25.207    0.000    0.000    0.000    0.000
   12.1    6.78    0.110    0.110    0.000    0.000    0.000    0.000    0.000
    4.8    7.07    3.373    1.217    2.156    0.000    0.000    0.000    0.000
    4.3    7.12    0.647    0.084    0.535    0.027    0.000    0.000    0.000

Summary statistics for above 0.2s PSA  deaggregation, R=distance, e=epsilon:
 Mean src-site R=    4.7 km; M= 6.71; eps0=   1.41. Mean calculated for all sources.
Modal src-site R=    4.6 km; M= 6.64; eps0=   1.39 from peak (R,M) bin
Gridded source distance metrics: Rseis Rrup and Rjb
 MODE R*=   4.5km; M*= 6.64; EPS.INTERVAL: 1 to 2 sigma  % CONTRIB.= 36.755

Principal sources (faults, subduction, random seismicity having >10% contribution)
Source Category:                % contr.   R(km)    M   epsilon0 (mean values)
California SS faults              92.18     4.6   6.75    1.38
Individual fault hazard details if contrib.>1%:
Elsinore-17                       46.60     4.8   6.75    1.46
Elsinore-16                       45.59     4.3   6.75    1.29
******************** Southern California ****************************************
```

Design Maps Detailed Report

ASCE 7-10 Standard (33.6849°N, 117.3358°W)

Site Class D – "Stiff Soil", Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From Figure 22-1 [1]	S_S = 2.293 g
From Figure 22-2 [2]	S_1 = 0.917 g

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Chapter 20.

Table 20.3–1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index *PI* > 20, • Moisture content *w* ≥ 40%, and • Undrained shear strength $\bar{s}_u$ < 500 psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 11.4.3 — Site Coefficients and Risk–Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Table 11.4–1: Site Coefficient F_a

Site Class	Mapped MCE_R Spectral Response Acceleration Parameter at Short Period				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_S

For Site Class = D and S_S = 2.293 g, F_a = 1.000

Table 11.4–2: Site Coefficient F_v

Site Class	Mapped MCE_R Spectral Response Acceleration Parameter at 1–s Period				
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \ge 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_1

For Site Class = D and S_1 = 0.917 g, F_v = 1.500

Equation (11.4–1): $S_{MS} = F_aS_S = 1.000 \times 2.293 = 2.293$ g

Equation (11.4–2): $S_{M1} = F_vS_1 = 1.500 \times 0.917 = 1.375$ g

Section 11.4.4 — Design Spectral Acceleration Parameters

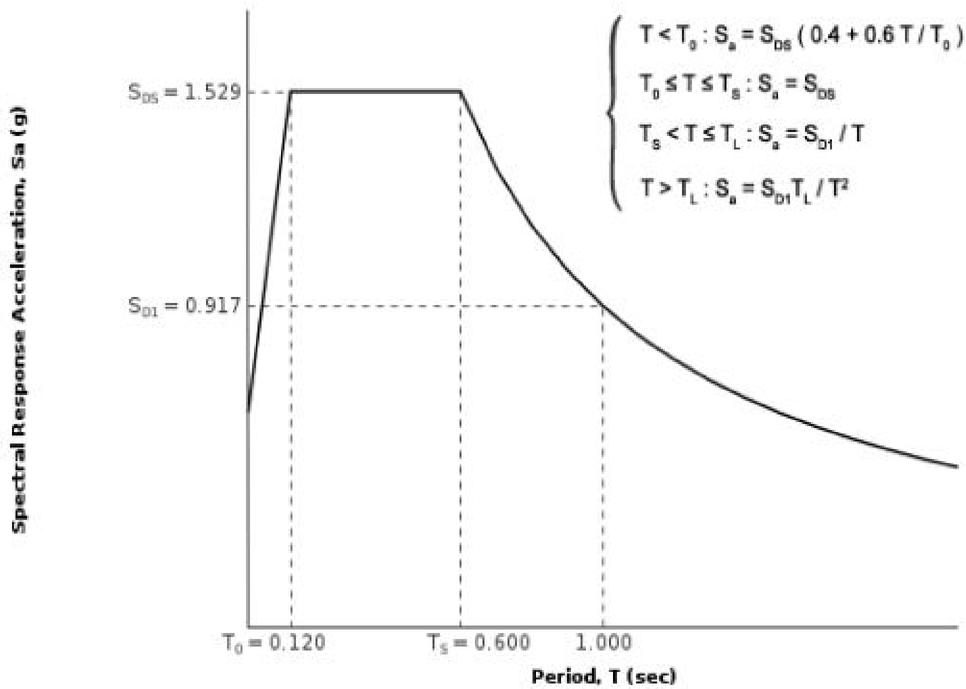
Equation (11.4–3): $S_{DS} = ⅔ S_{MS} = ⅔ \times 2.293 = 1.529$ g

Equation (11.4–4): $S_{D1} = ⅔ S_{M1} = ⅔ \times 1.375 = 0.917$ g

Section 11.4.5 — Design Response Spectrum

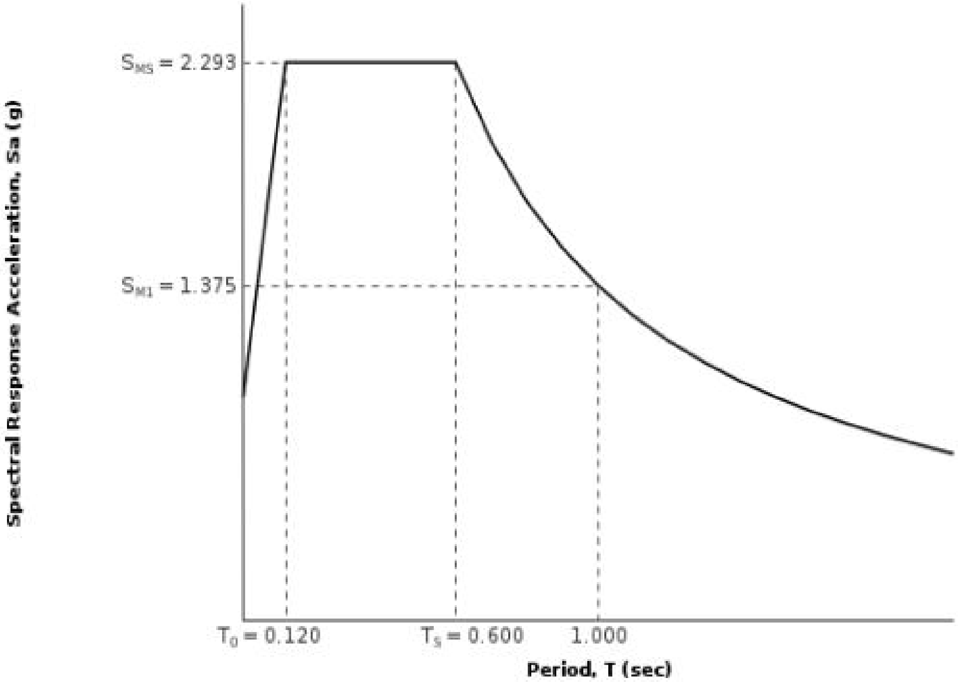
From Figure 22-12 [3] $T_L = 8$ seconds

Figure 11.4–1: Design Response Spectrum



Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4] PGA = 0.898

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 1.000 \times 0.898 = 0.898$ g

Table 11.8–1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of PGA

For Site Class = D and PGA = 0.898 g, F_{PGA} = 1.000

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 [5] C_{RS} = 0.922

From Figure 22-18 [6] C_{R1} = 0.911

Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
S_{DS} < 0.167g	A	A	A
0.167g ≤ S_{DS} < 0.33g	B	B	C
0.33g ≤ S_{DS} < 0.50g	C	C	D
0.50g ≤ S_{DS}	D	D	D

For Risk Category = I and S_{DS} = 1.529 g, Seismic Design Category = D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
S_{D1} < 0.067g	A	A	A
0.067g ≤ S_{D1} < 0.133g	B	B	C
0.133g ≤ S_{D1} < 0.20g	C	C	D
0.20g ≤ S_{D1}	D	D	D

For Risk Category = I and S_{D1} = 0.917 g, Seismic Design Category = D

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 11.6-1 or 11.6-2" = E

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 22-1*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. *Figure 22-2*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. *Figure 22-7*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. *Figure 22-17*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. *Figure 22-18*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf

APPENDIX E

LIQUEFACTION ANALYSIS

LIQUEFACTION & SETTLEMENT OF SANDS ANALYSIS

Project Name: Multi-Tenent Building
Project Number: 14396-10
Boring Number: B-2

Horizontal Ground Acceleration (% g)	0.917		Energy Ratio C_E (Auto-hammer)	1.50
Analyzed Groundwater Depth (feet)	5.0		Borehole Diameter C_B (6 - 8 inches)	1.00
Average Wet Unit Weight (pcf)	110.0		Groundwater Depth in Boring (feet)	17.5
Design Magnitude Earthquake	6.8			
Magnitude Scaling Factor (MSF)	1.3			

Depth (feet)	Blow Count SPT	Blow Count Cal. Mod.	SPT N_m	Total Stress (tons/ft2)	Effective Stress (tons/ft2)	Fines Content FC(%)	C_R	Overburden C_N	rd	Sampler Type C_S	$(N_1)_{60}$	$(N_1)_{60cs}$	NCEER 1998 CSR	NCEER 1998 CRR*MSF	Liquefaction Safety Factor	Layer Thickness t (ft)	Layer Thickness t (inches)	Percent Volumetric Strain	Settlement Per Sand Layer (inches)
7		7	5.292	0.385	0.323	30	0.75	1.39	0.98	1.00	8	14	0.70	0.1997	0.29	7.00	84.00	2.90	2.44
9		6	4.536	0.495	0.370	65	0.75	1.30	0.98	1.00	7	13	0.78	0.1833	Fine Grained, Not Liquifiable	2.00	24.00	3.50	Fine Grained
15		7	5.292	0.825	0.513	65	0.85	1.09	0.97	1.00	7	14	0.93	0.1940	Fine Grained, Not Liquifiable	6.00	72.00	3.00	Fine Grained
18		4	3.024	0.990	0.584	80	0.95	1.01	0.96	1.00	4	10	0.97	0.1508	Fine Grained, Not Liquifiable	3.00	36.00	4.50	Fine Grained
25	9		9.000	1.375	0.751	80	0.95	0.94	0.94	1.20	14	22	1.03	0.3236	Fine Grained, Not Liquifiable	7.00	84.00	2.00	Fine Grained
30		8	6.048	1.650	0.870	30	1.00	0.89	0.93	1.00	8	14	1.05	0.1975	0.19	5.00	60.00	2.90	1.74
33	27		27.000	1.815	0.941	15	1.00	0.87	0.91	1.20	42	47	1.04	---	Corrected SPT >30*	3.00	36.00	0.00	0.00
38	40		40.000	2.090	1.060	15	1.00	0.83	0.86	1.20	60	65	1.02	---	Corrected SPT >30*	5.00	60.00	0.00	0.00
43	32		32.000	2.365	1.179	15	1.00	0.79	0.82	1.20	46	50	0.98	---	Corrected SPT >30*	5.00	60.00	0.00	0.00
45	100		100.000	2.475	1.227	15	1.00	0.78	0.81	1.20	141	150	0.97	---	Corrected SPT >30*	2.00	24.00	0.00	0.00
																		Total Settlement (inches):	4.2

Procedure established by T.L. Youd and I.M. Idriss, et. al., 1996 NCEER-96-0022 Workshop & S.C.E.C. SP117
Evaluation of settlements in sand due to earthquake shaking, Tokimatsu and Seed, 1987

3 Extension of rod above boring (feet)

* CRR 7.5 is not defined for (N_1)60cs greater than 30. Soils with (N1)60cs > 30 are considered too dense to liquefy (NCEER Workshop)

$(N_1)_{60} = N_M C_N C_E C_B C_R C_S$ $(N_1)_{60CS} = K_S (N_1)_{60}$

LIQUEFACTION & SETTLEMENT OF SANDS ANALYSIS

Project Name: Multi-Tenent Building
Project Number: 14396-10
Boring Number: B-2 Compacted

Horizontal Ground Acceleration (% g)	0.917	Energy Ratio C_E (Auto-hammer)	1.50
Analyzed Groundwater Depth (feet)	5.0	Borehole Diameter C_B (6 - 8 inches)	1.00
Average Wet Unit Weight (pcf)	110.0	Groundwater Depth in Boring (feet)	17.5
Design Magnitude Earthquake	6.8		
Magnitude Scaling Factor (MSF)	1.3		

Depth (feet)	Blow Count SPT	Blow Count Cal. Mod.	SPT N_m	Total Stress (tons/ft2)	Effective Stress (tons/ft2)	Fines Content FC(%)	C_R	Overburden C_N	rd	Sampler Type C_S	$(N_1)_{60}$	$(N_1)_{60cs}$	NCEER 1998 CSR	NCEER 1998 CRR*MSF	Liquefaction Safety Factor	Layer Thickness t (ft)	Layer Thickness t (inches)	Percent Volumetric Strain	Settlement Per Sand Layer (inches)
7		30	22.680	0.385	0.323	30	0.75	1.39	0.98	1.00	35	46	0.70	---	Corrected SPT >30*	7.00	84.00	0.00	0.00
9		30	22.680	0.495	0.370	65	0.75	1.30	0.98	1.00	33	45	0.78	---	Corrected SPT >30*	2.00	24.00	0.00	Fine Grained
15		7	5.292	0.825	0.513	65	0.85	1.09	0.97	1.00	7	14	0.93	0.1940	Fine Grained, Not Liquifiable	6.00	72.00	3.00	Fine Grained
18		4	3.024	0.990	0.584	80	0.95	1.01	0.96	1.00	4	10	0.97	0.1508	Fine Grained, Not Liquifiable	3.00	36.00	4.50	Fine Grained
25	9		9.000	1.375	0.751	80	0.95	0.94	0.94	1.20	14	22	1.03	0.3236	Fine Grained, Not Liquifiable	7.00	84.00	2.00	Fine Grained
30		8	6.048	1.650	0.870	30	1.00	0.89	0.93	1.00	8	14	1.05	0.1975	0.19	5.00	60.00	2.90	1.74
33	27		27.000	1.815	0.941	15	1.00	0.87	0.91	1.20	42	47	1.04	---	Corrected SPT >30*	3.00	36.00	0.00	0.00
38	40		40.000	2.090	1.060	15	1.00	0.83	0.86	1.20	60	65	1.02	---	Corrected SPT >30*	5.00	60.00	0.00	0.00
43	32		32.000	2.365	1.179	15	1.00	0.79	0.82	1.20	46	50	0.98	---	Corrected SPT >30*	5.00	60.00	0.00	0.00
45	100		100.000	2.475	1.227	15	1.00	0.78	0.81	1.20	141	150	0.97	---	Corrected SPT >30*	2.00	24.00	0.00	0.00

Total Settlement (inches): 1.7

Procedure established by T.L. Youd and I.M. Idriss, et. al., 1996 NCEER-96-0022 Workshop & S.C.E.C. SP117
Evaluation of settlements in sand due to earthquake shaking, Tokimatsu and Seed, 1987

3 Extension of rod above boring (feet)

* CRR 7.5 is not defined for (N_1)60cs greater than 30. Soils with (N1)60cs > 30 are considered too dense to liquefy (NCEER Workshop)

$(N_1)_{60} = N_M C_N C_E C_B C_R C_S$ $(N_1)_{60CS} = K_S (N_1)_{60}$

CW SOILS

APPENDIX F

PAVEMENT DESIGN CALCULATIONS

PAVING DESIGN

PROJECT: **Multi-Tenant Building**

PROJECT NO.: **14396-10**

CONSULTANT: **CW**

CALCULATION SHEET NO.: **Auto Drives/Parking**

CALTRANS METHOD FOR DESIGN OF FLEXIBLE PAVEMENT

Input "R" value or "CBR" of native soil	50	
Type of Index Property - "R" value or "CBR" (C or R)	R	R Value
R Value used for Caltrans Method	50	
Input Traffic Index (TI)	5	
Calculated Total Gravel Equivalent (GE)	0.8	feet
Calculated Total Gravel Equivalent (GE)	9.6	inches
Calculated Gravel Factor (Gf) for A/C paving	2.53	
Gravel Factor for Base Course (Gf)	1.1	

Pavement sections provided below are considered equal; but, do not reflect reviewing agency minimums.

Gravel Equivalent GE (feet)	GE (inches)	Delta (inches)	INCHES A/C Section Thickness (inches)	INCHES Minimum Base (inches)	FEET A/C Section Thickness (feet)	FEET Minimum Base (feet)
0.63	7.60	2.00	3.0	1.8	0.25	0.15
0.74	8.87	0.73	3.5	0.6	0.29	0.05
0.84	10.14	-0.54	4.0		0.33	
1.06	12.67	-3.07	5.0		0.42	
1.27	15.21	-5.61	6.0		0.50	
1.48	17.74	-8.14	7.0		0.58	
1.69	20.28	-10.68	8.0		0.67	
1.90	22.81	-13.21	9.0		0.75	
2.11	25.35	-15.75	10.0		0.83	
2.32	27.88	-18.28	11.0		0.92	
2.53	30.42	-20.82	12.0		1.00	

PAVING DESIGN

PROJECT: **Multi-Tenant Building**

PROJECT NO.: **14396-10**

CONSULTANT: **CW**

CALCULATION SHEET NO.: **Entrances/Truck Drives**

CALTRANS METHOD FOR DESIGN OF FLEXIBLE PAVEMENT

Input "R" value or "CBR" of native soil	50	
Type of Index Property - "R" value or "CBR" (C or R)	R	R Value
R Value used for Caltrans Method	50	
Input Traffic Index (TI)	6	
Calculated Total Gravel Equivalent (GE)	0.96	feet
Calculated Total Gravel Equivalent (GE)	11.52	inches
Calculated Gravel Factor (Gf) for A/C paving	2.31	
Gravel Factor for Base Course (Gf)	1.1	

Pavement sections provided below are considered equal; but, do not reflect reviewing agency minimums.

Gravel Equivalent			INCHES		FEET	
GE (feet)	GE (inches)	Delta (inches)	A/C Section Thickness (inches)	Minimum Base (inches)	A/C Section Thickness (feet)	Minimum Base (feet)
0.58	6.94	4.58	3.0	4.2	0.25	0.35
0.67	8.10	3.42	3.5	3.0	0.29	0.25
0.77	9.26	2.26	4.0	1.8	0.33	0.15
0.96	11.57	-0.05	5.0		0.42	
1.16	13.88	-2.36	6.0		0.50	
1.35	16.20	-4.68	7.0		0.58	
1.54	18.51	-6.99	8.0		0.67	
1.74	20.83	-9.31	9.0		0.75	
1.93	23.14	-11.62	10.0		0.83	
2.12	25.45	-13.93	11.0		0.92	
2.31	27.77	-16.25	12.0		1.00	

APPENDIX G

GENERAL EARTHWORK AND GRADING SPECIFICATIONS

CW SOILS

General Earthwork and Grading Specifications

General

Intent: The following General Earthwork and Grading Specifications are intended to provide minimum requirements for grading operations and earthwork. These General Earthwork and Grading Specifications should be considered a part of the recommendations contained in the geotechnical report(s). If they are in conflict with the geotechnical report(s), the specific recommendations in the geotechnical report shall supersede these more general specifications. Observations made during earthwork operations by the Geotechnical Consultant may result in new or revised recommendations that may supersede these specifications and/or the recommendations in the geotechnical report(s).

The Geotechnical Consultant of Record: The Owner shall retain a qualified Consultant of Record (Geotechnical Consultant), prior to commencement of grading operations or construction. The Geotechnical Consultant shall be responsible for reviewing the approved geotechnical report(s) and accepting the adequacy of the preliminary geotechnical findings, conclusions, and recommendations prior to the commencement of the grading operations or construction.

Prior to commencement of grading operations or construction, the Owner shall coordinate with the Geotechnical Consultant, and Earthwork Contractor (Contractor) to schedule sufficient personnel for the appropriate level of observation, mapping, and compaction testing.

During earthwork and grading operations, the Geotechnical Consultant shall observe, map, and document the subsurface conditions to confirm assumptions made during the geotechnical design phase of the project. Should the actual conditions differ significantly from the interpretive assumptions made during the design phase, the Geotechnical Consultant shall recommend appropriate changes to accommodate the actual conditions, and notify the reviewing agency as needed.

The Geotechnical Consultant shall observe the moisture conditioning and processing of the excavations and fill operations. The Geotechnical Consultant should perform periodic compaction testing of engineered fills to verify that the required level of compaction is being accomplished as specified.

The Earthwork Contractor: The Earthwork Contractor (Contractor) shall be qualified, experienced, and knowledgeable in earthwork logistics, preparation and processing of excavations to receive compacted fill, moisture conditioning, processing of fill, and compacting fill. The Contractor shall be provided with the approved grading plans and geotechnical report(s) for his review and acceptance of responsibilities, prior to commencement of grading. The Contractor shall be solely responsible for performing the grading in accordance with the approved grading plans and geotechnical report(s). The Contractor shall inform the Owner and the Geotechnical Consultant of work schedule changes at least 24 hours in advance of such changes so that appropriate personnel will be available for observation and testing. Assumptions shall not be made by the Contractor with regard to whether the Geotechnical Consultant is aware of all grading operations.

It is the sole responsibility of the Contractor to provide adequate equipment and methods to accomplish the grading operations in accordance with the applicable grading codes and agency ordinances, these specifications, and the recommendations in the approved grading plan(s) and geotechnical report(s). Any unsatisfactory conditions, such as unsuitable soils, poor moisture conditioning, inadequate compaction, insufficient buttress keyway size, adverse weather conditions, etc., resulting in a quality of work less than required in the approved grading plans and geotechnical report(s), the Geotechnical Consultant shall reject the work and may recommend to the Owner that grading operations be stopped until operations are corrected, at the sole discretion of the Geotechnical Consultant.

Preparation of Areas for Compacted Fill

Clearing and Grubbing: Vegetation, such as brush, grass, roots, and other deleterious materials shall be sufficiently removed and properly disposed in a method acceptable to the Owner, Geotechnical Consultant, and governing agencies.

The Geotechnical Consultant shall evaluate the extent of these removals on a case by case basis. Soils to be placed as compacted fill shall not contain more than 1 percent organic materials (by volume). No compacted fill lift shall contain more than 10 percent organic matter.

If potentially hazardous materials are encountered, the Contractor shall stop work and exit the affected area, and a hazardous materials specialist shall immediately be consulted to evaluate the potentially hazardous materials, prior to continuing to work in that area.

It is our understanding that the State of California defines most refined petroleum products (gasoline, diesel fuel, motor oil, grease, coolant, etc.) as hazardous waste. As such, indiscriminate dumping or spillage of these fluids may constitute a misdemeanor, punishable by fines and/or imprisonment, and shall be prohibited.

The contractor is responsible for all hazardous waste related to his operations. The Geotechnical Consultant does not have expertise in this area. If hazardous waste is a concern, then the Owner should contract the services of a qualified environmental assessor.

Processing: Exposed soils that have been observed to be satisfactory for support of compacted fill by the Geotechnical Consultant shall be scarified to a minimum depth of 6 inches. Exposed soils that are not satisfactory shall be removed or alternative recommendations may be provided by the Geotechnical Consultant. Scarification shall continue until the exposed soils are free of oversize material and the working surface is reasonably uniform, flat, and free of uneven features that would inhibit uniform compaction. The soils should be moistened or air dried as necessary to achieve near optimum moisture content, prior to placement as engineered fill.

Overexcavation: The Typical Cut Lot Detail and Typical Cut/Fill Transition Lot Detail, included herein provide graphic illustrations that depicts typical overexcavation recommendations made in the approved grading plan(s) and/or geotechnical report(s).

Keyways and Benching: Where fills are to be placed on slopes steeper than 5:1 (horizontal to vertical), the ground shall be thoroughly benched as compacted fill is placed. Please see the three Typical Keyway and Benching Details with subtitles Cut Over Fill Slope, Fill Over Cut Slope, and Fill Slope for graphic illustrations. The lowest bench or smallest keyway shall be a minimum of 15 feet wide (or ½ the proposed slope height) and at least 2 feet into competent soils as advised by the Geotechnical Consultant. Typical benching shall be excavated a minimum height of 4 feet into competent soils or as recommended by the Geotechnical Consultant. Fill placed on slopes steeper than 5:1 should be thoroughly benched or otherwise excavated to provide a flat subgrade for the compacted fill. If unstable earth materials are encountered or anticipated the need for a buttress/stabilization fill may be required, see Typical Buttress/ Stabilization Detail herein.

Evaluation/Acceptance of Bottom Excavations: All areas to receive compacted fill (bottom excavations), including removal excavations, processed areas, keyways, and benching, shall be observed, mapped, general elevations recorded, and/or tested prior to being accepted by the Geotechnical Consultant as suitable to receive compacted fill. The Contractor shall obtain a written acceptance from the Geotechnical Consultant prior to placing compacted fill. A licensed surveyor shall provide the survey control for determining elevations of bottom excavations, processed areas, keyways, and benching. The Geotechnical Consultant is not responsible for erroneously located, fills, subdrain systems, or excavations.

Fill Materials

General: Soils to be used as compacted fill should be relatively free of organic matter and other deleterious substances as evaluated and accepted by the Geotechnical Consultant.

Oversize: Oversize material is rock that does not break down into smaller pieces and has a maximum diameter greater than 12 inches. Oversize rock shall not be included within compacted fill unless specific methods and guidelines acceptable to the Geotechnical Consultant are followed. For examples of methods and guidelines of oversize rock placement see the enclosed Typical Oversize Rock Disposal Detail. The inclusion of oversize materials in the compacted fill shall only be acceptable if the oversize material is completely surrounded by compacted fill or thoroughly jetted granular materials. No oversize material shall be placed within 10 vertical feet of finish grade or within 2 feet of proposed utilities or underground improvements.

Import: Should imported soils be required, the proposed import materials shall meet the requirements of the Geotechnical Consultant. Well graded, very low expansion potential soils free of organic matter and other deleterious substances are usually the most desirable as import materials. It is generally in the Owners best interest that potential import soils are provided to the Geotechnical Consultant to determine their suitability for the intended purpose. Prior to starting import operations, at least 48 hours should be allotted for the appropriate laboratory testing to be performed.

Fill Placement and Compaction Procedures

Fill Layers: Fill materials shall be placed in areas prepared to receive engineered fill in nearly horizontal layers not exceeding 8 inches in loose thickness. Thicker layers may be accepted by the Geotechnical Consultant, provided field density testing indicates that the grading procedures can obtain adequate compaction. Each layer of fill shall be spread evenly and thoroughly mixed to obtain uniformity within the soils along with a consistent moisture throughout the fill.

Moisture Conditioning of Fill: Soils to be placed as compacted fill shall be watered, dried, blended, and/or mixed, as needed to obtain relatively uniform moisture contents that are at or slightly above optimum. The maximum density and optimum moisture content tests should be performed using the guidelines of the American Society of Testing and Materials (ASTM test method D1557-00).

Compaction of Fill: After each layer has been moisture conditioned, mixed, and evenly spread, it should be uniformly compacted to a minimum of 90 percent of the maximum dry density as determined by ASTM test method D1557-00. Compaction equipment shall be adequately sized and be either specifically designed for compaction of soils or be proven to consistently achieve the required level of compaction.

Compaction of Fill Slopes: In addition to normal compaction procedures specified above, additional effort to obtain compaction on slopes is needed. This may be accomplished by backrolling of slopes with sheepsfoot rollers as the fill is being placed, by overbuilding the fill slopes, or by other methods producing results that are satisfactory to the Geotechnical Consultant. Upon completion of grading, compaction of the fill and the slope face shall be a minimum of 90 percent of maximum density per ASTM test method D1557-00.

Compaction Testing of Fill: Field tests for moisture content and density of the compacted fill shall be periodically performed by the Geotechnical Consultant. The location and frequency of tests shall be at the Geotechnical Consultant's discretion. Compaction test locations will not necessarily be random. The test locations may or may not be selected to verify minimum compaction requirements in areas that are typically prone to inadequate compaction, such as close to slope faces and near benching.

Frequency of Compaction Testing: Compaction tests shall be taken at minimum intervals of every 2 vertical feet and/or per 1,000 cubic yards of compacted materials placed. Additionally, as a guideline, at least one (1) test shall be taken on slope faces for each 5,000 square feet of slope face and/or for each 10 vertical feet of slope. The Contractor shall assure that fill placement is such that the testing schedule described herein can be accomplished by the Geotechnical Consultant. The Contractor shall stop or slow down the earthwork operations to a safe level so that these minimum standards can be obtained.

Compaction Test Locations: The approximate elevation and horizontal coordinates of each test location shall be documented by the Geotechnical Consultant. The Contractor shall coordinate with the Surveyor to assure that sufficient grade stakes are established. This will provide the Geotechnical Consultant with the ability to determine the approximate test locations and elevations. The Geotechnical Consultant can not be responsible for staking erroneously located by the Surveyor or Contractor. A minimum of two grade stakes should be provided at a maximum horizontal distance of 100 feet and vertical difference of less than 5 feet.

Subdrain System Installation

Subdrain systems shall be installed in accordance with the approved geotechnical report(s), the approved grading plan(s), and the typical details provided herein, such as the Typical

Canyon Subdrain System Detail, etc. The Geotechnical Consultant may recommend additional subdrain systems and/or changes to the subdrain systems described herein, with regard to the extent, location, grade, or materials depending on conditions observed during grading or other factors. All subdrain systems shall be surveyed by a licensed land surveyor, with the exception of retaining wall subdrain systems, to verify line and grade after installation and prior to burial. Adequate time should be allowed by the Contractor to complete these surveys.

Excavation

All excavations and overexcavations shall be evaluated by the Geotechnical Consultant during grading operations. Any remedial removal depths indicated on the geotechnical maps are estimates only. The actual removal depths and extent shall be determined by the Geotechnical Consultant based on the field observations of exposed conditions during grading operations. Where fill over cut slopes are planned, the cut portion of the slope shall be excavated, evaluated, and accepted by the Geotechnical Consultant prior to placement of the fill portion of the proposed slope, unless specifically addressed by the Geotechnical Consultant. Typical details for cut over fill slopes and fill over cut slopes are provided herein. Foundation excavations should be made in accordance with the Foundation Clearances from Slopes Detail unless otherwise specified by the site specific recommendations by the Geotechnical Consultant.

Trench Backfill

1) The Contractor shall follow all OHSA and Cal/OSHA requirements for trench excavation safety.

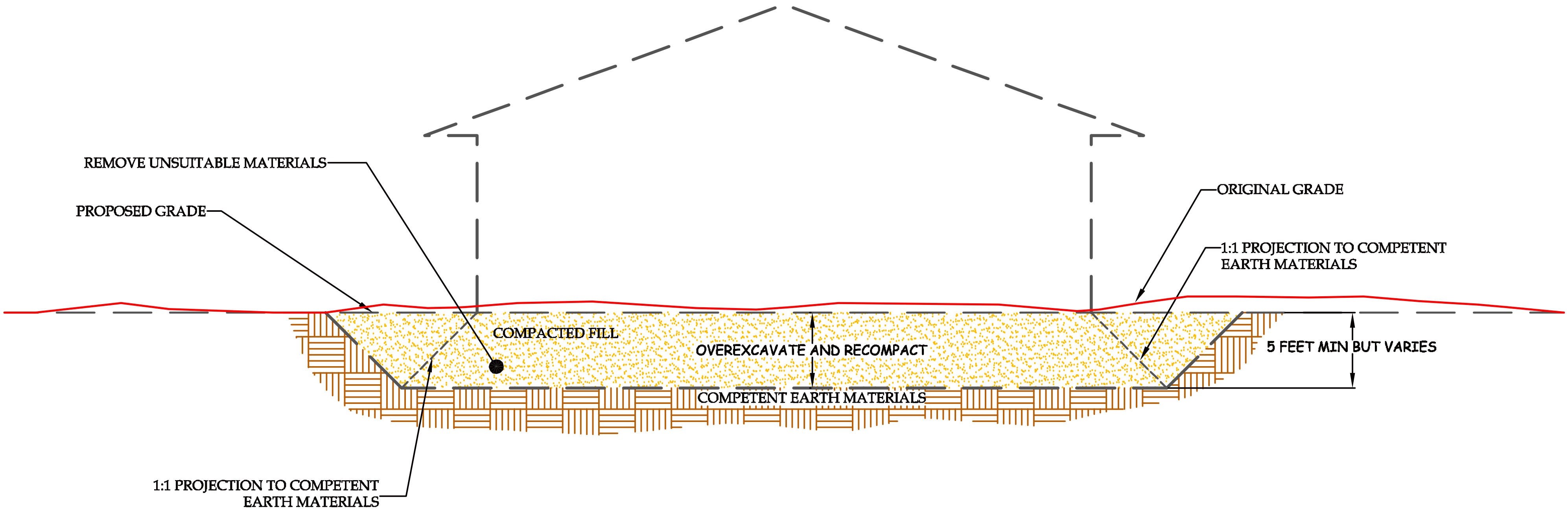
2) Bedding and backfill of utility trenches shall be done in accordance with the applicable provisions in the Standard Specifications of Public Works Construction. Bedding materials shall have a Sand Equivalency more than 30 (SE>30). The bedding shall be placed to 1 foot over the conduit and thoroughly jetting to provide densification. Backfill should be compacted to a minimum of 90 percent of maximum dry density, from 1 foot above the top of the conduit to the surface.

3) Jetting of the bedding materials around the conduits shall be observed by the Geotechnical Consultant.

4) The Geotechnical Consultant shall test trench backfill for the minimum compaction requirements recommended herein. At least one test should be conducted for every 300 linear feet of trench and for each 2 vertical feet of backfill.

5) For trench backfill the lift thicknesses shall not exceed those allowed in the Standard Specifications of Public Works Construction, unless the Contractor can demonstrate to the Geotechnical Consultant that the fill lift can be compacted to the minimum compaction requirements by the alternative equipment or method.

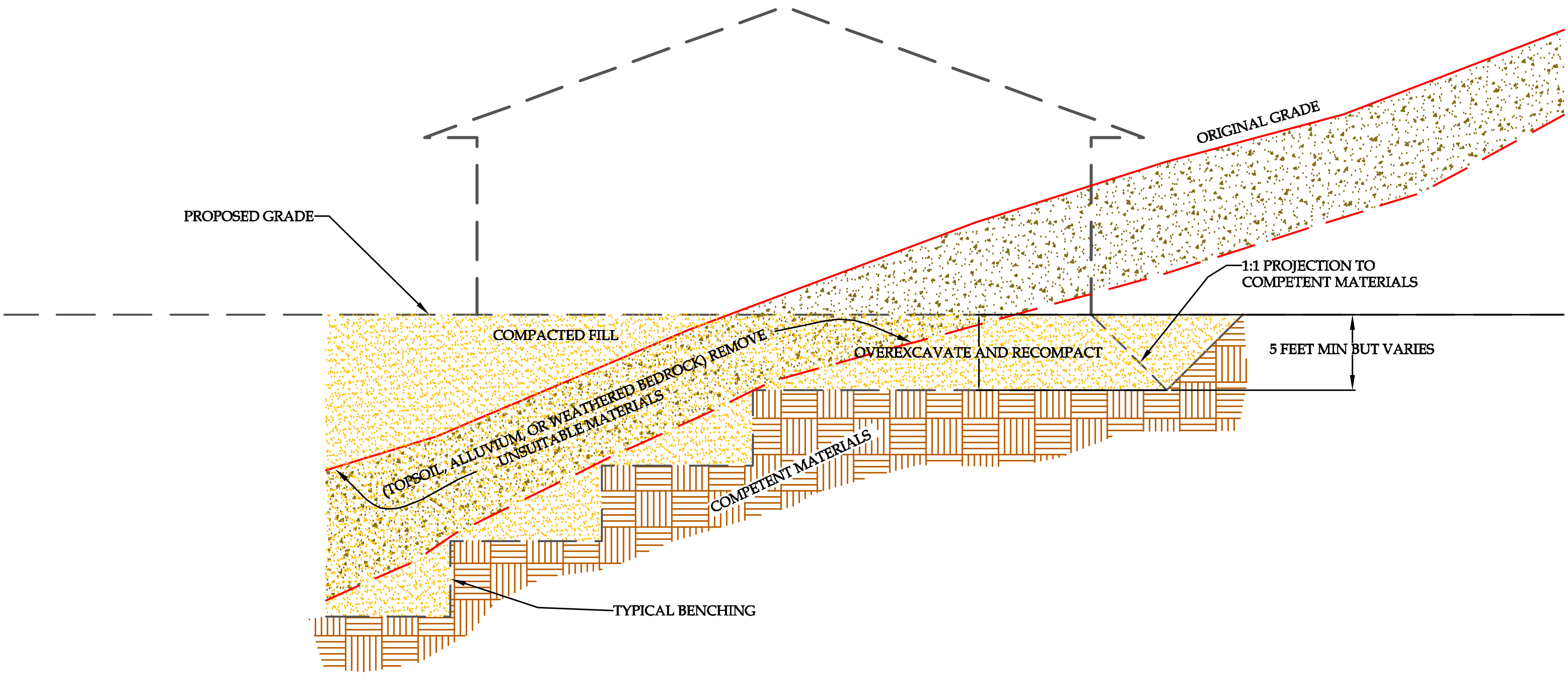
TYPICAL CUT LOT DETAIL



NOTE; REMOVAL BOTTOMS SHOULD BE GRADED WITH A MINIMUM 2% FALL TOWARDS STREET OR OTHER SUITABLE AREA (AS DETERMINED BY THE GEOTECHNICAL CONSULTANT) TO AVOID PONDING BELOW THE BUILDING

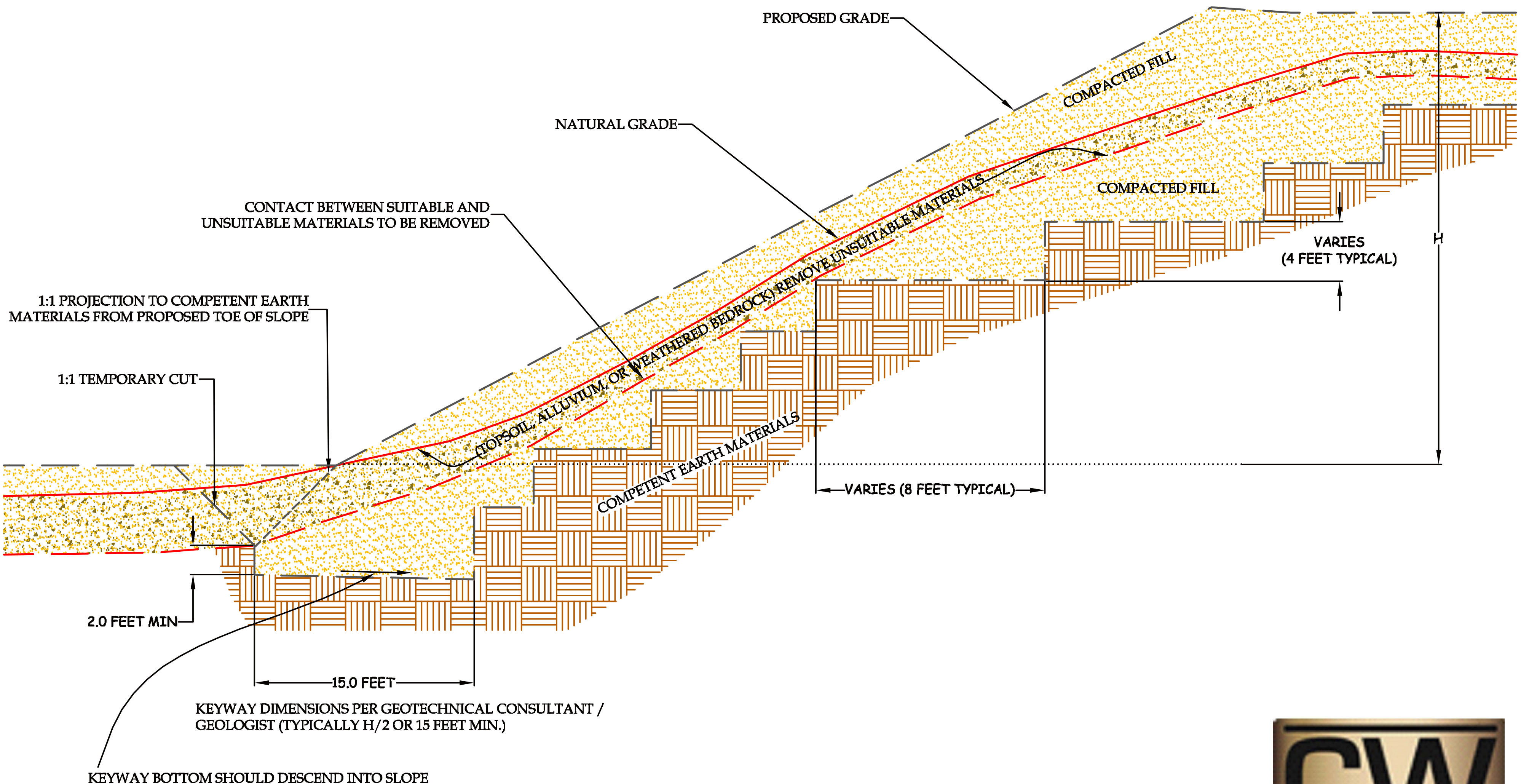
NOTE: WHERE DESIGN CUT LOTS ARE EXCAVATED ENTIRELY INTO COMPETENT EARTH MATERIALS, OVEREXCAVATION MAY STILL BY NEEDED FOR HARD-ROCK CONDITIONS OR MATERIALS WITH VARIABLE EXPANSION POTENTIALS

TYPICAL CUT / FILL TRANSITION LOT DETAIL



NOTE: WHERE DESIGN CUT LOTS ARE EXCAVATED ENTIRELY INTO COMPETENT MATERIALS, OVEREXCAVATION MAY STILL BY NEEDED FOR HARD-ROCK CONDITIONS OR MATERIALS WITH VARIABLE EXPANSION POTENTIALS

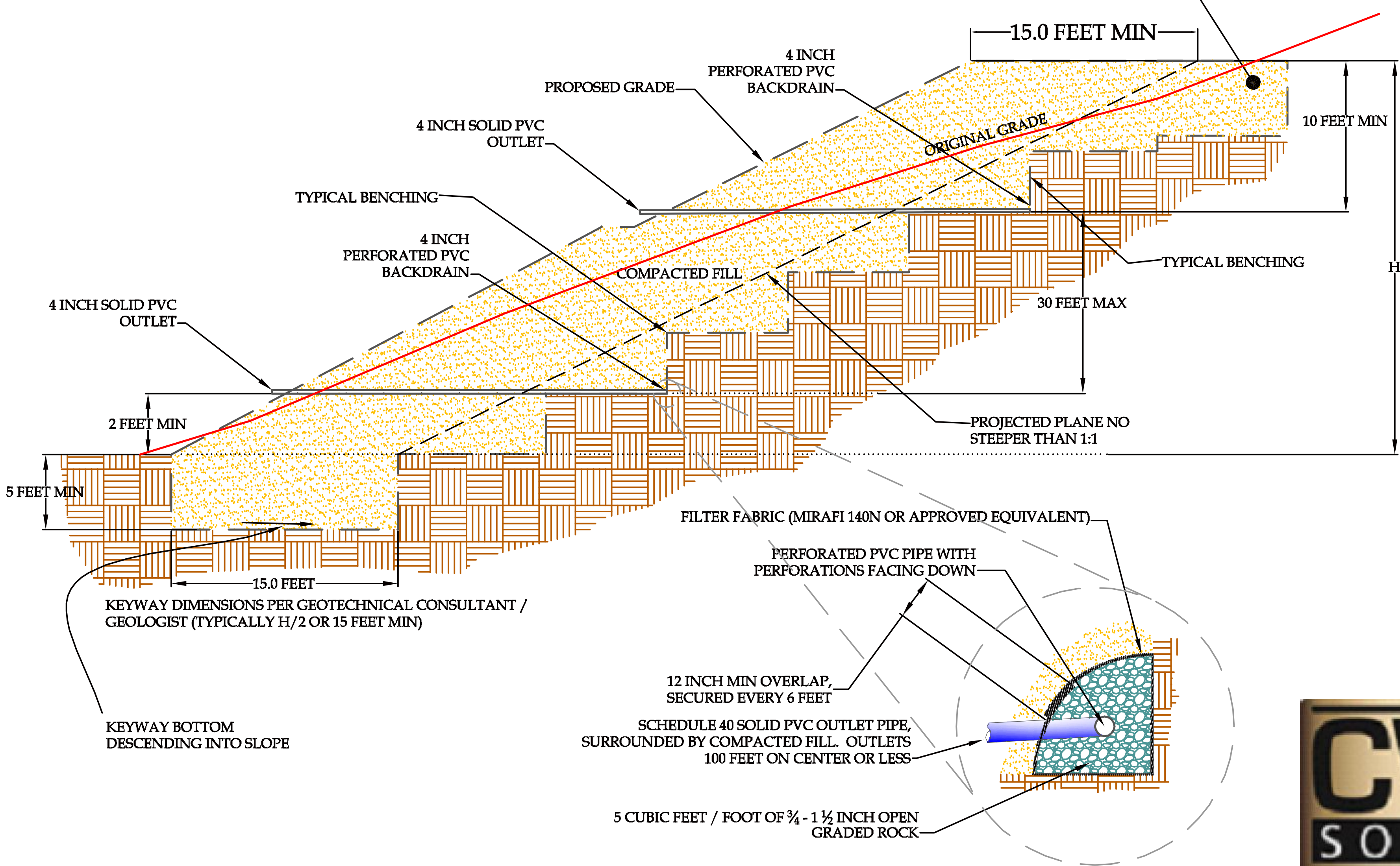
TYPICAL KEYWAY & BENCHING DETAIL
FILL SLOPE



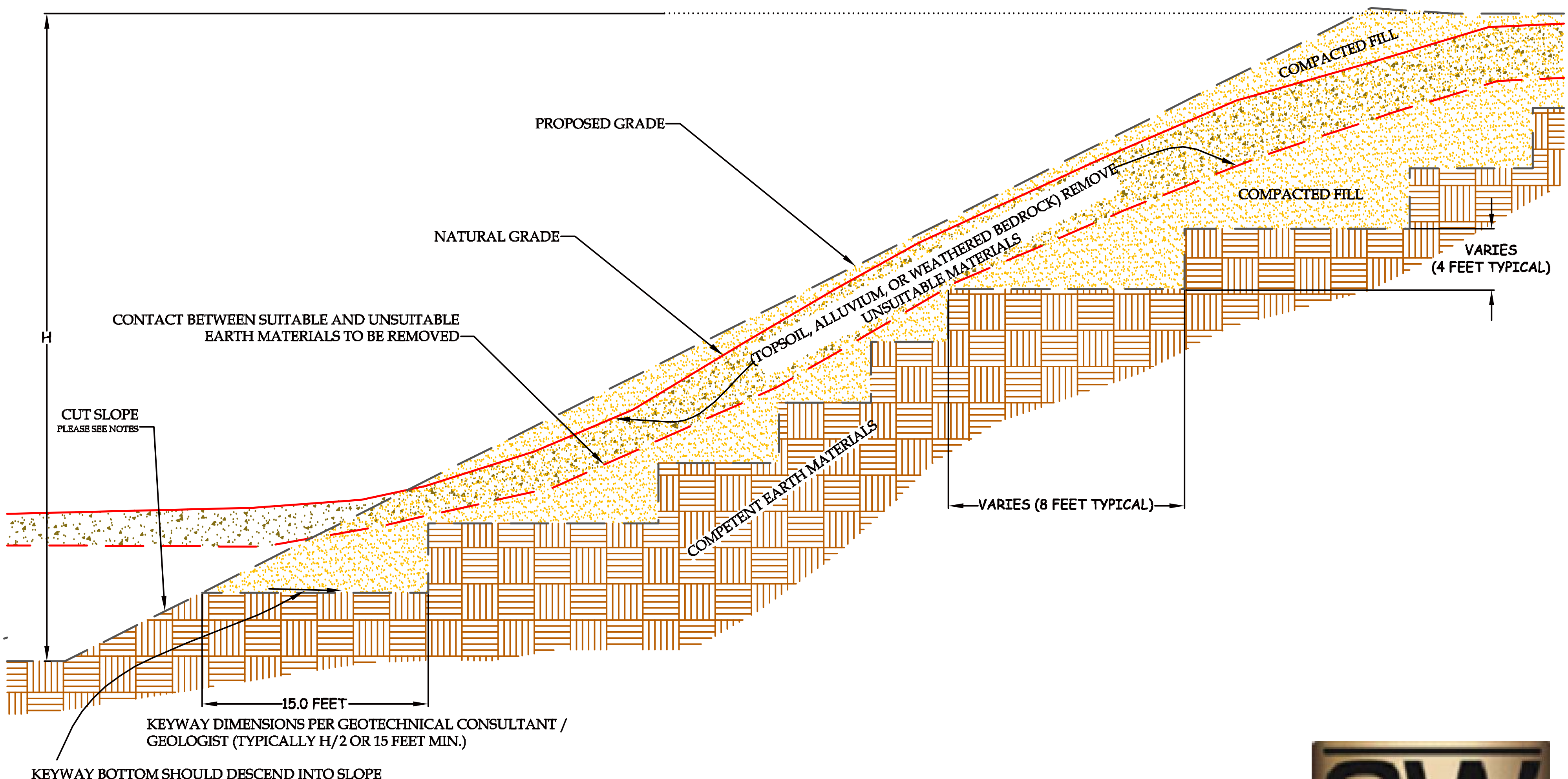
NOTES:

NATURAL SLOPES STEEPER THAN 5:1 (H:V) MUST BE KEYED AND BENCHED INTO COMPETENT EARTH MATERIALS

TYPICAL BUTTRESS / STABILIZATION DETAIL



TYPICAL KEYWAY & BENCHING DETAIL
FILL OVER CUT SLOPE



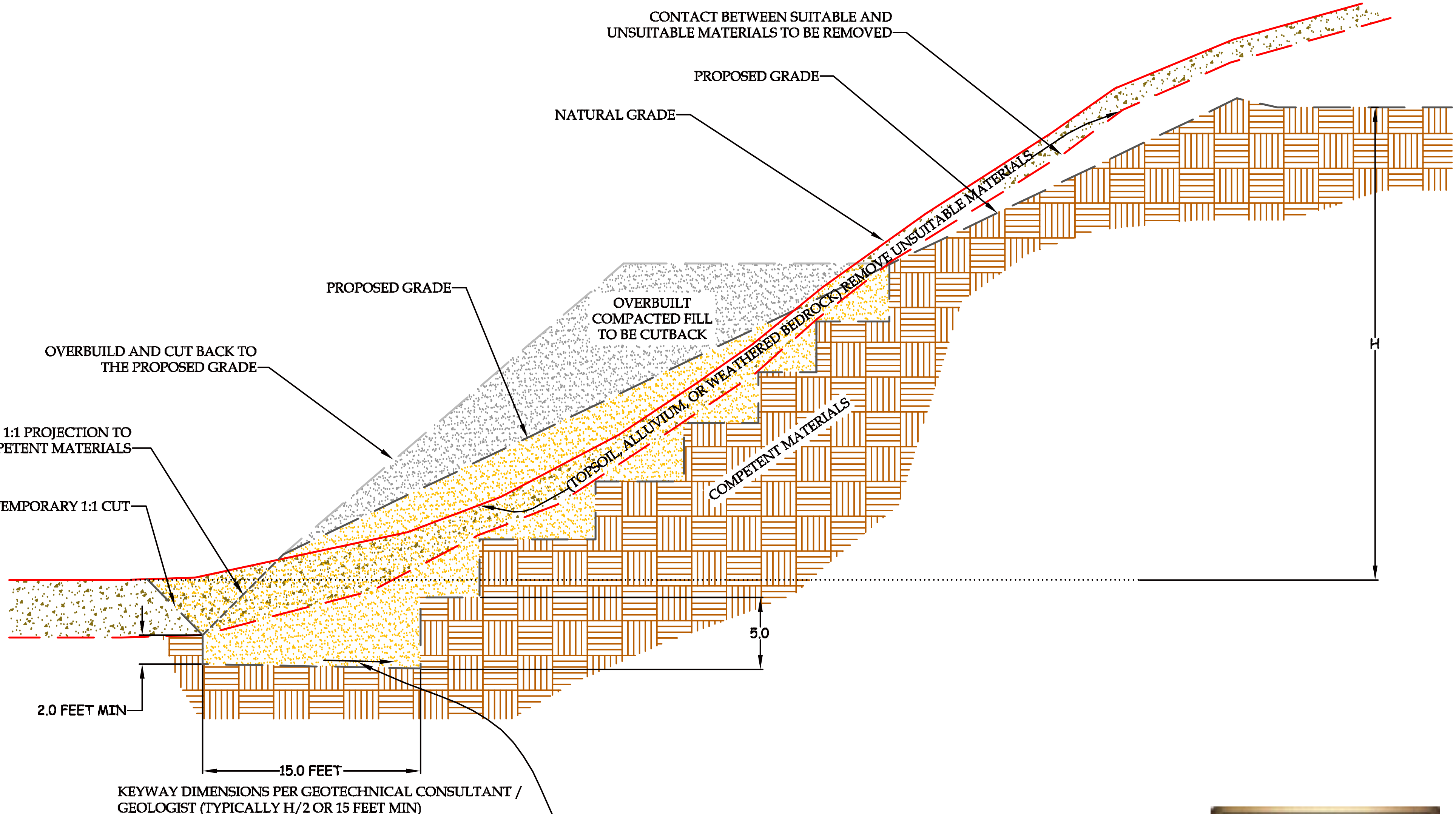
NOTES:

NATURAL SLOPES STEEPER THAN 5:1 (H:V) MUST BE KEYED AND BENCHED INTO COMPETENT EARTH MATERIALS

THE CUT SLOPE MUST BE CONSTRUCTED FIRST

CW SOILS

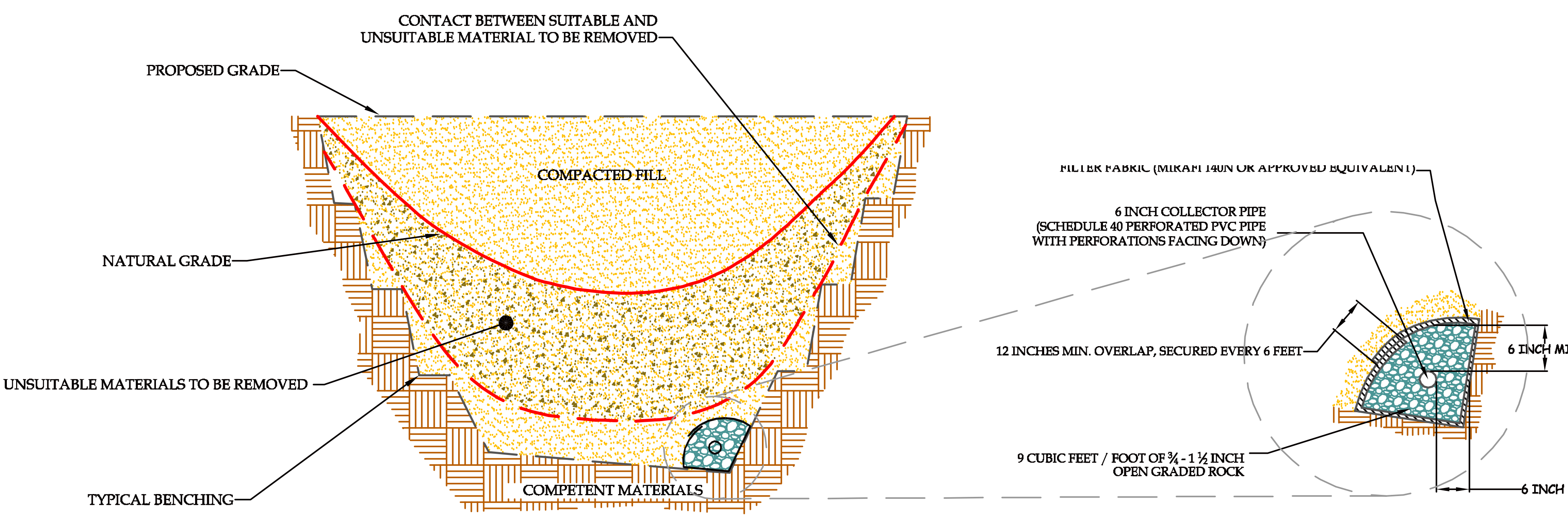
TYPICAL KEYWAY & BENCHING DETAIL
CUT OVER FILL SLOPE



NOTE:

NATURAL SLOPES STEEPER THAN 5:1 (H:V) MUST BE KEYED AND BENCHED INTO COMPETENT MATERIALS

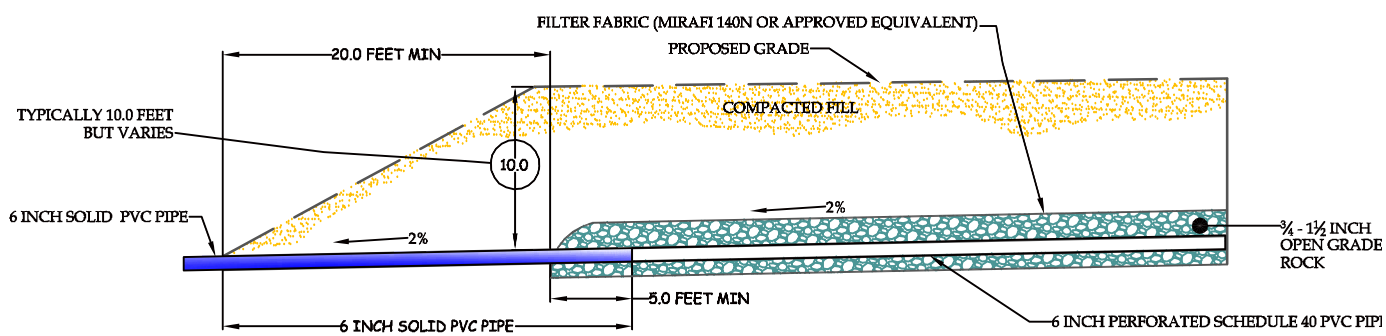
TYPICAL CANYON SUBDRAIN SYSTEM DETAIL



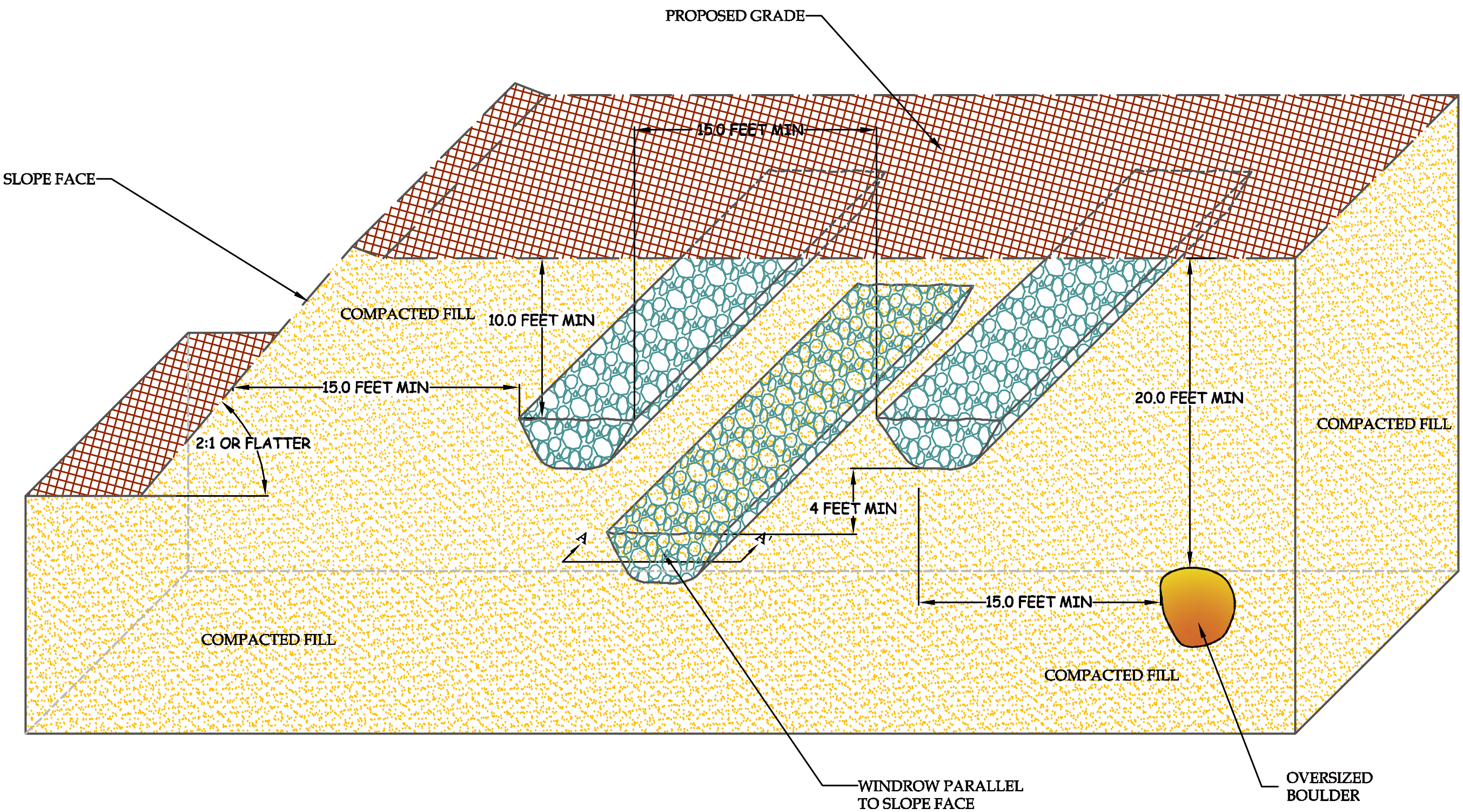
NOTES:
1 - CONTINUOUS RUNS IN EXCESS OF 500 FEET LONG WILL REQUIRE AN 8 INCH DIAMETER PIPE

2 - FINAL 20 FEET OF PIPE AT OUTLET WILL BE SOLID AND BACKFILLED WITH COMPACTED FINE-GRAINED MATERIALS

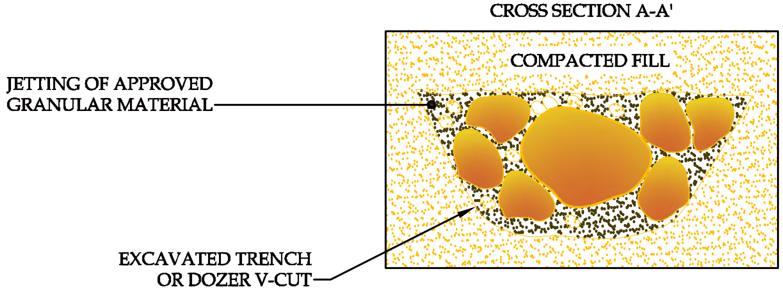
TYPICAL CANYON SUBDRAIN OUTLET



TYPICAL OVERSIZE ROCK DETAIL



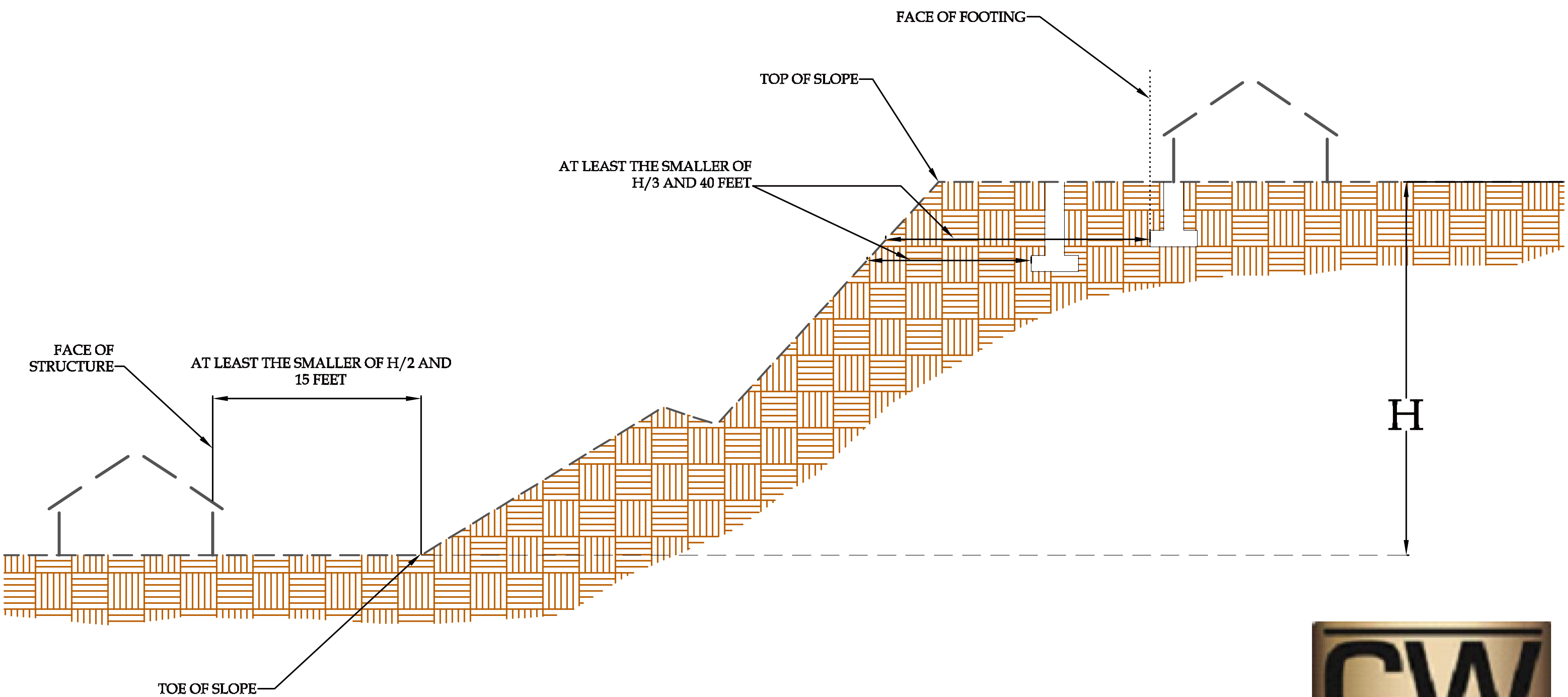
CROSS SECTION A-A'



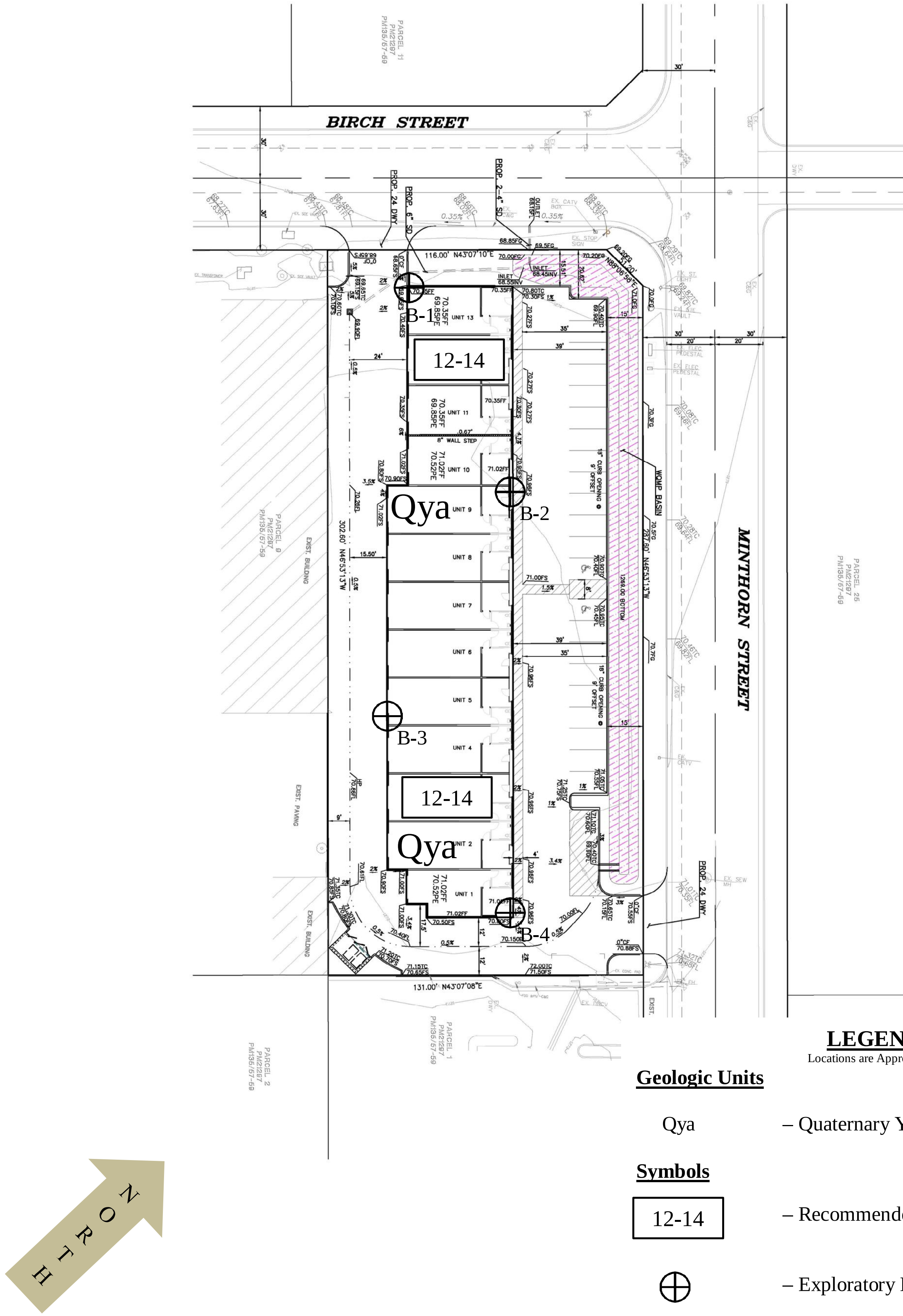
NOTE:

OVERSIZE ROCK IS LARGER THAN
12 INCHES IN MAX DIAMETER

FOUNDATION CLEARANCES FROM SLOPES DETAIL



(CBC, 2010)



LEGEND

Locations are Approximate

Geologic Units

Qya – Quaternary Young Alluvium

Symbols

$\boxed{12\text{-}14}$ – Recommended Removal Depth (feet)

B-4 – Exploratory Boring

REFERENCE: HLC, 2015, *Conceptual Grading Plan, Parcel 10 of PM21297*, dated March 16, 2015.

CW SOILS	Proposed Multi-Tenant Building		14396-10
	GEOTECHNICAL MAP	1" = 60 '	
		2015	PLATE 1