GEOCON
INCORPORATED

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

GEOTECHNICAL INVESTIGATION

MOXY HOTEL
835 SIXTH AVENUE
SAN DIEGO, CALIFORNIA

PREPARED FOR

GASLAMP QUARTER PROPERTIES
SAN DIEGO, CALIFORNIA

JUNE 26, 2015
PROJECT NO. G1863-52-01

GEOCON
INCORPORATED
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. G1863-52-01
June 26, 2015

Gaslamp Quarters Properties
515 Fifth Avenue, Suite 200
San Diego, California 92101

Attention: Mr. Alex Beaton

Subject: GEOTECHNICAL INVESTIGATION
MOXY HOTEL
835 SIXTH AVENUE
SAN DIEGO, CALIFORNIA

Dear Mr. Beaton:

In accordance with your request and our Proposal No. LG-14419 dated June 9, 2015 we prepared this geotechnical investigation report for the proposed hotel located in the downtown area of the City of San Diego, California. The accompanying report presents the findings of our study, and our conclusions and recommendations pertaining to the geotechnical aspects of proposed design and construction. Based on the results of our investigation, it is our opinion that the site can be developed as proposed provided the recommendations of this report are followed and implemented during design and construction.

Should you have questions regarding this report, or if we may be of further service, please contact the undersigned at your convenience.

Very truly yours,

GEOCON INCORPORATED

Lilian E. Rodriguez
RCE 83227

Shawn Foy Weedon
GE 2714

John Hoobs
CEG 1524

REGISTERED PROFESSIONAL ENGINEER LILIAN E. RODRIGUEZ No.83227 CIVIL STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER SHAWN WEEDON No. 2714 GEOTECHNICAL STATE OF CALIFORNIA

PROFESSIONAL GEOLOGIST JOHN HOOBS No. 1524 CERTIFIED ENGINEERING GEOLOGIST STATE OF CALIFORNIA

LER:SFW:JH:dmc

(e-mail) Addressee

TABLE OF CONTENTS

TABLE OF CONTENTS (Concluded)

GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation for the proposed hotel located at 835 Sixth Avenue in downtown San Diego, California (see Vicinity Map, Figure 1). The purpose of this geotechnical investigation is to evaluate the surface and subsurface soil conditions and general site geology and to identify geotechnical constraints that may impact development of the property. We previously performed a fault study to evaluate if faults traverse the subject property. The site is located within the City of San Diego Downtown Special Fault Zone and requires a detailed fault evaluation to satisfy the City of San Diego Building Department requirements.

The scope of this investigation included performing a site reconnaissance, a review of previous geotechnical reports and readily available published and unpublished geologic literature (see *Nisl oJ References),* engineering analyses, and the preparation of this report. The scope of this investigation also included a review of the following geotechnical reports:

1. *Geotechnical Recommendations for Mechanically Stabilized Earth Retaining Wall and Preliminary Pavement Design, Central Apartments, 831-845 Sixth Avenue, San Diego, California,* prepared by Geocon Incorporated, dated May 1, 2008 (Project No. 07980-52-01).

2. *Preliminary Geotechnical and Geologic Fault Investigation, Gaslamp Apartments 831 through 845 Sixth Avenue, San Diego, California,* prepared by Geocon Incorporated, dated July 10, 2008 (Project No. 07980-52-01).

3. *Report of Testing and Observation Services Performed During Site Grading and Improvements, Temporary Parking Lot, Gaslamp Apartments, 831-845 Sixth Avenue, San Diego, California,* prepared by Geocon Incorporated, dated August 11, 2008 (Project No. 07980-52-01).

Recommendations presented herein are based on analysis of data obtained from our previous site investigation, previous grading, and our understanding of proposed site development. References reviewed to prepare this report are provided in the *List of References.* If project details vary significantly from those described herein, Geocon should be contacted to evaluate the necessity for review and possible revision of this report.

2. PREVIOUS SITE DEVELOPMENT

The site previously housed a commercial structure which was demolished in 2008. Portions of the basement walls and columns of the previous structure were left in place along the perimeter of the basement excavation, and the remainder of the excavation was filled to approximately street grade by placing compacted fill imported to the site. Concrete slurry was used to fill void spaces beneath the sidewalk along Sixth Avenue. A mechanically stabilized earth (MSE) retaining wall consisting of geogrid reinforcement (without the concrete block facing units) was constructed along the northern portion of the site to help prevent imposing loads acting on the northern structure basement wall. Currently, the site has been graded and paved with asphalt concrete for use as a temporary parking lot until the proposed development occurs.

Geocon Incorporated prepared a preliminary geotechnical and geologic fault investigation for the subject site dated July 10, 2008 (see List of References). The previous geotechnical investigation included excavating a fault trench approximately 95 feet long across the site and drilling two exploratory borings to a maximum depth of approximately 71 U2 feet. The locations of the fault trench and borings are presented on Figure 2, Geologic Map. The fault trench log, the exploratory boring logs, and details of the field investigation are presented in Appendix A. We performed laboratory tests on selected soil samples obtained during the field investigation to evaluate pertinent physical and chemical properties for engineering analyses and to assist in providing recommendations for site grading and foundation design criteria. Details of the laboratory tests and a summary of the test results are presented in Appendix B and on the boring logs in Appendix A.

Geocon Incorporated also performed testing and observation services during the placement of fi11 during grading operations, the construction of the MSE wall, and the placement of subgrade and base materials from May through July of 2008 as discussed in the referenced report dated August 11, 2008. During grading operations and construction of the temporary asphalt parking lot, we performed laboratory tests on samples of fil1 and base materials to evaluate maximum dry density and optimum moisture content. The results of the laboratory tests are summarized in Appendix B.

3. SITE AND PROJECT DESCRIPTION

The project site is located on the east side of Sixth Avenue, south of E Street and north of F Street. The subject site consists of an existing temporary parking lot that is approximately 75 feet wide by 100 feet long. The site is bordered to the north by an 8-story commercial building, to the east by an additional parking area, to the south by a 6-story hotel, and to the west by Sixth Avenue. We understand the structures to the north and south possess 1 subterranean level each. The property is at an approximate elevation of 45 feet above Mean Sea Level (MSL) and is relatively flat.

We understand that the proposed development will consist of the construction of a hotel structure with potential subterranean levels. We expect the lowest subterranean level, if any will be approximately 25 feet deep below the street elevation. We expect shoring will likely be required for any subterranean excavation and underpinning of the northern and southern adjacent buildings may be required.

The locations and descriptions of the site and proposed development are based on the preliminary architectural plans and discussions with you. If project details vary significantly from those described herein, Geocon Incorporated should be contacted to evaluate the necessity for review and revision of this report.

4. GEOLOGIC SETTING

The site is located in the coastal plain within the southern portion of the Peninsular Ranges Geomorphic Province of southern California. The Peninsular Ranges is a geologic and geomorphic province that extends from the Imperial Valley to the Pacific Ocean and from the Transverse Ranges to the north and into Baja California to the south. The coastal plain of San Diego County is underlain by a thick sequence of relatively undisturbed and non-conformable sedimentary rocks that thicken to the west and range in age from Upper Cretaceous through the Pleistocene with intermittent deposition. The sedimentary units are deposited on bedrock Cretaceous to Jurassic age igneous and metavolcanic rocks. Geomorphically, the coastal plain is characterized by a series of twenty-one, stair-stepped marine terraces (younger to the west) that have been dissected by west flowing rivers that drain the Peninsular Ranges located to the east. The coastal plain is a relatively stable block that is dissected by relatively few faults consisting of the potentially active La Nacion Fault Zone and the active Rose Canyon Fault Zone. The Peninsular Ranges Province is also dissected by the Elsinore Fault Zone that is associated with and sub-parallel to the San Andreas Fault Zone, which is the plate boundary between the Pacific and North American Plates.

The site is located on the western portion of the coastal plain. Marine sedimentary units make up the geologic sequence encountered on the site and consist of Pleistocene-age Old Paralic Deposits overlying the early Pleistocene to late Pliocene-age San Diego Formation. The Old Paralic Deposits are near shore shallow marine and non-marine sandstone units with layers containing silt and clay. This unit is a maximum of approximately 20 feet thick. The San Diego Formation is the lowest geologic unit encountered on the site and generally consists of marine, silty, fine sandstone. The regional geology in the area is predominately controlled by the active Rose Canyon Fault Zone (RCFZ) which transitions from a strike slip fault to the northwest of the site to several faults that have oblique movements of both strike slip and normal faulting to the west. The San Diego Bay was created as a down dropped block within this fault zone. The zone extends southward and branches into three segments through Coronado. Potentially active Florida Canyon and Texas Street Faults are located to the east of the site and were likely active in the Tertiary and potentially extending into the Pleistocene. The site generally sloped to the San Diego Bay prior to development with a former canyon drainage roughly 6 blocks to the east.

5. SOIL AND GEOLOGIC CONDITIONS

Based on the information obtained during the previous field investigation and grading report, the site is underlain by one surficial soil type and two geologic units. The surficial soil consists of previously placed fill. The geologic units consist of Old Paralic Deposits, Unit 6 (formerly called the Bay Point

Formation) and the San Diego Formation. The occurrence and distribution of the units are presented on the fault trench log and boring logs in Appendix A, and on the Geologic Map, Figure 2. The Geologic Cross-Sections, Figures 3 and 4, depict the subsurface relationship between the geologic units. We prepared the geologic cross-sections using interpolation between exploratory borings; therefore, actual geotechnical conditions between the borings may be different than those illustrated. The surficial soil type and geologic units are described below in order of increasing age.

5.1 Previously Placed Fill (Qpf)

Previously placed fill exists from the existing surface to a depth of approximately 12 to 15 feet. The fi11 was placed under the observation of Geocon Incorporated during backfilling operations for the previous basement and during the construction of the existing temporary parking lot. The previously placed fill is composed of medium dense to dense, moist, yellowish brown, silty sand. We expect the fill will be removed during excavation for the proposed subterranean parking levels.

5.2 Old Paralic Deposits (Qop)

The late to middle Pleistocene-age Old Paralic Deposits (formerly Bay Point Formation) are mapped underlying the site by Kennedy and Tan (2008). We encountered the Old Paralic Deposits within the fault trench and in the borings to a maximum depth of approximately 40a› feet below existing grade. The Old Paralic Deposits consist of very dense, moist, yellowish to reddish brown, weakly to moderately cemented, silty, fine- to medium-grained sandstone with layers of olive gray, sandy to clayey siltstone. We encountered conglomeratic sandstone beds approximately 10 feet thick near the base of the unit and lenses of gravel and cobble one to two feet thick within the upper portions of the unit. The Old Paralic Deposits are considered suitable for the support of compacted fill or structural loads.

5.3 San Diego Formation (Tsd)

Tertiary-age San Diego Formation underlies the Old Paralic Deposits to the total depths explored. The San Diego Formation generally consists of weakly to moderately cemented, micaceous, moist to wet, yellowish brown to olive gray and olive brown to grayish brown, fine- to medium-grained sandstone. The San Diego Formation is considered suitable for support of structural loads.

6. GROUNDWATER

We encountered groundwater in our exploratory excavations at elevations of about 2U and 5U feet below MSL) corresponding to current depths of about 46'/2 and 49'/2 feet. However, groundwater elevations in the downtown area normally range from about 0 to 4 feet above MSL. We do not expect groundwater to significantly affect future project development; however, if excavations are planned near or below the groundwater elevation, additional recommendations will be required. It is not uncommon for groundwater seepage conditions to develop where none previously existed due to the permeability characteristics of the geologic units encountered on site. During the rainy season, seepage conditions may develop within the sidewalls of the excavation that may require special consideration during grading operations. Groundwater elevations are dependent on seasonal precipitation, irrigation and land use, among other factors, and vary as a result. Proper surface drainage will be critical to future performance of the project.

7. GEOLOGIC HAZARDS

7.1 Faulting

The *City of San Diego Seismic Safety Study, Geologic Hazards and Faults, Sheet 17* defines the site with a *Hazard Category 13.' Downtown Special Fault Zone.* In addition, the California Geological Survey (CGS) has issued a revised State of California Earthquake Fault Zone Map for the Point Lorna Quadrangle dated May 1, 2003, which includes portions of the downtown San Diego area. The subject property is not mapped within a State of California Earthquake Fault Zone. The site is presented on Figure 5 in relation to the locations of the Earthquake Fault Zones and known active and potentially active faults in the Downtown area of San Diego. A review of geologic literature, previous fault evaluations, our on-site fault investigation, and our experience with geologic conditions in the general area indicate that known active, potentially active, or inactive faults are not located at the site. The site is, however, located approximately Us mile from known active faults.

The site is located near the southern onshore portion of the Rose Canyon Fault Zone in an area that is transitional between the predominately right-lateral slip faulting characteristic of the faults north of the downtown area and the predominately dip-slip faulting characteristic of faults making up the southern portion of the Rose Canyon Fault Zone (Treiman, 1993). South of the downtown area, the major faults that compose the southern end of the Rose Canyon Fault Zone are the Spanish Bight, Coronado, and Silver Strand Faults. The east side of this zone is represented by the La Nacifin Fault (Treiman, 1993). Together, these faults define a wide and complexly faulted basin occupied by San Diego Bay and a narrow section of the continental shelf west of the Silver Strand.

Trenching by Lindvall and others (1990) on the Rose Canyon Fault in Rose Canyon several miles north of the site, by Owen Consultants (referenced by ICG, 1990) for the police station on a site north of E Street, and by Kleinfelder Incorporated at a site near First Avenue and Market Street in the downtown area have shown that Holocene soil (soil 11,000 years old or less) has been displaced by faulting within the Rose Canyon Fault Zone.

We previously performed fault investigations for the adjacent blocks to the south and southwest of the site between 5h and 7* Avenues and south of F Street (Geocon, 1995 and 1996, respectively). We excavated three fault trenches in an east-west direction across the width of the block between 6h and

7th Avenues directly south of the site. We did not observe evidence of faulting and did not recommend building setbacks. The locations of previous investigations for neighboring properties are shown on the Earthquake Fault Zone Map, Figure 5.

7.2 On-Site Faulting Evaluation

We excavated a fault trench to a maximum depth of approximately 8 feet below the preexisting building pad (after demolition of the previously erected structure) to evaluate the existence of faulting. The trench was approximately 95 feet long and extended across the property in a generally east-west direction, approximately perpendicular to the dominantly north-south trend of the faulting within the downtown area. The location of the fault trench is shown on the Geologic Map, Figure 2 and the log of trench is included as Figure A- l (map pocket). Faulting in the southern portion of the Rose Canyon Fault Zone, which includes the downtown area, is predominately dip slip (Treiman, 1993). Relatively large offsets and discordance in the stratigraphy would be expected if faulting were present. For the purposes of our fault evaluation, the Old Paralic Deposits were divided into separate subunits. The stratigraphic position of the units and their lithologic descriptions are presented on the trench log. Stratigraphic correlation within the fault trench indicates that the relative positions of units within the Old Paralic Deposits are continuous or depositionally pinch out. An old channel scour deposit within the Old Paralic Deposits was observed and is considered a depositional feature. The beds exposed in the trench are not offset and gently dip westward.

We did not observe indications of faulting, such as discordant bedding, clay gouge, shearing, or slickensides in the fault trench, in the base of the previous excavation for the basement, or within samples obtained from our borings. In our opinion, active, potentially active, or inactive faults do not underlie the site and building setbacks will not be required.

7.3 Seismicity

The historic seismicity or instrumental seismic record in the San Diego area indicates that there have been minor earthquakes in the San Diego Bay area, including events in 1964 and 1985 between M3 and 4+ (Treiman, 1993). Surface rupture has not been recorded with any of the seismic activity. Anderson and others (1989) indicate that the greatest peak acceleration recorded in the downtown area (at San Diego Light and Power) was 34 cm/sec' (0.03g) produced by an offshore earthquake in 1964 (M5.6).

Anderson and others (1989) have also estimated recurrence times for major earthquakes that may affect the San Diego Region. By combining geologic data with their model for ground motion attenuation for each earthquake event, they have estimated the recurrence rate of various levels of peak ground acceleration in the San Diego area. The results of their work indicate that peak accelerations of 10 to 20 percent gravity (g) are expected approximately once every 100 years

(Anderson and others, 1989). Higher peak accelerations will also occur but with a lower probability of occurrence or higher return period.

Lindvall and others (1991) have postulated a maximum likely slip rate of about 2 mm per year and a best estimate of about 1.5 mm per year, based on three-dimensional trenching on the Rose Canyon Fault in Rose Canyon several miles north of the site. They found stratigraphic evidence of at least three events during the past 8,100 years. The most recent surface rupture displaces the modern "A" horizon (topsoil), suggesting that this event probably occurred within the past 500 years.

Historically, the Rose Canyon Fault has exhibited low seismicity with respect to earthquakes in excess of magnitude 5.0 or greater. Earthquakes on the Rose Canyon Fault having a maximum magnitude of 7.2 are considered representative of the potential for seismic ground shaking within the property. The "maximum magnitude earthquake" is defined as the maximum earthquake that appears capable of occurring under the presently known tectonic framework

According to the computer program *EZ-FRISK* (Version 7.65), six known active faults are located within a search radius of 50 miles from the property. We used the 2008 USGS fault database that provides several models and combinations of fault data to evaluate the fault information. Based on this database, the nearest known active fault are the Newport-Inglewood and Rose Canyon Faults, located approximately ⅛ miles west of the site and are the dominant source of potential ground motion. Earthquakes that might occur on these fault zones or other faults within the southern California and northern Baja California area are potential generators of significant ground motion at the site. The estimated deterministic maximum earthquake magnitude and peak ground acceleration for the Newport-Inglewood Fault are 7.5 and 0.66g, respectively. Table 7.3.1 lists the estimated maximum earthquake magnitude and peak ground acceleration for the most dominant faults in relationship to the site location. We calculated peak ground acceleration (PGA) using Boore-Atkinson (2008) NGA USGS2008, Campbell-Bozorgnia (2008) NGA USGS2008, and Chiou-Youngs (2007) NGA USGS2008 acceleration-attenuation relationships. The subject site can be classified as Site Class C (Very Dense Soil).

TABLE 7.3.1
DETERMINISTIC SPECTRA SITE PARAMETERS

Fault Name	Distance from Site (miles)	Maximum Earthquake Magnitude **(Mw)**	Peak Ground Acceleration		
			Boore-Atkinson **2008(g)**	Campbell-Bozorgnia **2008(g)**	Chiou-Youngs **2007(g)**
Newport-Inglewood	'3	7.5	0.56	0.49	0.66
Rose Canyon	3	6.9	0.55	0.48	0.62
Coronado Bank	12	7.4	0.21	0.17	0.21
Palos Verdes Connected	12	7.7	0.23	0.18	0.24
Elsinore	42	7.9	0.11	0.08	0.09
Earthquake Valley	46	6.8	0.06	0.05	0.03

We used the computer program EZ-FRISK to perform a probabilistic seismic hazard analysis. The computer program EZ-FRISK operates under the assumption that the occurrence rate of earthquakes on each mappable Quaternary fault is proportional to the faults slip rate. The program accounts for fault rupture length as a function of earthquake magnitude, and site acceleration estimates are made using the earthquake magnitude and distance from the site to the rupture zone. The program also accounts for uncertainty in each of following: (1) earthquake magnitude, (2) rupture length for a given magnitude, (3) location of the rupture zone, (4) maximum possible magnitude of a given earthquake, and (5) acceleration at the site from a given earthquake along each fault. By calculating the expected accelerations from considered earthquake sources, the program calculates the total average annual expected number of occurrences of site acceleration greater than a specified value. We utilized acceleration-attenuation relationships suggested by Boore-Atkinson (2008) NGA USGS2008, Campbell-Bozorgnia (2008) NGA USGS2008, and Chiou-Youngs (2007) NGA USGS2008 in the analysis. Table 7.3.2 presents the site-specific probabilistic seismic hazard parameters including acceleration-attenuation relationships and the probability of exceedence.

TABLE 7.3.2
PROBABILISTIC SEISMIC HAZARD PARAMETERS

Probability of Exceedence	Peak Ground Acceleration		
	Boore-Atkinson, **2008 (g)**	Campbell-Bozorgnia, **2008 (g)**	Chiou-Youngs, **2007** (g)
2% in a 50 Year Period	0.63	0.57	0.73
5% in a 50 Year Period	0.37	0.35	0.42
10% in a 50 Year Period	0.22	0.22	0.24

The California Geologic Survey (CGS) has a program that calculates the ground motion for a 10 percent of probability of exceedence in 50 years based on an average of several attenuation relationships. Table 7.3.3 presents the calculated results from the Probabilistic Seismic Hazards Mapping Ground Motion Page from the CGS website.

TABLE 7.3.3
PROBABILISTIC SITE PARAMETERS FOR SELECTED FAULTS
CALIFORNIA GEOLOGIC SURVEY

Calculated Acceleration (g) Firm Rock	Calculated Acceleration (g) Soft Rock	Calculated Acceleration (g) Alluvium
0.27	0.29	0.33

While listing peak accelerations is useful for comparison of potential effects of fault activity in a region, other considerations are important in seismic design, including the frequency and duration of motion and the soil conditions underlying the site. Seismic design of the structures should be evaluated in accordance with the California Building Code (CBC) guidelines currently adopted by the City of San Diego.

It is our opinion the site could be subjected to moderate to severe ground shaking in the event of an earthquake along any of the faults listed on Table 7.3.1 or other faults in the southern Ca1ifornia/ northern Baja California region. We do not consider the site to possess a greater risk than that of the surrounding developments.

7.4 Liquefaction

Liquefaction typically occurs when a site is located in a zone with seismic activity, onsite soil is cohesionless or silt/clay with low plasticity, groundwater is encountered within 50 feet of the surface, and soil relative densities are less than about 70 percent. If the four of the previous criteria are met, a seismic event could result in a rapid pore-water pressure increase from the earthquake-generated ground accelerations. Seismically induced settlement may occur whether the potential for liquefaction exists or not. The potential for liquefaction and seismically induced settlement occurring within the site soil is considered to be very low due to the dense nature and age of the Very Old Paralic Deposits and the San Diego Formation.

7.5 Seiches and Tsunamis

Seiches are free or standing-wave oscillations of an enclosed water body that continue, pendulum fashion, after the original driving forces have dissipated. Seiches usually propagate in the direction of longest axis of the basin. The potential of seiches to occur is considered to be very low due to the absence of a nearby inland body of water.

A tsunami is a series of long-period waves generated in the ocean by a sudden displacement of large volumes of water. Causes of tsunamis may include underwater earthquakes, volcanic eruptions, or offshore slope failures. Wave heights and run-up elevations from tsunamis along the San Diego Coast have historically fallen within the normal range of the tides. The subject site is located approximately 1 mile from the San Diego Bay at an elevation of approximately 45 feet above MSL. Our review of the map titled *Tsunami Inundation Mapfor Emergency Planning, State of California, County of San Diego, Point Lorna Quadrangle, June 1. 2009,* by CEMA, CGS, and USC, shows that the site is not located within the mapped tsunami inundation zone.

7.6 Landslides

Examination of aerial photographs in our files, review of published geologic maps for the site vicinity, and the relatively level topography, it is our opinion landslides are not present at the property or at a location that could impact the subject site.

8. CONCLUSIONS AND RECOMMENDATIONS

8.1 General

8.1.1 The site is located within the Downtown Special Fault Zone established by the City of San Diego. The site is not located within a currently established State of California Earthquake Fault Zone. We performed this investigation in compliance with the City of San Diego Building Department and the *City of San Diego Seismic Safety Study, Geologic Hazards and Faults,* 2008.

8.1.2 We did not observe evidence of faulting in the Old Paralic Deposits encountered during our subsurface fault investigation and previous observations. Accordingly, the potential for surface rupture due to faulting in the area of the proposed development is considered to be very low and it is our opinion building setbacks are not required.

8.1.3 With the exception of possible strong seismic shaking, significant geologic hazards were not observed or are known to exist on the site that would adversely affect the proposed project. Special seismic design considerations, other than those recommended herein, are not required.

8.1.4 From a geotechnical standpoint, it is our opinion that the site is suitable to be developed provided the recommendations presented herein are implemented in design and construction of the project. Development plans for the proposed structure are not available at this time. We prepared this report on the assumption that there will be two levels of below grade parking.

8.1.5 Our previous field investigation indicates that the site is underlain by approximately 12 to 15 feet of previously placed fill overlying Old Paralic Deposits and the San Diego Formation. While the previously placed fill is suitable in its present condition for support of settlement-sensitive structures, we expect the fill and portions of the Old Paralic Deposits will be removed within the planned building area during excavation for the proposed subterranean parking levels. The Old Paralic Deposits and the San Diego Formation are suitable for the support of the proposed structure.

8L6 We encountered groundwater in our exploratory excavations at elevations of about 2'/z and 5'/z feet below MSL) corresponding to current depths of about 46'/ and 49'/2 feet. However, groundwater elevations in the downtown area normally range from about 0 to 4 feet above MSL. We do not expect groundwater to significantly affect project development based on an excavation depth of 25 feet from street grade.

8.1.7 The proposed structure can be supported on conventional shallow foundations bearing in Old Paralic Deposits or San Diego Formation.

8.1.8 We have assumed excavations for any subterranean level would extend approximately 25 feet below street grade, max. We expect the proposed excavations for any subterranean levels would be accomplished by constructing vertical excavations using a temporary shoring wall with soldier piles and tie-backs.

8.1.9 Excavation of the fill, Old Paralic Deposits, and San Diego Formation should generally be possible with moderate to heavy effort using conventional, heavy-duty equipment. Very heavy effort should be expected in localized areas for excavations into conglomeratic units and cemented sandstone.

8.1.10 Excavations within the existing soil should generally be possible with moderate to heavy effort using conventional heavy-duty equipment. Localized cemented or very hard zones may be encountered that will require very heavy effort to excavate with oversize material generated. We encountered very difficult drilling in Boring B-I at a depth of about 22 feet within the San Diego Formation.

8.1.11 Surface settlement monuments will not be required on this project; however, monitoring of the temporary shoring and adjacent structures as discussed herein should be performed.

8.1.12 With the exception of wall drains, other subdrains are not required for this project.

8.1.13 Civil or architectural drawings have not been provided for our review. We should review the plans when they are available to evaluate if additional recommendations are required.

82 Excavation and Soil Characteristics

8.2.I The soil encountered in the previous field investigation is considered to be "expansive" (Expansion Index [EI] greater than 20) as defined by 2013 California Building Code (CBC) Section 1803.5.3. Table 8.2 presents soil classifications based on the expansion index. We expect a majority of the soil encountered possess a "very low" to "low" expansion potential (expansion index of 50 or less).

TABLE 8.2
EXPANSION CLASSIFICATION BASED ON EXPANSION INDEX

Expansion Index (EI)	Expansion Classification	**2013** CBC Expansion Classification
0—20	Very Low	Non-Expansive
21—50	Low	Expansive
51—90	Medium	
91—130	High	
Greater Than 130	Very High	

8.2.2 We performed laboratory tests on samples of the site materials to evaluate the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate content tests are presented in Appendix B and indicate that the on-site materials at the locations tested possess "Not Applicable" and "50" sulfate exposure to concrete structures as defined by 2013 CBC Section 1904 and ACI 318-11 Sections 4.2 and 4.3. The presence of water-soluble sulfates is not a visually discernible characteristic; therefore, other soil samples from the site could yield different concentrations. Additionally, over time landscaping activities (i.e., addition of fertilizers and other soil nutrients) may affect the concentration.

8.2.3 We tested samples for potential of hydrogen (pH), resistivity, and water-soluble chloride ion content laboratory tests to aid in evaluating the corrosion potential to subsurface metal structures. Appendix B presents the laboratory test results.

8.2.4 Geocon Incorporated does not practice in the field of corrosion engineering. Therefore, further evaluation by a corrosion engineer may be performed if improvements that could be susceptible to corrosion are planned.

8.2.5 We expect excavation of existing fill soil and Old Paralic Deposits will require medium to heavy effort using conventional heavy-duty equipment during excavation operations. We expect that some gravel, cobble and cemented zones within the Old Paralic Deposits may be encountered during shoring, grading and trenching operations requiring very heavy effort. We do not expect to encounter the San Diego Formation unless soldier beams extend more than 15 to 20 feet below the basement.

8.2.6 Portions of the walls were left in place during the previous demolition operations. The walls are located on the north, west, and south property lines. The walls are located directly adjacent to the existing basement walls for the buildings located to the north and south. The retaining wall on the west is located below the curb/gutter. In addition, 2-sack slurry was placed below the sidewalk zone to a depth of about 6 feet below the sidewalk because compaction could not be attained for the soil. We expect the slurry and the masonry retaining walls will be encountered during the excavation operations.

8.3 Seismic Design Criteria

8.3.1 We used the computer program *U.S. Seismic Design Maps,* provided by the USGS. Table 8.3.1 summarizes site-specific design criteria obtained from the 2013 California Building Code (CBC; Based on the 2012 International Building Code [IBC] and ASCE 7-10), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The short spectral response uses a period of 0.2 second. The building structure and improvements should be designed using a soil Site Class C. We evaluated the Site Class using blow count data based on the discussion in Section 1613.3.2 of the 2013 CBC and Table 20.3-1 of ASCE 7-10. The values presented in Table 8.3.1 are for the risk-targeted maximum considered earthquake (MCER)

TABLE 8.3.1
2013 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	**2013** CBC Reference
Soil Site Class	C	Section 1613.3.2
MCE_R Ground Motion Spectral Response Acceleration — Class B (short).	1.225g	Figure 1613.3.1(1)
MCER Ground Motion Spectral Response Acceleration – Class B (1 sec), S_0	0.472g	Figure 1613.3.1(2)
Site Coefficient, F_A	1.000	Table 1613.3.3(1)
Site Coefficient, F_V	1.328	Table 1613.3.3(2)
Site Class Modified MCER Spectral Response Acceleration (short), SMS	1.225g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCER Spectral Response Acceleration (1 sec), SMl	0.627g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short). DS	0.8l6g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), Sp,	0.418g	Section 1613.3.4 (Eqn 16-40)

8.3.2 Table 8.3.2 presents additional seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-10 for the mapped maximum considered geometric mean (MCE_C)

TABLE 8.3.2
2013 CBC SITE ACCELERATION DESIGN PARAMETERS

Parameter	Value	ASCE **7-10** Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.550g	Figure 22-7
Site Coefficient, F_{PGA}	1.000	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.550g	Section 11.8.3 (Eqn 11.8-1)

8.3.3 Conformance to the criteria in Tables 8.3.1 and 8.3.2 for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a large earthquake occurs. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive.

8.4 Grading

8.4.1 Grading should be performed in accordance with the recommendations provided in this report, the *Recommended Grading Specifications* contained in Appendix C and the City of San Diego Grading Ordinance. Where the recommendations of this report conflict with Appendix C, the recommendations of this section take precedence.

8.4.2 Earthwork should be observed and compacted fill tested by representatives of Geocon Incorporated.

8.4.3 Prior to commencing grading, a preconstruction conference should be held at the site with the city inspector, owner or developer, grading contractor, civil engineer, and geotechnical engineer in attendance. Special soil handling and/or the grading plans can be discussed at that time.

8.4.4 Site preparation should begin with the removal of deleterious material, debris and vegetation. Material generated during stripping and/or site demolition should be exported from the site and should not be mixed with the fill soil. Existing underground improvements within the proposed building areas should be removed and the resulting depressions properly backfilled in accordance with the procedures described herein.

8.4.5 We expect that existing compacted fill and a portion of the Old Paralic Deposits will be removed during excavation for the subterranean parking structure. We do not expect remedial grading will be required subsequent to achieving finish pad elevation.

8.4.6 The site materials are considered suitable for use as fill (if necessary) provided it is generally free from vegetation, debris and other deleterious matter. Layers of fill should not be thicker than will allow for adequate bonding and compaction. Fill, including wall and trench backfill and scarified ground surfaces, should be compacted to a dry density of at least 90 percent of laboratory maximum dry density near to slightly above optimum moisture content as determined by ASTM Test Procedure D 1557.

8.4.7 Import fill soil (if necessary) should consist of granular materials with a "very low" to "low" expansion potential (EI of 50 or less) free of deleterious material and stones larger than 3 inches and should be compacted as recommended herein. Geocon Incorporated should be notified of the import soil source and should perform laboratory testing of import soil prior to its arrival at the site to determine its suitability as fill material.

8.5 Excavation Slopes, Shoring, and Tiebacks

8.5.1 The recommendations included herein are provided for stable excavations. It is the responsibility of the contractor to provide a safe excavation during the construction of the proposed project.

8.5.2 Temporary excavations should be made in conformance with OSHA requirements. The undocumented fill can be considered a Type B soil (Type C soil if seepage or groundwater is encountered) and the Very Old Paralic Deposits and San Diego Formation can be considered a Type A soil (Type B soil if seepage or groundwater is encountered) in accordance with OSHA requirements. In general, special shoring requirements will not be necessary if temporary excavations will be less than 4 feet in height. Temporary excavations greater than 4 feet in height, however, should be sloped back at an appropriate inclination. These excavations should not be allowed to become saturated or to dry out. Surcharge loads should not be permitted to a distance equal to the height of the excavation from the top of the excavation. The top of the excavation should be a minimum of 15 feet from the edge of existing improvements. Excavations steeper than those recommended or closer than 15 feet from an existing surface improvement should be shored in accordance with applicable OSHA codes and regulations.

8.5.3 The design of temporary shoring is governed by soil and groundwater conditions, and by the depth and width of the excavated area. Continuous support of the excavation face can be provided by a system of soldier piles and wood lagging. Excavations exceeding 15 feet may require soil nails, tieback anchors, or internal bracing to provide additional wall restraint.

8.5.4 Portions of the walls were left in place during the previous demolition operations. The walls are located on the north, west, and south property lines. The walls are located directly adjacent to the existing basement walls for the buildings located to the north and south. The retaining wall on the west is located below the curb/gutter. In addition, 2-sack slurry was placed below the sidewalk zone to a depth of about 6 feet below the sidewalk because compaction could not be attained for the soil. We expect the slurry and the masonry retaining walls will be encountered during the construction of the temporary shoring and during the excavation operations.

8.5.5 Temporary shoring with a level backfill should be designed using a lateral pressure envelope acting on the back of the shoring and applying a pressure equal to 26H, 17H, or 21H, for a triangular, rectangular, or trapezoidal distribution, respectively, where H is the height of the shoring in feet (resulting pressure in pounds per square foot) as shown in Figure 6. These values are based on a retaining wall height of 25 feet and we should be contacted if higher retaining wall are planned. Triangular distribution should be used for cantilevered shoring and, the trapezoidal and rectangular distribution should be used for multi-braced systems such as tieback anchors and rakers. The project shoring engineer should determine the applicable soil distribution for the design of the temporary shoring system. Additional lateral earth pressure due to the surcharging effects of adjacent structures or traffic loads should be considered, where appropriate, during design of the shoring system.

8.5.6 Passive soil pressure resistance for embedded portions of soldier piles can be based upon an equivalent passive soil fluid weight of 400 D +500 where D is the depth of embedment, in feet (resulting in pounds per square foot), as shown on Figure 7. The passive resistance can be assumed to act over a width of three pile diameters. Typically, soldier piles are embedded a minimum of 0.5 times the maximum height of the excavation (this depth is to include footing excavations) if tieback anchors are not employed. The project structural engineer should determine the actual embedment depth.

8.5.7 Drilled shafts for the soldier piles should be observed by Geocon Incorporated prior to the placement of steel reinforcement to check that the exposed soil conditions are similar to those expected and that footing excavations have been extended to the appropriate bearing strata, and design depths. If unexpected soil conditions are encountered, foundation modifications may be required

8.5.8 Lateral movement of shoring is associated with vertical ground settlement outside of the excavation. Therefore, it is essential that the soldier pile and tieback system allow very limited amounts of lateral displacement. Earth pressures acting on a lagging wall can cause movement of the shoring toward the excavation and result in ground subsidence outside of the excavation. Consequently, horizontal movements of the shoring wall should be accurately monitored and recorded during excavation and anchor construction.

8.5.9 Survey points should be established at the top of the pile on at least 20 percent of the soldier piles. An additional point located at an intermediate point between the top of the pile and the base of the excavation should be monitored on at least 20 percent of the piles if tieback anchors will be used. These points should be monitored on a weekly basis during excavation work and on a monthly basis thereafter until the permanent support system is constructed.

8.5.10 The shoring system should be designed to limit horizontal and vertical soldier pile movement to a maximum of 1 inch and ½ inch, respectively, as shown in Figure 8. The amount of horizontal deflection can be assumed to be essentially zero along the Active Zone and Effective Zone boundary. The magnitude of movement for intermediate depths and distances from the shoring wall can be linearly interpolated. The approximate limits of the active zone area are shown on the Geologic Map, Figure 2.

8.5.11 The project civil and/or shoring engineer should determine the allowable amount of horizontal movement associated with the shoring system that could affect existing utilities and structures, if present. In addition, the project civil and/or shoring engineer should evaluate the existing utilities and improvements and provide a conclusion regarding the ability of the utilities and improvements to withstand the expected lateral and vertical movement associated with the planned excavation.

8.5.12 If a raker system is employed, the rakers should not be inclined steeper than 1:1 (horizontal:vertical) to provide an excavation to the raker foundation system with an inclination less than 1:1. A shallow or deep foundation system can be used for the raker system. We should be contacted if a raker system is planned.

8.5.13 Tieback anchors employed in shoring should be designed such that anchors fully penetrate the Active Zone behind the shoring. The Active Zone can be considered the wedge of soil from the face of the shoring to a plane extending upward from the base of the excavation at a 29-degree angle from vertical, as shown on Figure 8. Normally, tieback anchors are contractor-designed and installed, and there are numerous anchor construction methods available. Non-shrinkage grout should be used for the construction of the tieback anchors.

8.5.14 Experience has shown that the use of pressure grouting during formation of the bonded portion of the anchor will increase the soil-grout bond stress. A pressure grouting tube should be installed during the construction of the tieback. Post grouting should be performed if adequate capacity cannot be obtained by other construction methods.

8.5.15 Anchor capacity is a function of construction method, depth of anchor, batter, diameter of the bonded section, and the length of the bonded section. Anchor capacity should be evaluated using the strength parameters shown in Table 8.5.

TABLE 8.5
SOIL STRENGTH PARAMETERS FOR TEMPORARY SHORING

Description	Cohesion (psf)	**Friction** Angle (Degrees)
Old Paralic Deposits	300 psf	33 degrees
San Diego Formation	300 psf	33 degrees

8.5.16 Grout should only be placed in the tieback anchor's bonded section prior to testing. Tieback anchors should be proof-tested to at least 130 percent of the anchor's design working load. Following a successful proof test, the tieback anchors should be locked off at 80 percent of the allowable working load. Tieback anchor test failure criteria should be established in project plans and specifications. The tieback anchor test failure criteria should be based upon a maximum allowable displacement at 130 percent of the anchor's working load (anchor creep) and a maximum residual displacement within the anchor following stressing. Tieback anchor stressing should only be conducted after sufficient hydration has occurred within the grout. Tieback anchors that fail to meet project specified test criteria should be replaced or additional anchors should be constructed.

8.5.17 Lagging should keep pace with excavation and tieback anchor construction. The excavation should not be advanced deeper than three feet below the bottom of lagging at any time. These unlagged gaps of up to three feet should only be allowed to stand for short periods of time in order to decrease the probability of soil instability and should never be unsupported overnight. Backfilling should be conducted when necessary between the back of lagging and excavation sidewalls to reduce sloughing in this zone and all voids should be filled by the end of each day. Further, the excavation should not be advanced further than four feet below a row of tiebacks prior to those tiebacks being proof tested and locked off.

8.5.18 If tieback anchors are employed, an accurate survey of existing utilities and other underground structures adjacent to the shoring wall should be conducted. The survey should include both locations and depths of existing utilities. Locations of anchors should be adjusted as necessary during the design and construction process to accommodate the existing and proposed utilities.

8.5.19 The condition of existing buildings, streets, sidewalks, and other structures/improvements around the perimeter of the planned excavation should be documented prior to the start of shoring and excavation work. Special attention should be given to documenting existing cracks or other indications of differential settlement within these adjacent structures, pavements and other improvements. Underground utilities sensitive to settlement should be videotaped prior to construction to check the integrity of pipes. In addition, monitoring points should be established indicating location and elevation around the excavation and upon existing buildings. These points should be monitored on a weekly basis during excavation work and on a monthly basis thereafter. Inclinometers should be installed and monitored behind any shoring sections that will be advanced deeper than 30 feet below the existing ground surface.

Site Drainage and Moisture Protection

Adequate site drainage is critical to reduce the potential for differential soil movement, erosion and subsurface seepage. Under no circumstances should water be allowed to pond.

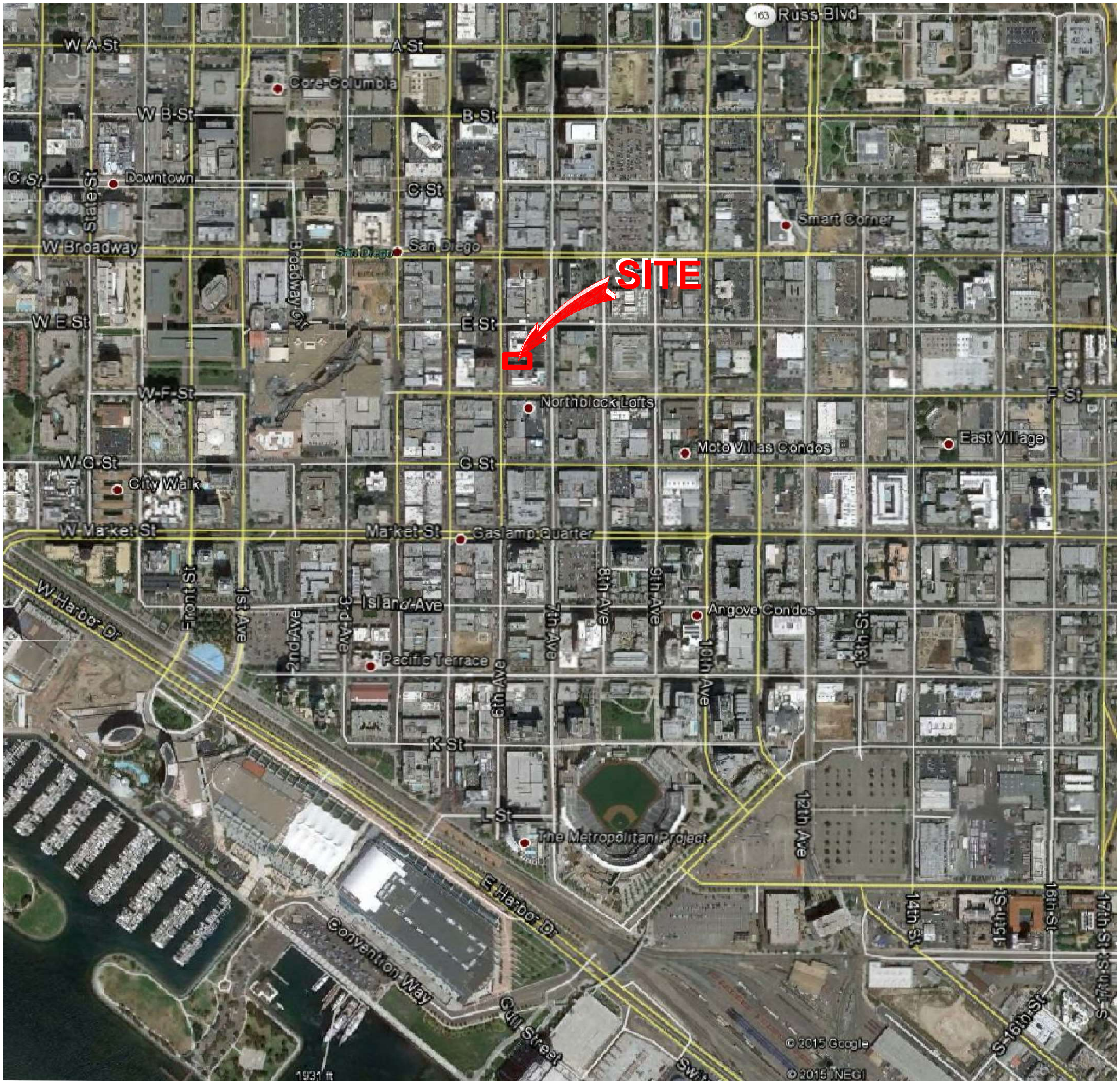
LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. Recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geocon Incorporated should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous or corrosive materials was not part of the scope of services provided by Geocon Incorporated.

2. This report is issued with the understanding that it is the responsibility of the owner, or of his representative, to ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project and incorporated into the plans, and the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

3. The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.

4. The firm that performed the geotechnical investigation for the project should be retained to provide testing and observation services during construction to provide continuity of geotechnical interpretation and to check that the recommendations presented for geotechnical aspects of site development are incorporated during site grading, construction of improvements, and excavation of foundations. If another geotechnical firm is selected to perform the testing and observation services during construction operations, that firm should prepare a letter indicating their intent to assume the responsibilities of project geotechnical engineer of record. A copy of the letter should be provided to the regulatory agency for their records. In addition, that firm should provide revised recommendations concerning the geotechnical aspects of the proposed development, or a written acknowledgement of their concurrence with the recommendations presented in our report. They should also perform additional analyses deemed necessary to assume the role of Geotechnical Engineer of Record.



THE GEOGRAPHICAL INFORMATION MADE AVAILABLE FOR DISPLAY WAS PROVIDED BY GOOGLE EARTH, SUBJECT TO A LICENSING AGREEMENT. THE INFORMATION IS FOR ILLUSTRATIVE PURPOSES ONLY; IT IS NOT INTENDED FOR CLIENT'S USE OR RELIANCE AND SHALL NOT BE REPRODUCED BY CLIENT. CLIENT SHALL INDEMNIFY, DEFEND AND HOLD HARMLESS GEOCON FROM ANY LIABILITY INCURRED AS A RESULT OF SUCH USE OR RELIANCE BY CLIENT.

N

NO SCALE

VICINITY MAP

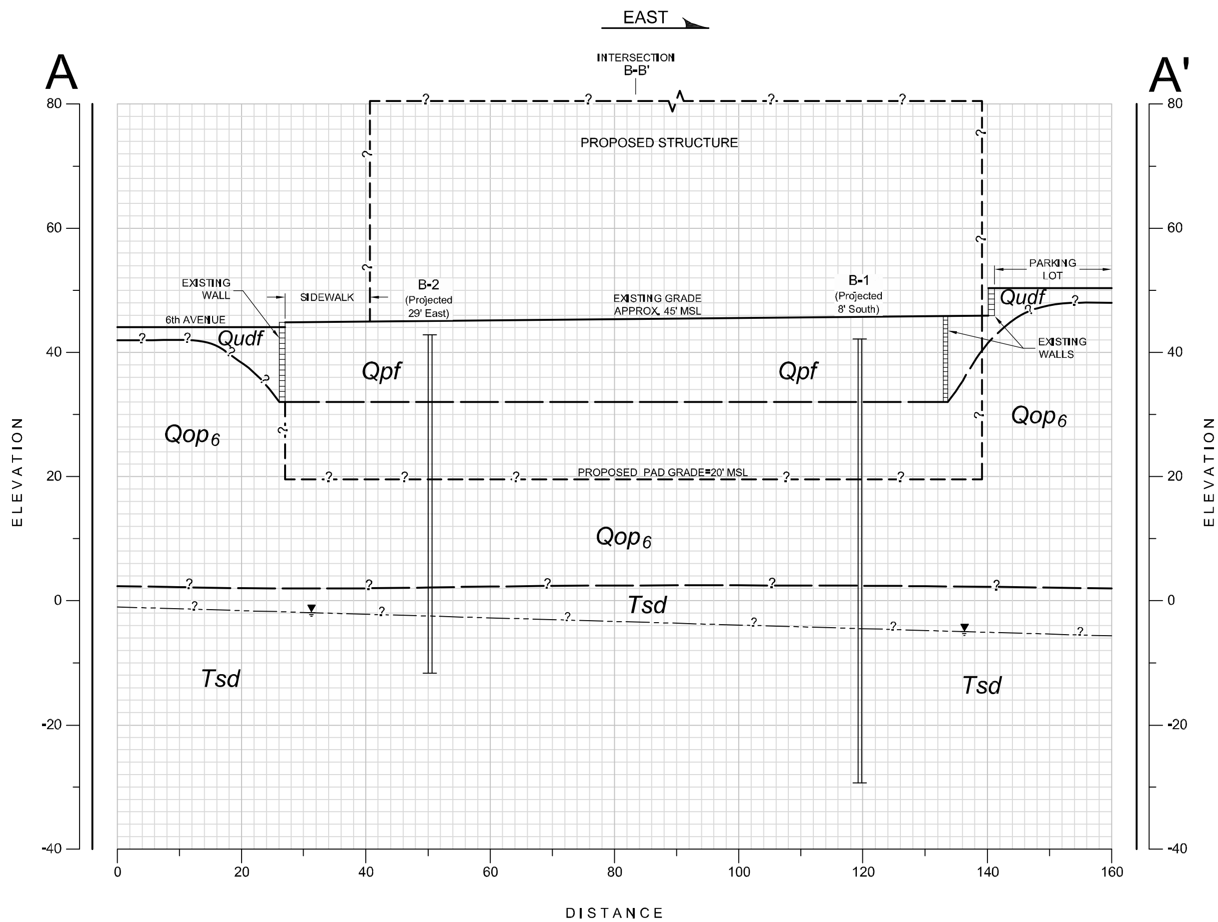
GEOCON
INCORPORATED
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974
PHONE 858 558-6900 - FAX 858 558-6159

MOXY HOTEL
835 SIXTH AVENUE
SAN DIEGO, CALIFORNIA

LR / RS		DSK/GTYPD	DATE 06 - 26 - 2015	PROJECT NO. G1863 - 52 - 01	FIG. 1

Plotted:06/26/2015 11:14AM | By:ALVIN LADRILLONO | File Location:Y:\PROJECTS\G1863-52-01 Moxy Hotel\DETAILS\G1863-52-01_VicMap.dwg

MOXY HOTEL
835 SIXTH AVENUE
SAN DIEGO, CALIFORNIA



GEOLOGIC CROSS-SECTION A-A'

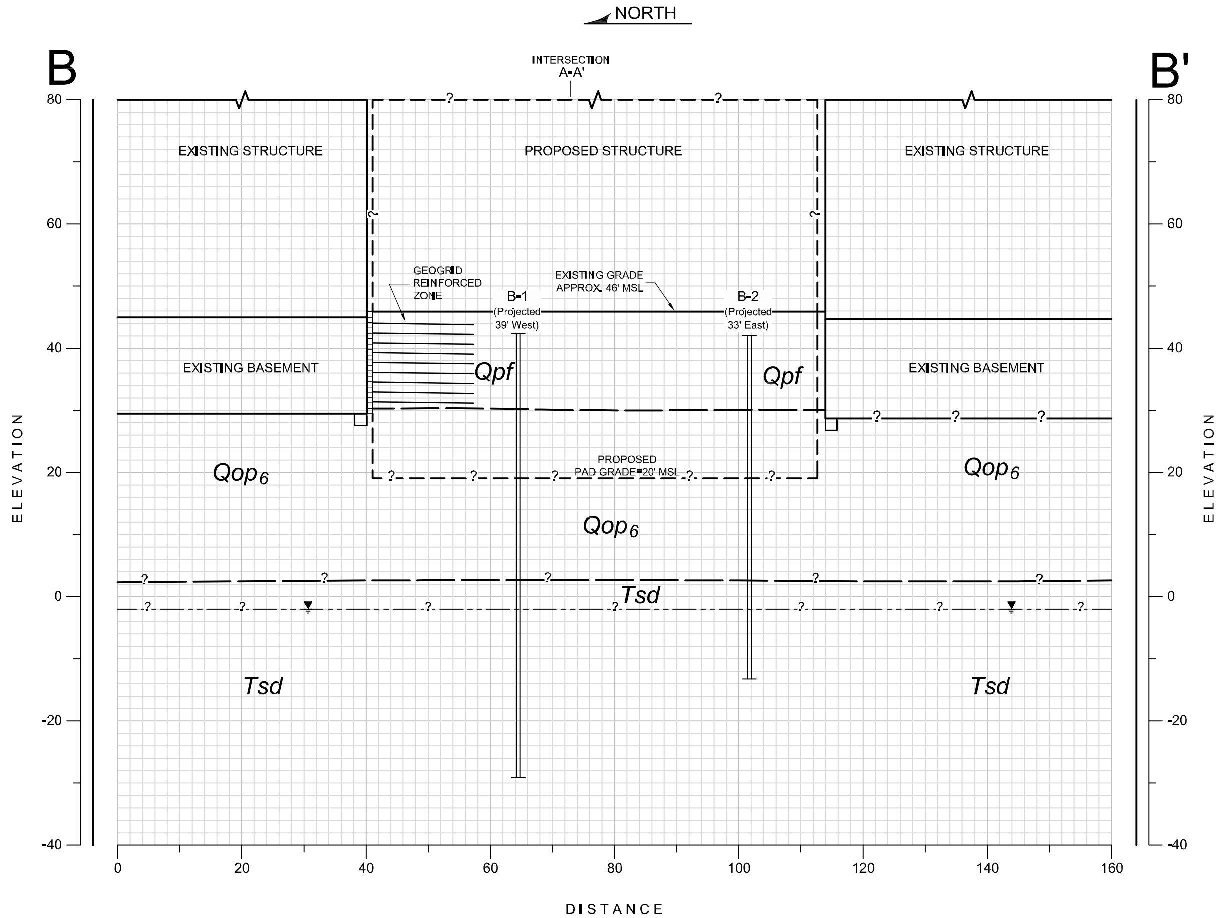
SCALE: 1" = 20' (Vert. = Horiz.)

LEGEND

- Qudf........UNDOCUMENTED FILL
- Qpf........PREVIOUSLY COMPACTED FILL
- Qop_6........OLD PARALIC DEPOSITS
- Tsd........SAN DIEGO FORMATION
- ....APPROX. LOCATION OF GEOLOGIC CONTACT (Queried Where Uncertain)
- B-1 ....APPROX. LOCATION OF EXPLORATORY BORING (GEOCON, 2008)
- ....APPROX. LOCATION OF GROUNDWATER TABLE

GEOCON INCORPORATED
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974
PHONE 858 558-6900 - FAX 858 558-6159
PROJECT NO. G1863 - 52 - 01
FIGURE 3
DATE 06 - 26 - 2015

MOXY HOTEL
835 SIXTH AVENUE
SAN DIEGO, CALIFORNIA



GEOLOGIC CROSS-SECTION B-B'

SCALE: 1" = 20' (Vert. = Horiz.)

LEGEND

Qpf........PREVIOUSLY COMPACTED FILL

Qop_6........OLD PARALIC DEPOSITS

Tsd........SAN DIEGO FORMATION

........APPROX. LOCATION OF GEOLOGIC CONTACT (Queried Where Uncertain)

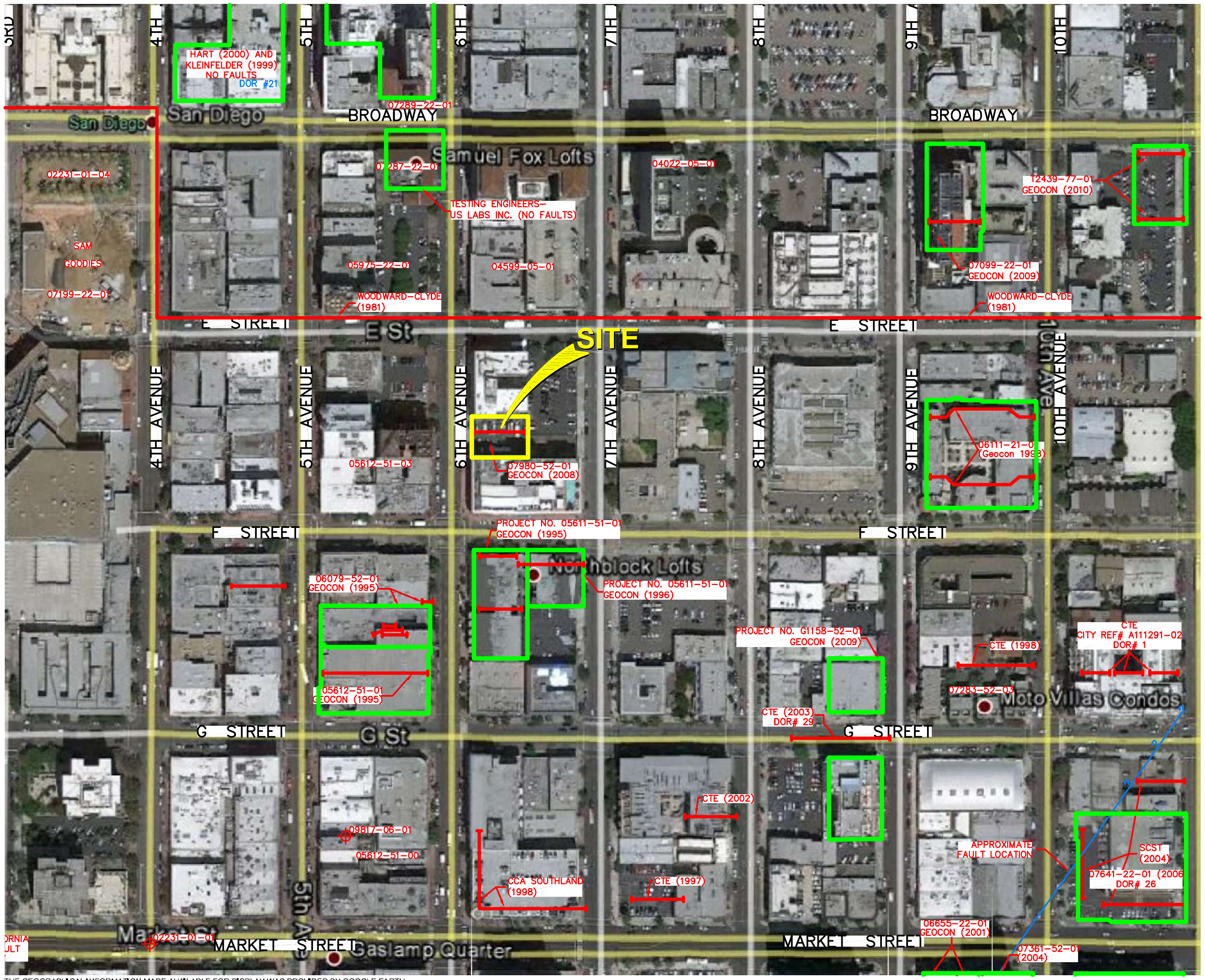
B-1 ....APPROX. LOCATION OF EXPLORATORY BORING (GEOCON, 2008)

........APPROX. LOCATION OF GROUNDWATER TABLE

GEOCON
INCORPORATED
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974
PHONE 858 558-6900 - FAX 858 558-6159

PROJECT NO. G1863 - 52 - 01
FIGURE 4
DATE 06 - 26 - 2015

MOXY HOTEL
835 SIXTH AVENUE
SAN DIEGO, CALIFORNIA



THE GEOGRAPHICAL INFORMATION MADE AVAILABLE FOR DISPLAY WAS PROVIDED BY GOOGLE EARTH, SUBJECT TO A LICENSING AGREEMENT. THE INFORMATION IS FOR ILLUSTRATIVE PURPOSES ONLY; IT IS NOT INTENDED FOR CLIENT'S USE OR RELIANCE AND SHALL NOT BE REPRODUCED BY CLIENT. CLIENT SHALL INDEMNIFY, DEFEND AND HOLD HARMLESS GEOCON FROM ANY LIABILITY INCURRED AS A RESULT OF SUCH USE OR RELIANCE BY CLIENT.

0' 200' 400'

SCALE: 1" = 200"

GEOCON LEGEND

........APPROX. LOCATION OF FAULT TRENCH, COMPANY THAT PERFORMED THE TRENCH (Year Reported)

........APPROX. LOCATION OF PREVIOUS PROJECT BOUNDARY

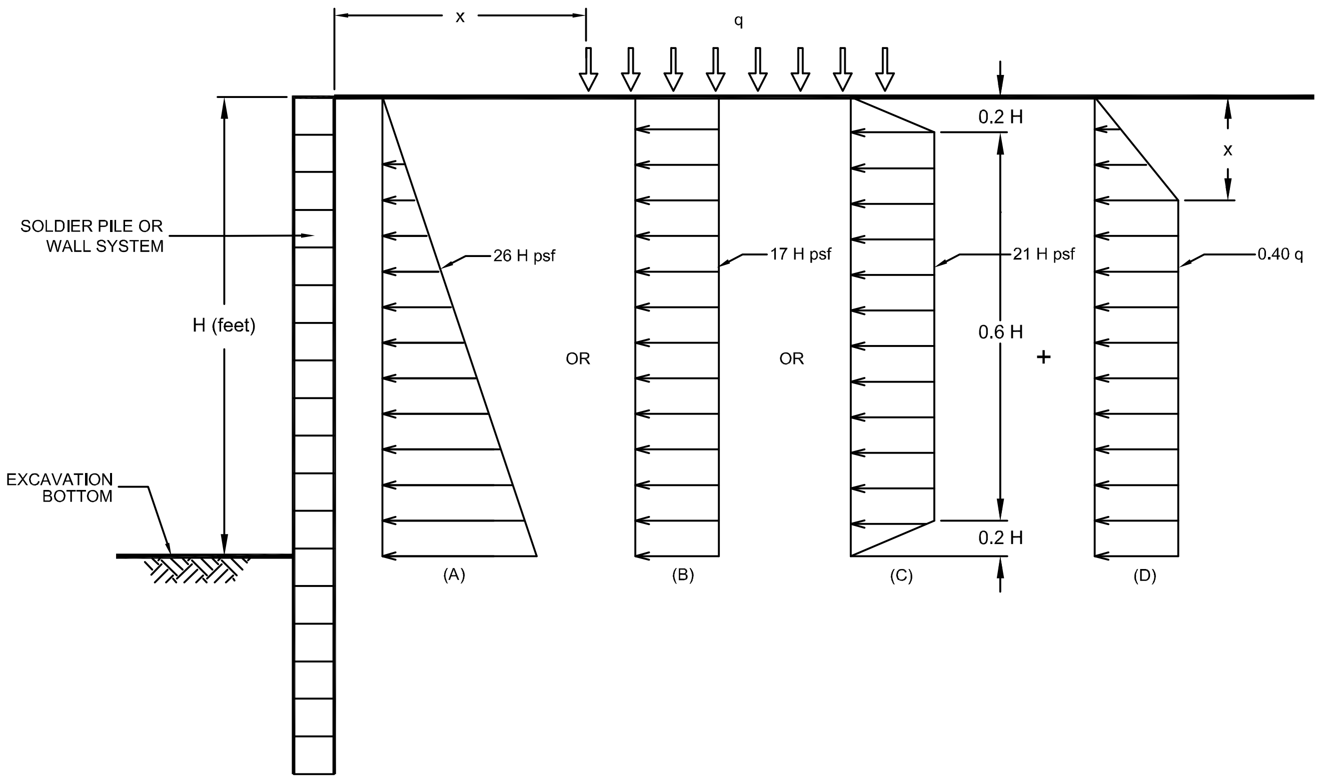
—?—?—?— ....APPROX. LOCATION OF ENCOUNTERED FAULT (Queried Where Uncertain)

FAULT LOCATION MAP

GEOCON INCORPORATED
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974
PHONE 858 558-6900 - FAX 858 558-6159

PROJECT NO. G1863 - 52 - 01
FIGURE 5
DATE 06 - 26 - 2015

Plotted:06/29/2015 1:50PM | By:RUDY SURYA | File Location:Y:\PROJECTS\G1863-52-01 Moxy Hotel\SHEETS\G1863-52-01_FaultLocMap.dwg



(A)......TRIANGULAR DISTRIBUTION - CANTILEVERED SHORING

(B)......RECTANGULAR DISTRIBUTION - MULTI-BRACED SHORING

(C)......TRAPEZOIDAL DISTRIBUTION - MULTI BRACED SHORING

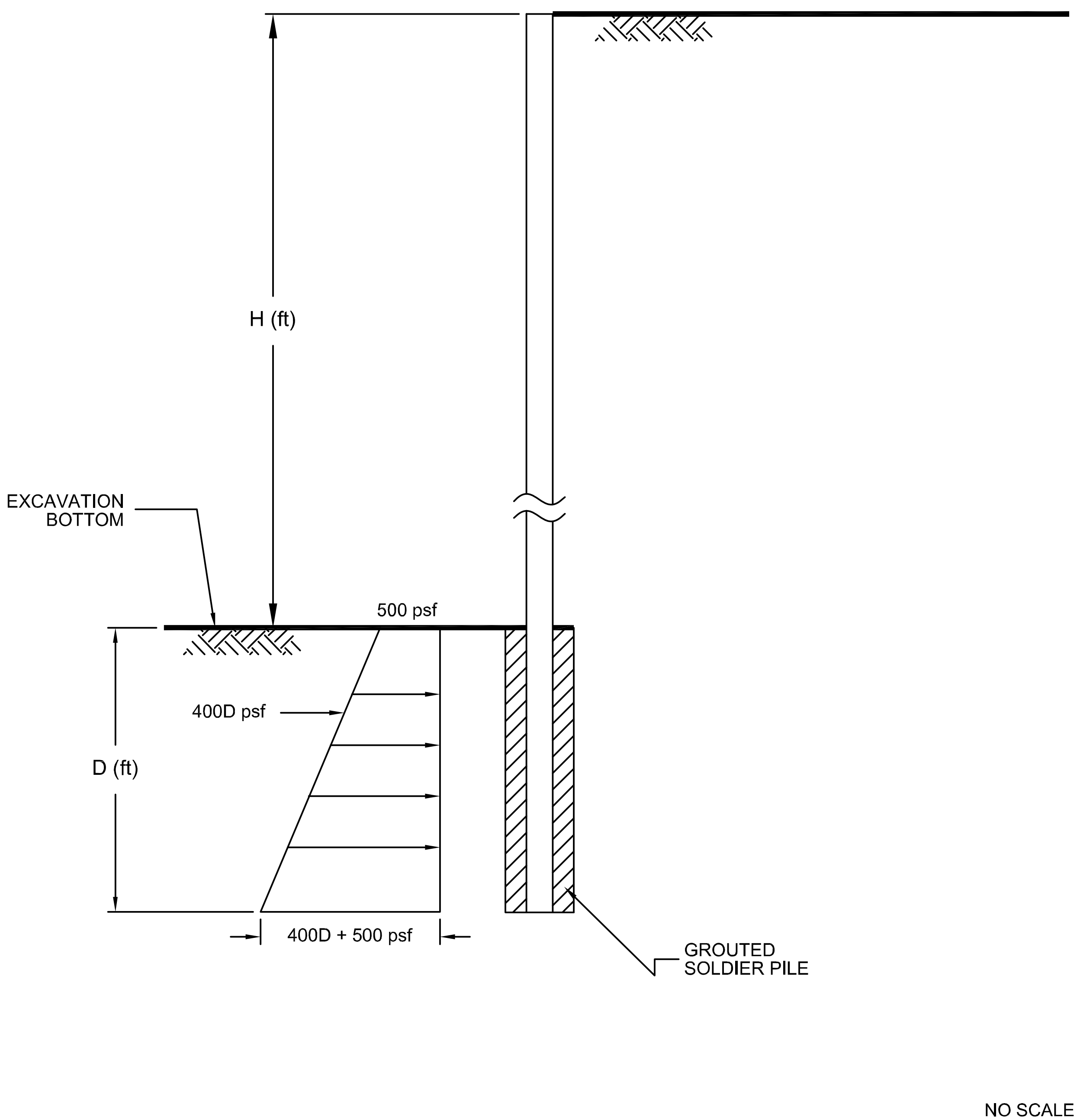
NO SCALE

LATERAL ACTIVE PRESSURES FOR VERTICAL EXCAVATIONS

GEOCON
INCORPORATED
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974
PHONE 858 558-6900 - FAX 858 558-6159

MOXY HOTEL
835 SIXTH AVENUE
SAN DIEGO, CALIFORNIA

LR / RS		DSK/GTYPD	DATE 06 - 26 - 2015	PROJECT NO. G1863 - 52 - 01	FIG.6

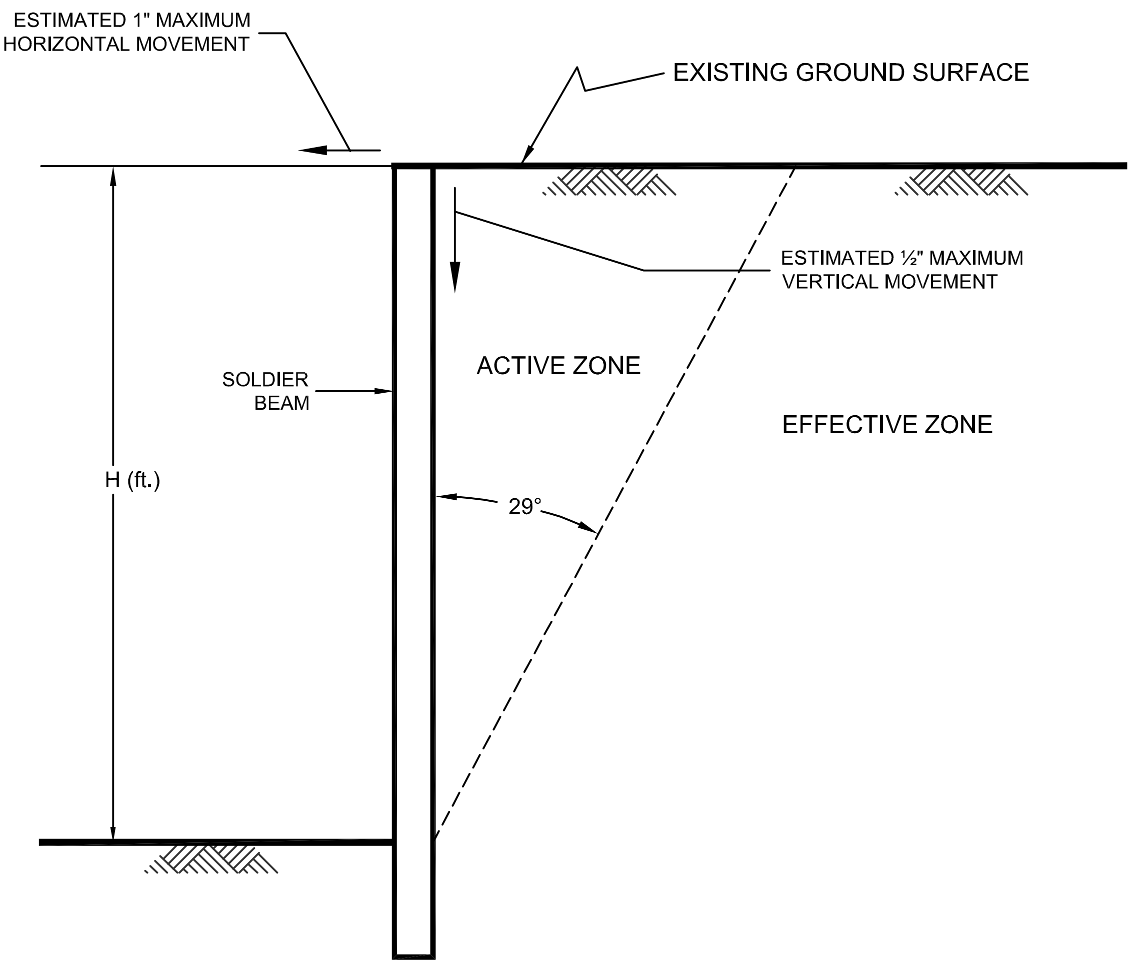


RECOMMENDED GROUTED SOLDIER PILE PRESSURE DISTRIBUTION

GEOCON
INCORPORATED
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974
PHONE 858 558-6900 - FAX 858 558-6159

MOXY HOTEL
835 SIXTH AVENUE
SAN DIEGO, CALIFORNIA

LR / RS		DSK/GTYPD	DATE 06 - 26 - 2015	PROJECT NO. G1863 - 52 - 01	FIG.7



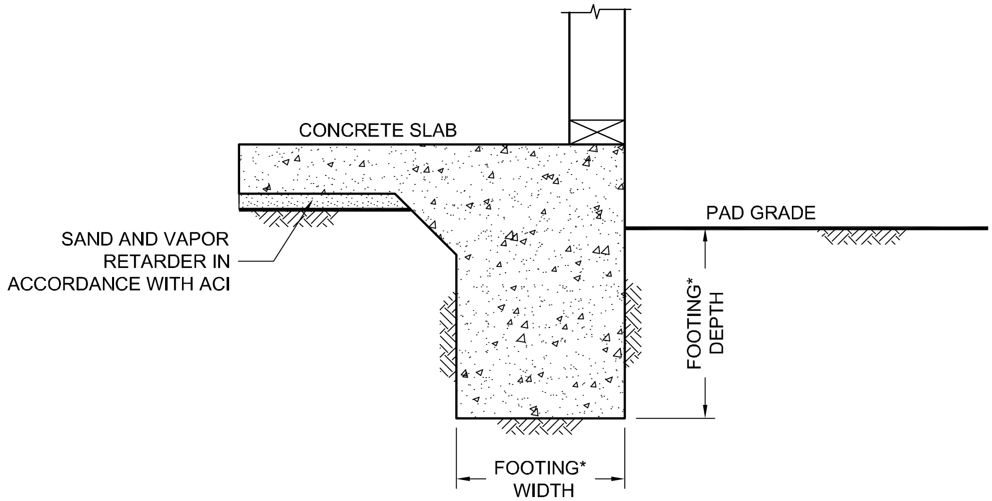
RECOMMENDED EFFECTIVE ZONE FOR TIEBACK ANCHORS

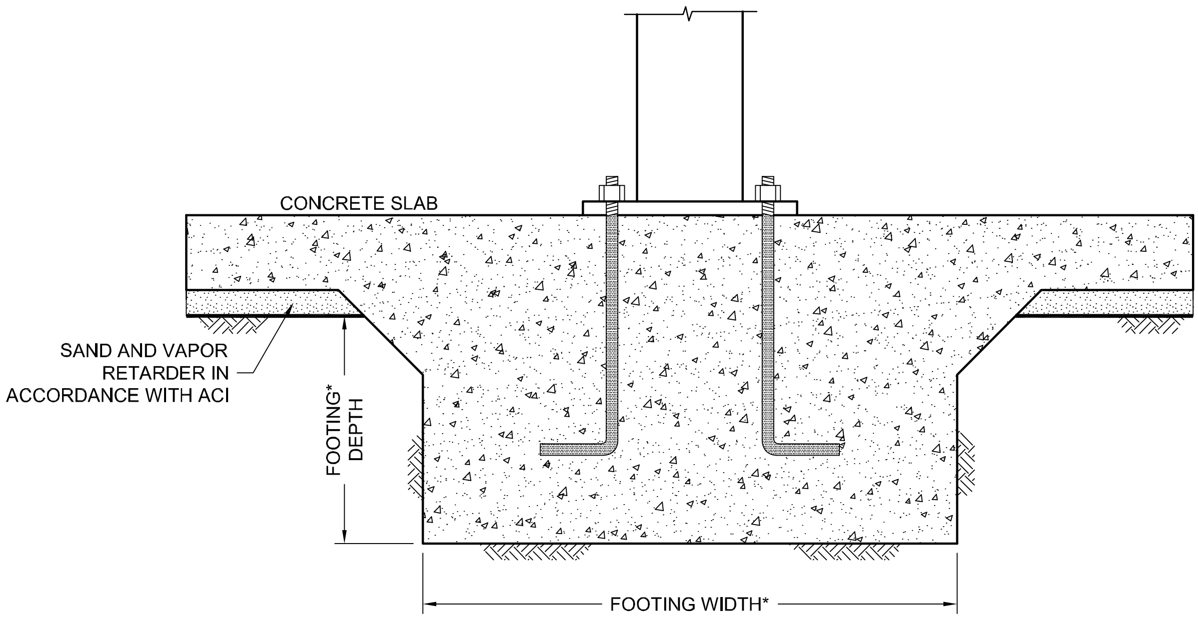
GEOCON
INCORPORATED
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974
PHONE 858 558-6900 - FAX 858 558-6159

MOXY HOTEL
835 SIXTH AVENUE
SAN DIEGO, CALIFORNIA

LR / RS		DSK/GTYPD	DATE 06 - 26 - 2015	PROJECT NO. G1863 - 52 - 01	FIG. 8

Plotted:06/29/2015 10:37AM | By:RUDY SURYA | File Location:Y:\PROJECTS\G1863-52-01 Moxy Hotel\DETAILS\REZTA3.dwg





*....SEE REPORT FOR FOUNDATION WIDTH AND DEPTH RECOMMENDATION

NO SCALE

WALL / COLUMN FOOTING DIMENSION DETAIL

GEOCON INCORPORATED

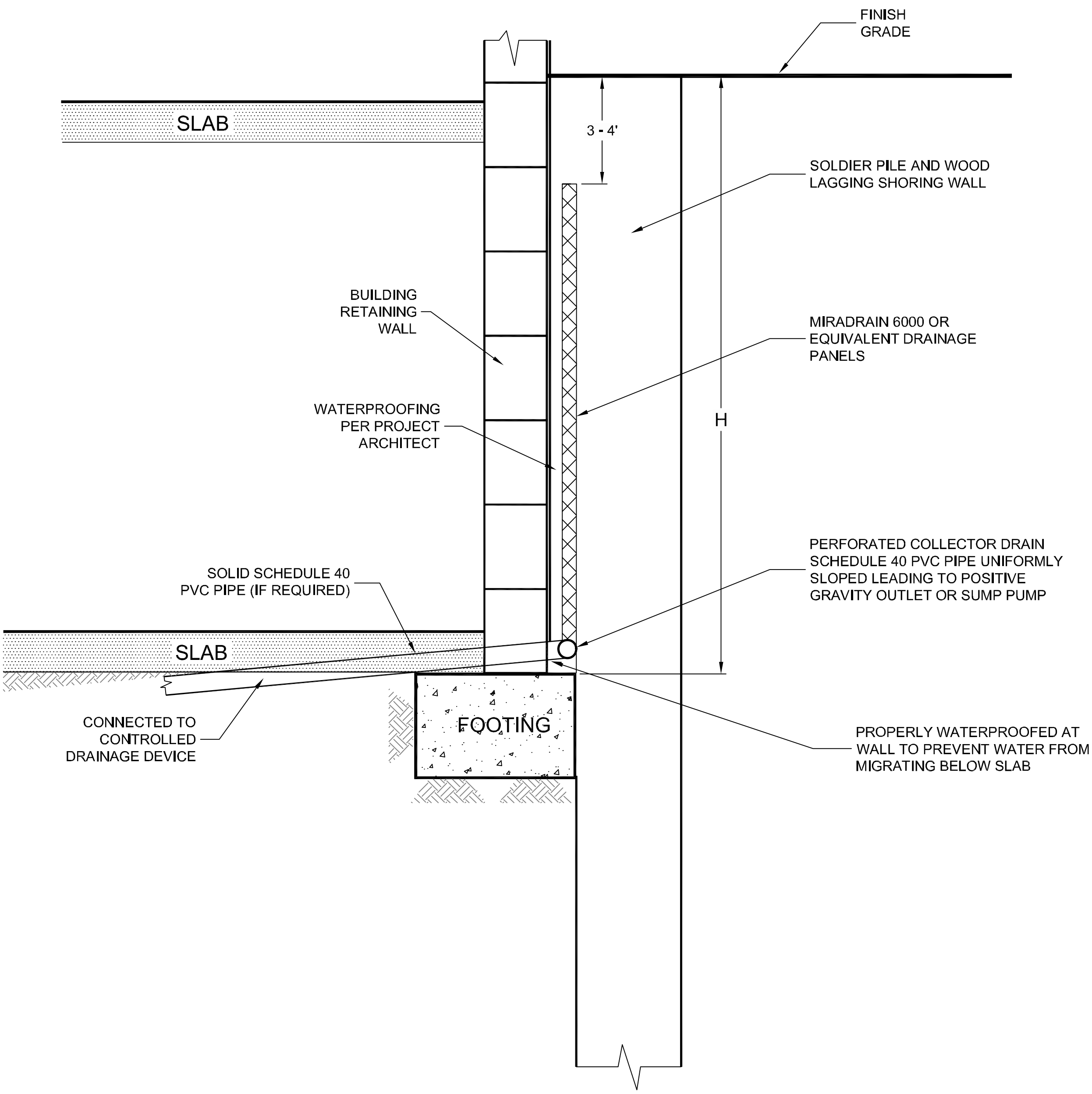
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974

PHONE 858 558-6900 - FAX 858 558-6159

MOXY HOTEL
835 SIXTH AVENUE
SAN DIEGO, CALIFORNIA

LR / RS		DSK/GTYPD	DATE 06 - 26 - 2015	PROJECT NO. G1863 - 52 - 01	FIG. 9



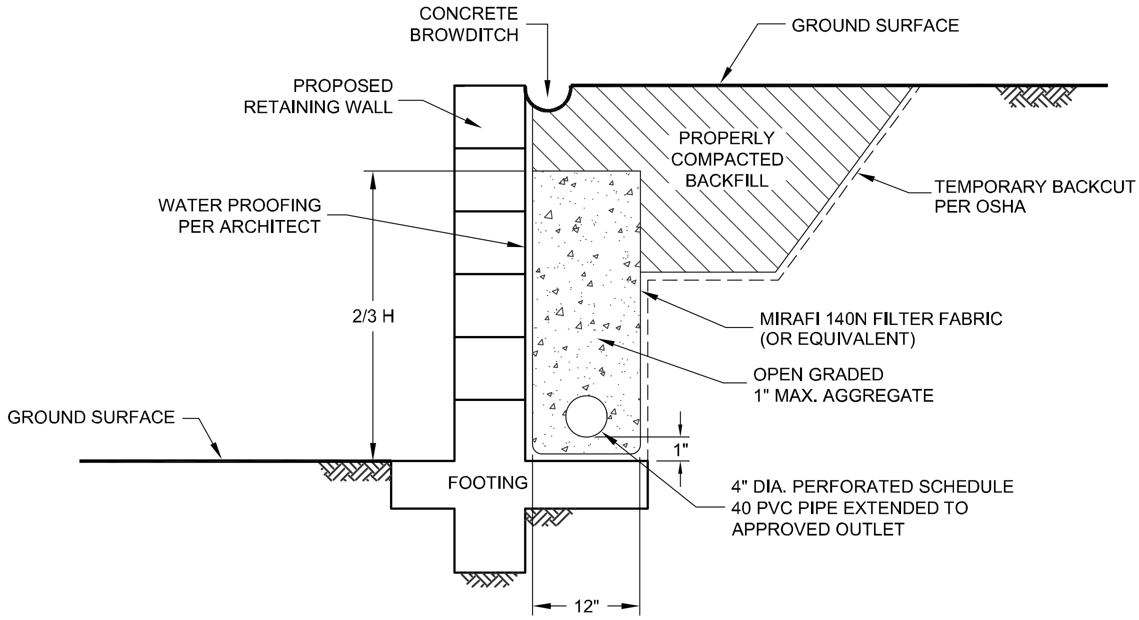
NO SCALE

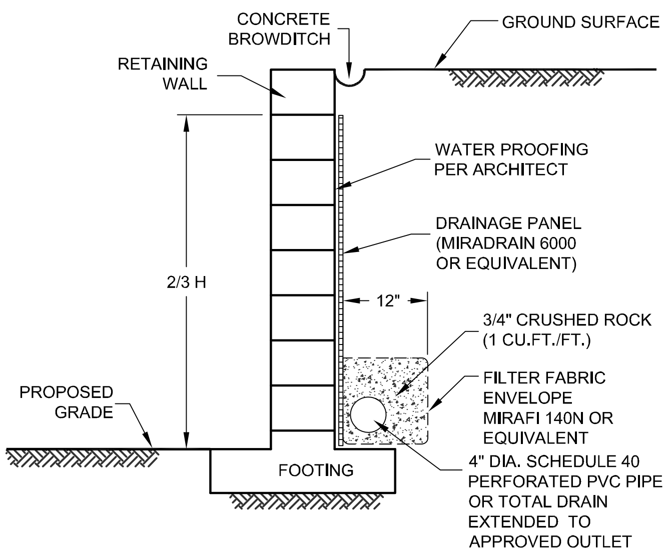
SOLDIER PILE WALL DRAINAGE DETAIL

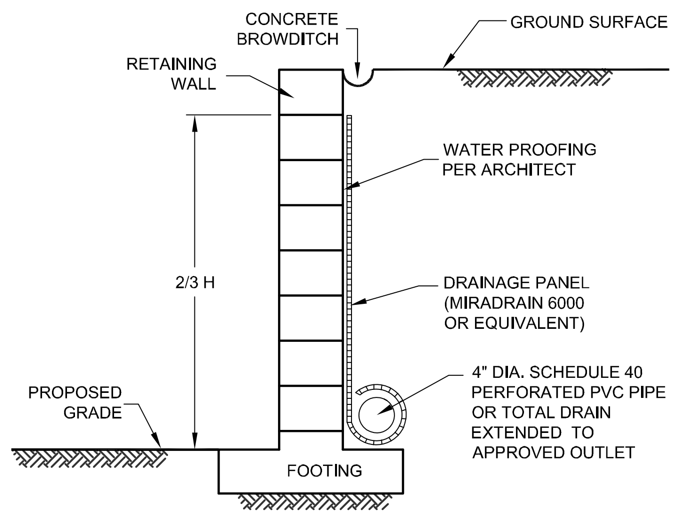
GEOCON
INCORPORATED
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974
PHONE 858 558-6900 - FAX 858 558-6159

MOXY HOTEL
835 SIXTH AVENUE
SAN DIEGO, CALIFORNIA

LR / RS		DSK/GTYPD	DATE 06 - 26 - 2015	PROJECT NO. G1863 - 52 - 01	FIG. 10







NOTE :

DRAIN SHOULD BE UNIFORMLY SLOPED TO GRAVITY OUTLET
OR TO A SUMP WHERE WATER CAN BE REMOVED BY PUMPING

TYPICAL RETAINING WALL DRAIN DETAIL

GEOCON
INCORPORATED
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974
PHONE 858 558-6900 - FAX 858 558-6159

MOXY HOTEL
835 SIXTH AVENUE
SAN DIEGO, CALIFORNIA

LR / RS		DSK/GTYPD	DATE 06 - 26 - 2015	PROJECT NO. G1863 - 52 - 01	FIG.11

APPENDIX A

APPENDIX A

PREVIOUS FIELD INVESTIGATION (GEOCON, 2008)

We performed the fieldwork for our investigations between May 2 and June 17, 2008. The exploratory excavations consisted of the observation and logging of one fault trench and two small-diameter borings. The locations of the exploratory trenches and borings are shown on the Geologic Map, Figure 2. Trench logs, boring logs, and descriptions of the geologic units encountered are presented on Figures A-1 through A-3.

We excavated a fault trench through the central portion of the site after the demolition contractor had removed the previously existing structure. The top of the fault trench was located at approximately 12 feet below the adjacent street grade and the trench extended to a maximum depth of approximately 8 feet. The trench was excavated using a track-mounted excavator equipped with a 48-inch wide bucket. The trench was oriented in a generally east-west direction at close to right angles to the regional and local trend of splays within the Rose Canyon Fault Zone. We logged a total of approximately 95 linear feet of fault trench during the investigation.

We logged the trench walls at a scale of 1 inch equals 5 feet (1" = 5'). Stationing along the trench surfaces was established during logging for accurate location of features and for ease of description. Also, a horizontal string line was established within the trenches for use as an internal reference. The entire surface of the geologic units exposed along the north and south sides of each trench was cleaned and examined for indications of faulting. These indications could include offset units, contacts, or laminations, tectonically disturbed or deformed clay layers, clay gouge, fissures, or slickensides.

We excavated two small diameter borings to a maximum depth of approximately 71½ feet using a CME 75 truck mounted drill rig equipped with 8-inch diameter hollow stem augers. We obtained samples using a Modified California Sampler and a Standard Penetration Test (SPT) sampler during the drilling operations. Both samplers are composed of steel and are driven to obtain undisturbed samples. The Modified California sampler has an inside diameter of 2.5 inches and an outside diameter of 3 inches. Up to 18 rings are placed inside the sampler that is 2.4 inches in diameter and 1 inch in height. The SPT sampler has an inside diameter of 1.5 inches and an outside diameter of 2 inches. Up to 4 rings (depending on the length of the sampler) may be placed inside the sampler that is 1.375 inches in diameter and 6 inches in height. We obtained ring samples at appropriate intervals, placed them in moisture-tight containers, and transported them to the laboratory for testing. The type of sample is noted on the exploratory boring logs.

The sampler was driven 12 inches and 18 inches for Modified California sampler and SPT sampler, respectively. Sampler was driven into the bottom of the excavations with the use of an automatic hammer and the use of A rods. The sampler is connected to the A rods and driven into the bottom of the excavation using a 140-pound hammer with a 30-inch drop. Blow counts are recorded for every 6 inches the sampler is driven. The penetration resistances shown on the boring logs are shown in terms of blows per foot. The values indicated on the boring logs are the sum of the last 12 inches of the sampler. If the sampler was not driven for 12 inches, an approximate value is calculated in term of blows per foot or the final 6-inch interval is reported. These values are not to be taken as N-values as adjustments have not been applied. We estimated elevations shown on the boring logs either from a topographic map or by using a benchmark. Each excavation was backfilled as noted on the boring logs.

The soil encountered in the borings and fault trench were visually examined, classified, and logged in general accordance with American Society for Testing and Materials (ASTM) practice for Description and Identification of Soils (Visual-Manual Procedure D 2488). The logs depict the soil and geologic conditions observed and the depth at which samples were obtained.

BORING B 1

ELEV. (MSL.) **42'** DATE COMPLETED **06-17-2008**

EQUIPMENT **CME 75** BY: **M. ERTWINE**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SC	**COMPACTED FILL (Qcf)** Dense, moist, olive brown, Clayey SAND			
2								
4								
6								
8								
10								
12				SC	**OLD PARALIC DEPOSITS (Qop6)** Dense, moist, yellowish to reddish brown, Clayey, fine- to medium-grained SANDSTONE; weakly cemented			
14								
	B1-1					60		
16								
	B1-2							
18					-Some shell fragments and cobbles			
20	B1-3			ML	Hard, moist, olive gray to light yellowish brown, Clayey SILTSTONE	37		13.5
22								
24								
	B1-4			SM	Very dense, moist, yellowish to reddish brown, Silty, fine-grained SANDSTONE	71		
26								
28								

Figure A-2,
Log of Boring B 1, Page 1 of 3

07980-52-01.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

BORING B 1

ELEV. (MSL.) 42' DATE COMPLETED 06-17-2008

EQUIPMENT CME 75 BY: M. ERTWINE

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30				GP+SM	Very dense, moist, olive brown, conglomeratic SANDSTONE; gravel- and cobble-sized clasts in sandy matrix; moderately cemented	50/1"		
32								
34					-Very poor recovery, excavates to olive brown, sandy clay with some gravel			
36	B1-5					50/5"		
38								
40					-Very poor recovery, some gravel			
	B1-6			SM	**SAN DIEGO FORMATION (Tsd)** Very dense, moist, light reddish to yellowish brown, Silty, fine-grained SANDSTONE; slightly micaceous; moderately cemented	85		6.8
42								
44								
46					-No recovery	50/4"		
			▼		-Static groundwater at 47½ feet			
48								
50	B1-7				-Wet, grayish to light yellowish brown, and fine- to medium-grained; slightly micaceous; uncemented	50/5"		
52								
54								
56	B1-8				-Saturated, grayish brown, and micaceous; black fissile laminations; weakly cemented	50/5"		22.7
58								

Figure A-2,
Log of Boring B 1, Page 2 of 3

07980-52-01.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING B 1 ELEV. (MSL.) 42' DATE COMPLETED 06-17-2008 EQUIPMENT CME 75 BY: M. ERTWINE MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
60	B1-9			SM	-Becomes dense	59		22.4
62								
64								
66	B1-10				-Becomes very dense	50/4"		24.7
68								
70	B1-11				-Massive	64		25.3
					BORING TERMINATED AT 71½ FEET Groundwater encountered at 47½ feet Backfilled with 25 ft³ of bentonite grout slurry with tremie method			

Figure A-2,
Log of Boring B 1, Page 3 of 3

07980-52-01.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

BORING B 2

ELEV. (MSL.) 42' DATE COMPLETED 06-17-2008

EQUIPMENT CME 75 BY: M. ERTWINE

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SC	**COMPACTED FILL (Qcf)** Dense, moist, olive brown, Clayey SAND			
12				SM-ML	**OLD PARALIC DEPOSITS (Qop6)** Very dense, damp, light olive brown and reddish brown, fine-grained Sandy SILTSTONE to Silty, fine-grained SANDSTONE; moderately cemented			
15	B2-1					84	109.7	11.8
18					-Cobble and gravel lens			
20					-No recovery; excavates to olive brown to brown, clayey sand with some gravel	50/4"		
21	B2-2							
25	B2-3			SM-SC	Very dense, moist, yellowish to reddish brown, Silty to Clayey, fine- to medium-grained SANDSTONE; moderately cemented	50/5"	113.4	15.1

Figure A-3,
Log of Boring B 2, Page 1 of 2

07980-52-01.GPJ

SAMPLE SYMBOLS
- ... SAMPLING UNSUCCESSFUL
- ... STANDARD PENETRATION TEST
- ... DRIVE SAMPLE (UNDISTURBED)
- ... DISTURBED OR BAG SAMPLE
- ... CHUNK SAMPLE
- ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

BORING B 2

ELEV. (MSL.) 42' DATE COMPLETED 06-17-2008

EQUIPMENT CME 75 BY: M. ERTWINE

DEPTH IN FEET	SAMPLE NO.	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B2-4	SM+GP	Very dense, moist, olive brown, conglomeratic SANDSTONE; gravel- and cobble-sized clasts in sandy matrix; moderately cemented	50/3"		
36	B2-5		-No recovery; excavates to olive brown, sandy clay	50/3"		11.2
40				50/1"		
41		SM	**SAN DIEGO FORMATION (Tsd)** Very dense, moist, light yellowish brown, Silty, fine-grained SANDSTONE; moderately cemented			
44½			-Static groundwater level at 44½ feet			
45	B2-6	SP	Very dense, saturated, gray to light yellowish brown, fine- to medium-grained SANDSTONE; slightly micaceous; weakly cemented	50/4"	113.9	17.2
49		SM	Very dense, saturated, yellowish brown, Silty, fine-grained SANDSTONE			
50	B2-7			50/4"		22.8

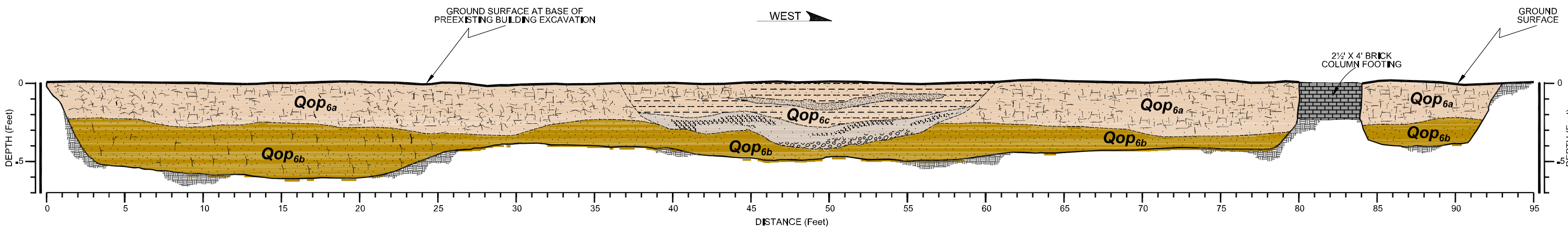
BORING TERMINATED AT 55 FEET
Groundwater encountered at 44½ feet
Backfilled with 19 ft^3 of bentonite grout slurry

Figure A-3,
Log of Boring B 2, Page 2 of 2

07980-52-01.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.



FAULT TRENCH FT-1 (South Wall)

SCALE: 1" = 5' (Horiz. = Vert.)

GEOCON LEGEND

Qop6 ....OLD PARALIC DEPOSITS

Qop6a ....Medium dense to dense, damp to moist, yellowish to reddish brown, Clayey, fine-grained SANDSTONE, weakly to moderately cemented; prominent pedigenic blocky texture with illuviated clay films along ped faces; common manganese oxide stringers and coating on ped faces; some pinhole voids; evidence of in-place weathering and weak soil development; local carbonate cemented burrow casts.

Qop6b ....Medium dense to dense, damp, brown to light olive brown, Silty, fine- to medium-grained SANDSTONE; weakly cemented; massive with no discernable bedding; locally mottled with olive gray.

Qop6c ....Loose to medium dense or stiff to very stiff, moist, olive gray, light yellowish brown, and reddish brown, interbedded fine to medium SAND and Clayey SILT; represents channel deposit scoured into older units; discontinuous basal gravel and lenses of loose sand; local pedigenic development and secondary cementation.

........Approximate location of interformational contact.

FAULT TRENCH FT-1

GASLAMP APARTMENTS
831 THROUGH 845 SIXTH AVENUE
SAN DIEGO, CALIFORNIA

GEOCON INCORPORATED
GEOTECHNICAL CONSULTANTS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974
PHONE 858 558-6900 - FAX 858 558-6159

SCALE 1" = 5'	DATE 07 - 10 - 2008	
PROJECT NO. 07980 - 52 - 01		FIGURE A-1
SHEET 1 OF 1		

Y:/07980-52-01/RA/NA_7980 FAULT TRENCH.DWG

APPENDIX B

APPENDIX B

PREVIOUS LABORATORY TESTING (GEOCON, 2008)

We performed laboratory tests in accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. We tested selected samples for their in-place dry density and moisture content, maximum dry density and optimum moisture content, shear strength, expansion index, water-soluble sulfate characteristics, pH and resistivity, and chloride content. The results of our laboratory tests are presented in Tables B-I through B-VI. In addition, the in-place dry density and moisture content results are presented on the exploratory boring logs.

TABLE B-I
SUMMARY OF LABORATORY MAXIMUM DRY DENSITY AND OPTIMUM MOISTURE CONTENT TEST RESULTS
ASTM D 1557

Sample No.	Description	Maximum Dry Density (pcf)	Optimum Moisture Content (% dry wt.)
B1-2	Reddish brown, Silty, fine to medium SAND	132.1	8.6

TABLE B-II
SUMMARY OF LABORATORY DIRECT SHEAR TEST RESULTS
ASTM D 3080

Sample No.	Dry Density (pcf)	Moisture Content (%)		Unit Cohesion (psf) Peak [Ultimate]	Angle of Shear Resistance (degrees) Peak [Ultimate]
		Initial	Final		
B1-1	105.2	5.0	18.2	450 [250]	36 [34]
B1-4	111.7	14.4	18.5	650	26
B1-7	112.3	17.4	17.1	450 [240]	43 [36]

TABLE B-III
SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS
ASTM D 4829

Sample No.	Moisture Content (%)		Dry Density (pcf)	Expansion Index	Expansion Classification	2013 CBC Expansion Classification
	Before Test	After Test				
B1-2	8.7	17.4	114.0	33	Low	Expansive

TABLE B-IV
SUMMARY OF LABORATORY WATER-SOLUBLE SULFATE TEST RESULTS
CALIFORNIA TEST NO. 417

Sample No.	Water-Soluble Sulfate (%)	Sulfate Severity
B1-2	0.023	Not Applicable (S0)

TABLE B-V
SUMMARY OF LABORATORY POTENTIAL OF HYDROGEN (PH) AND RESISTIVITY TEST RESULTS
CALIFORNIA TEST NO. 643

Sample No.	pH	Minimum Resistivity (ohm-centimeters)
B1-2	8.4	676

TABLE B-VI
SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS
AASHTO TEST NO. T291

Sample No.	Chloride Content (ppm)	Chloride Content (%)
B1-2	250	0.025

APPENDIX C

APPENDIX C

RECOMMENDED GRADING SPECIFICATIONS

FOR

MOXY HOTEL
835 SIXTH AVENUE
SAN DIEGO, CALIFORNIA

PROJECT NO. G1863-52-01

RECOMMENDED GRADING SPECIFICATIONS

1. GENERAL

1.1 These Recommended Grading Specifications shall be used in conjunction with the Geotechnical Report for the project prepared by Geocon Incorporated. The recommendations contained in the text of the Geotechnical Report are a part of the earthwork and grading specifications and shall supersede the provisions contained hereinafter in the case of conflict.

1.2 Prior to the commencement of grading, a geotechnical consultant (Consultant) shall be employed for the purpose of observing earthwork procedures and testing the fills for substantial conformance with the recommendations of the Geotechnical Report and these specifications. The Consultant should provide adequate testing and observation services so that they may assess whether, in their opinion, the work was performed in substantial conformance with these specifications. It shall be the responsibility of the Contractor to assist the Consultant and keep them apprised of work schedules and changes so that personnel may be scheduled accordingly.

1.3 It shall be the sole responsibility of the Contractor to provide adequate equipment and methods to accomplish the work in accordance with applicable grading codes or agency ordinances, these specifications and the approved grading plans. If, in the opinion of the Consultant, unsatisfactory conditions such as questionable soil materials, poor moisture condition, inadequate compaction, adverse weather, result in a quality of work not in conformance with these specifications, the Consultant will be empowered to reject the work and recommend to the Owner that grading be stopped until the unacceptable conditions are corrected.

2. DEFINITIONS

2.1 **Owner** shall refer to the owner of the property or the entity on whose behalf the grading work is being performed and who has contracted with the Contractor to have grading performed.

2.2 **Contractor** shall refer to the Contractor performing the site grading work.

2.3 **Civil Engineer** or **Engineer of Work** shall refer to the California licensed Civil Engineer or consulting firm responsible for preparation of the grading plans, surveying and verifying as-graded topography.

2.4 **Consultant** shall refer to the soil engineering and engineering geology consulting firm retained to provide geotechnical services for the project.

2.5 **Soil Engineer** shall refer to a California licensed Civil Engineer retained by the Owner, who is experienced in the practice of geotechnical engineering. The Soil Engineer shall be responsible for having qualified representatives on-site to observe and test the Contractor's work for conformance with these specifications.

2.6 **Engineering Geologist** shall refer to a California licensed Engineering Geologist retained by the Owner to provide geologic observations and recommendations during the site grading.

2.7 **Geotechnical Report** shall refer to a soil report (including all addenda) which may include a geologic reconnaissance or geologic investigation that was prepared specifically for the development of the project for which these Recommended Grading Specifications are intended to apply.

3. MATERIALS

3.1 Materials for compacted fill shall consist of any soil excavated from the cut areas or imported to the site that, in the opinion of the Consultant, is suitable for use in construction of fills. In general, fill materials can be classified as *soil* fills, *soil-rock* fills or *rock* fills, as defined below.

3.1.1 **Soil fills** are defined as fills containing no rocks or hard lumps greater than 12 inches in maximum dimension and containing at least 40 percent by weight of material smaller than ¾ inch in size.

3.1.2 **Soil-rock fills** are defined as fills containing no rocks or hard lumps larger than 4 feet in maximum dimension and containing a sufficient matrix of soil fill to allow for proper compaction of soil fill around the rock fragments or hard lumps as specified in Paragraph 6.2. **Oversize rock** is defined as material greater than 12 inches.

3.1.3 **Rock fills** are defined as fills containing no rocks or hard lumps larger than 3 feet in maximum dimension and containing little or no fines. Fines are defined as material smaller than ¾ inch in maximum dimension. The quantity of fines shall be less than approximately 20 percent of the rock fill quantity.

3.2 Material of a perishable, spongy, or otherwise unsuitable nature as determined by the Consultant shall not be used in fills.

3.3 Materials used for fill, either imported or on-site, shall not contain hazardous materials as defined by the California Code of Regulations, Title 22, Division 4, Chapter 30, Articles 9 and 10; 40CFR; and any other applicable local, state or federal laws. The Consultant shall not be responsible for the identification or analysis of the potential presence of hazardous materials. However, if observations, odors or soil discoloration cause Consultant to suspect the presence of hazardous materials, the Consultant may request from the Owner the termination of grading operations within the affected area. Prior to resuming grading operations, the Owner shall provide a written report to the Consultant indicating that the suspected materials are not hazardous as defined by applicable laws and regulations.

3.4 The outer 15 feet of *soil-rock* fill slopes, measured horizontally, should be composed of properly compacted *soil* fill materials approved by the Consultant. *Rock* fill may extend to the slope face, provided that the slope is not steeper than 2:1 (horizontal:vertical) and a soil layer no thicker than 12 inches is track-walked onto the face for landscaping purposes. This procedure may be utilized provided it is acceptable to the governing agency, Owner and Consultant.

3.5 Samples of soil materials to be used for fill should be tested in the laboratory by the Consultant to determine the maximum density, optimum moisture content, and, where appropriate, shear strength, expansion, and gradation characteristics of the soil.

3.6 During grading, soil or groundwater conditions other than those identified in the Geotechnical Report may be encountered by the Contractor. The Consultant shall be notified immediately to evaluate the significance of the unanticipated condition

4. CLEARING AND PREPARING AREAS TO BE FILLED

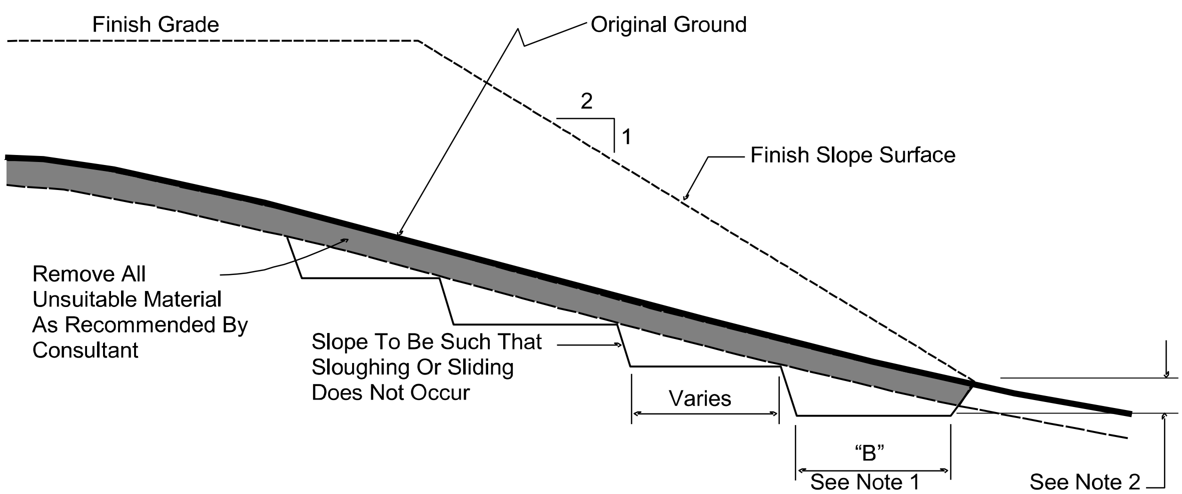
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material and shall be performed in areas to be graded. Roots and other projections exceeding 1½ inches in diameter shall be removed to a depth of 3 feet below the surface of the ground. Borrow areas shall be grubbed to the extent necessary to provide suitable fill materials.

4.2 Any asphalt pavement material removed during clearing operations should be properly disposed at an approved off-site facility. Concrete fragments that are free of reinforcing steel may be placed in fills, provided they are placed in accordance with Section 6.2 or 6.3 of this document.

4.3 After clearing and grubbing of organic matter and other unsuitable material, loose or porous soils shall be removed to the depth recommended in the Geotechnical Report. The depth of removal and compaction should be observed and approved by a representative of the Consultant. The exposed surface shall then be plowed or scarified to a minimum depth of 6 inches and until the surface is free from uneven features that would tend to prevent uniform compaction by the equipment to be used.

4.4 Where the slope ratio of the original ground is steeper than 5:1 (horizontal:vertical), or where recommended by the Consultant, the original ground should be benched in accordance with the following illustration.

TYPICAL BENCHING DETAIL



No Scale

DETAIL NOTES: (1) Key width "B" should be a minimum of 10 feet, or sufficiently wide to permit complete coverage with the compaction equipment used. The base of the key should be graded horizontal, or inclined slightly into the natural slope.

(2) The outside of the key should be below the topsoil or unsuitable surficial material and at least 2 feet into dense formational material. Where hard rock is exposed in the bottom of the key, the depth and configuration of the key may be modified as approved by the Consultant.

4.5 After areas to receive fill have been cleared and scarified, the surface should be moisture conditioned to achieve the proper moisture content, and compacted as recommended in Section 6 of these specifications.

5. COMPACTION EQUIPMENT

5.1 Compaction of *soil* or *soil-rock* fill shall be accomplished by sheepsfoot or segmented-steel wheeled rollers, vibratory rollers, multiple-wheel pneumatic-tired rollers, or other types of acceptable compaction equipment. Equipment shall be of such a design that it will be capable of compacting the *soil* or *soil-rock* fill to the specified relative compaction at the specified moisture content.

5.2 Compaction of *rock* fills shall be performed in accordance with Section 6.3.

6. PLACING, SPREADING AND COMPACTION OF FILL MATERIAL

6.1 *Soil* fill, as defined in Paragraph 3.1.1, shall be placed by the Contractor in accordance with the following recommendations:

6.1.1 *Soil* fill shall be placed by the Contractor in layers that, when compacted, should generally not exceed 8 inches. Each layer shall be spread evenly and shall be thoroughly mixed during spreading to obtain uniformity of material and moisture in each layer. The entire fill shall be constructed as a unit in nearly level lifts. Rock materials greater than 12 inches in maximum dimension shall be placed in accordance with Section 6.2 or 6.3 of these specifications.

6.1.2 In general, the *soil* fill shall be compacted at a moisture content at or above the optimum moisture content as determined by ASTM D 1557-09.

6.1.3 When the moisture content of *soil* fill is below that specified by the Consultant, water shall be added by the Contractor until the moisture content is in the range specified.

6.1.4 When the moisture content of the *soil* fill is above the range specified by the Consultant or too wet to achieve proper compaction, the *soil* fill shall be aerated by the Contractor by blading/mixing, or other satisfactory methods until the moisture content is within the range specified.

6.1.5 After each layer has been placed, mixed, and spread evenly, it shall be thoroughly compacted by the Contractor to a relative compaction of at least 90 percent. Relative compaction is defined as the ratio (expressed in percent) of the in-place dry density of the compacted fill to the maximum laboratory dry density as determined in accordance with ASTM D 1557-09. Compaction shall be continuous over the entire area, and compaction equipment shall make sufficient passes so that the specified minimum relative compaction has been achieved throughout the entire fill.

6.1.6 Where practical, soils having an Expansion Index greater than 50 should be placed at least 3 feet below finish pad grade and should be compacted at a moisture content generally 2 to 4 percent greater than the optimum moisture content for the material.

6.1.7 Properly compacted *soil* fill shall extend to the design surface of fill slopes. To achieve proper compaction, it is recommended that fill slopes be over-built by at least 3 feet and then cut to the design grade. This procedure is considered preferable to track-walking of slopes, as described in the following paragraph.

6.1.8 As an alternative to over-building of slopes, slope faces may be back-rolled with a heavy-duty loaded sheepsfoot or vibratory roller at maximum 4-foot fill height intervals. Upon completion, slopes should then be track-walked with a D-8 dozer or similar equipment, such that a dozer track covers all slope surfaces at least twice.

6.2 *Soil-rock* fill, as defined in Paragraph 3.1.2, shall be placed by the Contractor in accordance with the following recommendations:

6.2.1 Rocks larger than 12 inches but less than 4 feet in maximum dimension may be incorporated into the compacted *soil* fill, but shall be limited to the area measured 15 feet minimum horizontally from the slope face and 5 feet below finish grade or 3 feet below the deepest utility, whichever is deeper.

6.2.2 Rocks or rock fragments up to 4 feet in maximum dimension may either be individually placed or placed in windrows. Under certain conditions, rocks or rock fragments up to 10 feet in maximum dimension may be placed using similar methods. The acceptability of placing rock materials greater than 4 feet in maximum dimension shall be evaluated during grading as specific cases arise and shall be approved by the Consultant prior to placement.

6.2.3 For individual placement, sufficient space shall be provided between rocks to allow for passage of compaction equipment.

6.2.4 For windrow placement, the rocks should be placed in trenches excavated in properly compacted *soil* fill. Trenches should be approximately 5 feet wide and 4 feet deep in maximum dimension. The voids around and beneath rocks should be filled with approved granular soil having a Sand Equivalent of 30 or greater and should be compacted by flooding. Windrows may also be placed utilizing an "open-face" method in lieu of the trench procedure, however, this method should first be approved by the Consultant.

6.2.5 Windrows should generally be parallel to each other and may be placed either parallel to or perpendicular to the face of the slope depending on the site geometry. The minimum horizontal spacing for windrows shall be 12 feet center-to-center with a 5-foot stagger or offset from lower courses to next overlying course. The minimum vertical spacing between windrow courses shall be 2 feet from the top of a lower windrow to the bottom of the next higher windrow.

6.2.6 Rock placement, fill placement and flooding of approved granular soil in the windrows should be continuously observed by the Consultant.

6.3 *Rock* fills, as defined in Section 3.1.3, shall be placed by the Contractor in accordance with the following recommendations:

6.3.1 The base of the *rock* fill shall be placed on a sloping surface (minimum slope of 2 percent). The surface shall slope toward suitable subdrainage outlet facilities. The *rock* fills shall be provided with subdrains during construction so that a hydrostatic pressure buildup does not develop. The subdrains shall be permanently connected to controlled drainage facilities to control post-construction infiltration of water.

6.3.2 *Rock* fills shall be placed in lifts not exceeding 3 feet. Placement shall be by rock trucks traversing previously placed lifts and dumping at the edge of the currently placed lift. Spreading of the *rock* fill shall be by dozer to facilitate *seating* of the rock. The *rock* fill shall be watered heavily during placement. Watering shall consist of water trucks traversing in front of the current rock lift face and spraying water continuously during rock placement. Compaction equipment with compactive energy comparable to or greater than that of a 20-ton steel vibratory roller or other compaction equipment providing suitable energy to achieve the required compaction or deflection as recommended in Paragraph 6.3.3 shall be utilized. The number of passes to be made should be determined as described in Paragraph 6.3.3. Once a *rock* fill lift has been covered with *soil* fill, no additional *rock* fill lifts will be permitted over the *soil* fill.

6.3.3 Plate bearing tests, in accordance with ASTM D 1196-09, may be performed in both the compacted *soil* fill and in the *rock* fill to aid in determining the required minimum number of passes of the compaction equipment. If performed, a minimum of three plate bearing tests should be performed in the properly compacted *soil* fill (minimum relative compaction of 90 percent). Plate bearing tests shall then be performed on areas of *rock* fill having two passes, four passes and six passes of the compaction equipment, respectively. The number of passes required for the *rock* fill shall be determined by comparing the results of the plate bearing tests for the *soil* fill and the *rock* fill and by evaluating the deflection variation with number of passes. The required number of passes of the compaction equipment will be performed as necessary until the plate bearing deflections are equal to or less than that determined for the properly compacted *soil* fill. In no case will the required number of passes be less than two.

6.3.4 A representative of the Consultant should be present during *rock* fill operations to observe that the minimum number of "passes" have been obtained, that water is being properly applied and that specified procedures are being followed. The actual number of plate bearing tests will be determined by the Consultant during grading.

6.3.5 Test pits shall be excavated by the Contractor so that the Consultant can state that, in their opinion, sufficient water is present and that voids between large rocks are properly filled with smaller rock material. In-place density testing will not be required in the *rock* fills.

6.3.6 To reduce the potential for "piping" of fines into the *rock* fill from overlying *soil* fill material, a 2-foot layer of graded filter material shall be placed above the uppermost lift of *rock* fill. The need to place graded filter material below the *rock* should be determined by the Consultant prior to commencing grading. The gradation of the graded filter material will be determined at the time the *rock* fill is being excavated. Materials typical of the *rock* fill should be submitted to the Consultant in a timely manner, to allow design of the graded filter prior to the commencement of *rock* fill placement.

6.3.7 *Rock* fill placement should be continuously observed during placement by the Consultant.

7. OBSERVATION AND TESTING

7.1 The Consultant shall be the Owner's representative to observe and perform tests during clearing, grubbing, filling, and compaction operations. In general, no more than 2 feet in vertical elevation of *soil* or *soil-rock* fill should be placed without at least one field density test being performed within that interval. In addition, a minimum of one field density test should be performed for every 2,000 cubic yards of *soil* or *soil-rock* fill placed and compacted.

7.2 The Consultant should perform a sufficient distribution of field density tests of the compacted *soil* or *soil-rock* fill to provide a basis for expressing an opinion whether the fill material is compacted as specified. Density tests shall be performed in the compacted materials below any disturbed surface. When these tests indicate that the density of any layer of fill or portion thereof is below that specified, the particular layer or areas represented by the test shall be reworked until the specified density has been achieved.

7.3 During placement of *rock* fill, the Consultant should observe that the minimum number of passes have been obtained per the criteria discussed in Section 6.3.3. The Consultant should request the excavation of observation pits and may perform plate bearing tests on the placed *rock* fills. The observation pits will be excavated to provide a basis for expressing an opinion as to whether the *rock* fill is properly seated and sufficient moisture has been applied to the material. When observations indicate that a layer of *rock* fill or any portion thereof is below that specified, the affected layer or area shall be reworked until the *rock* fill has been adequately seated and sufficient moisture applied.

7.4 A settlement monitoring program designed by the Consultant may be conducted in areas of *rock* fill placement. The specific design of the monitoring program shall be as recommended in the Conclusions and Recommendations section of the project Geotechnical Report or in the final report of testing and observation services performed during grading.

7.5 The Consultant should observe the placement of subdrains, to verify that the drainage devices have been placed and constructed in substantial conformance with project specifications.

7.6 Testing procedures shall conform to the following Standards as appropriate:

7.6.1 Soil and Soil-Rock Fills:

7.6.1.1 Field Density Test, ASTM D 1556-07, *Density of Soil In-Place By the Sand-Cone Method.*

7.6.1.2 Field Density Test, Nuclear Method, ASTM D 6938-08A, *Density of Soil and Soil-Aggregate In-Place by Nuclear Methods (Shallow Depth).*

7.6.1.3 Laboratory Compaction Test, ASTM D 1557-09, *Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 10-Pound Hammer and 18-Inch Drop*.

7.6.1.4. Expansion Index Test, ASTM D 4829-08A, *Expansion Index Test*.

7.6.2 Rock Fills

7.6.2.1 Field Plate Bearing Test, ASTM D 1196-09 (Reapproved 1997) *Standard Method for Nonreparative Static Plate Load Tests of Soils and Flexible Pavement Components, For Use in Evaluation and Design of Airport and Highway Pavements.*

8. PROTECTION OF WORK

8.1 During construction, the Contractor shall properly grade all excavated surfaces to provide positive drainage and prevent ponding of water. Drainage of surface water shall be controlled to avoid damage to adjoining properties or to finished work on the site. The Contractor shall take remedial measures to prevent erosion of freshly graded areas until such time as permanent drainage and erosion control features have been installed. Areas subjected to erosion or sedimentation shall be properly prepared in accordance with the Specifications prior to placing additional fill or structures.

8.2 After completion of grading as observed and tested by the Consultant, no further excavation or filling shall be conducted except in conjunction with the services of the Consultant.

9. CERTIFICATIONS AND FINAL REPORTS

9.1 Upon completion of the work, Contractor shall furnish Owner a certification by the Civil Engineer stating that the lots and/or building pads are graded to within 0.1 foot vertically of elevations shown on the grading plan and that all tops and toes of slopes are within 0.5 foot horizontally of the positions shown on the grading plans. After installation of a section of subdrain, the project Civil Engineer should survey its location and prepare an *as-built* plan of the subdrain location. The project Civil Engineer should verify the proper outlet for the subdrains and the Contractor should ensure that the drain system is free of obstructions.

9.2 The Owner is responsible for furnishing a final as-graded soil and geologic report satisfactory to the appropriate governing or accepting agencies. The as-graded report should be prepared and signed by a California licensed Civil Engineer experienced in geotechnical engineering and by a California Certified Engineering Geologist, indicating that the geotechnical aspects of the grading were performed in substantial conformance with the Specifications or approved changes to the Specifications.

LIST OF REFERENCES

1. *2013 California Building Code, California Code of Regulations, Title 24, Part 2, based on the 2012 International Building Code*, prepared by California Building Standards Commission, dated July, 2013.

2. *ACI 318-11, Building Code Requirements for Structural Concrete and Commentary*, prepared by the American Concrete Institute, dated August, 2011.

3. Anderson, J. G., T. K. Rockwell, and D. C. Agnew, *Past and Possible Future Earthquakes of Significance to the San Diego Region*: Earthquake Spectra, 1989, v.5, no. 2, p.299-333.

4. ASCE 7-10, *Minimum Design Loads for Buildings and Other Structures,* Second Printing, April 6, 2011.

5. Boore, D. M., and G. M Atkinson*, Ground-Motion Prediction for the Average Horizontal Component of PGA, PGV, and 5%-Damped PSA at Spectral Periods Between 0.01 and 10.0 S,* Earthquake Spectra, Volume 24, Issue 1, pages 99-138, February 2008.

6. California Department of Conservation, California Geological Survey (CGS), *State of California Earthquake Fault Zones*, Point Loma Quadrangle, dated May 1, 2003.

7. California Geologic Survey, *Seismic Shaking Hazards in California, Based on the USGS/CGS Probabilistic Seismic Hazards Assessment (PSHA) Model,* 2002 (revised April 2003). 10% probability of being exceeded in 50 years.
http://redirect.conservation.ca.gov/cgs/rghm/pshamap/pshamain.html

8. California Geologic Survey, *Tsunami Inundation Map For Emergency Planning, State of California- County of San Diego, Point Loma Quadrangle,* dated June 1, 2009.

9. Campbell, K. W., and Y. Bozorgnia, *NGA Ground Motion Model for the Geometric Mean Horizontal Component of PGA, PGV, PGD and 5% Damped Linear Elastic Response Spectra for Periods Ranging from 0.01 to 10 s,* Preprint of version submitted for publication in the NGA Special Volume of Earthquake Spectra, Volume 24, Issue 1, pages 139-171, February 2008.

10. Chiou, Brian S. J., and Robert R. Youngs, *A NGA Model for the Average Horizontal Component of Peak Ground Motion and Response Spectra,* preprint for article to be published in NGA Special Edition for Earthquake Spectra, Spring 2008.

11. *City of San Diego Seismic Safety Study, Geologic Hazards and Faults,* 2008 edition, Map Sheet 17.

12. Geocon Incorporated, *Fault Investigation, Ace Parking Site, Fifth Avenue and G Street, San Diego, California*, dated November 9, 1995 (Project No. 05612-51-01).

13. Geocon Incorporated, *Fault Investigation, Sixth and F Street Commercial Site, San Diego, California*, dated November 15, 1995 (Project No. 05611-51-01).

14. Geocon Incorporated, *Fault Investigation, Seventh and F Street Commercial Site, San Diego, California*, dated February 20, 1996 (Project No. 05680-51-01).

15. Geocon Incorporated, *Geotechnical Recommendations for Mechanically Stabilized Earth Retaining Wall and Preliminary Pavement Design, Central Apartments, 831-845 Sixth Avenue, San Diego, California,* dated May 1, 2008 (Project No. 07980-52-01).

16. Geocon Incorporated, *Report of Testing and Observation Services Performed During Site Grading and Improvements, Temporary Parking Lot, Gaslamp Apartments, 831-845 Sixth Avenue, San Diego, California,* dated August 11, 2008 (Project No. 07980-52-01).

17. Geocon Incorporated, *Preliminary Geotechnical and Geologic Fault Investigation, Gaslamp Apartments 831 through 845 Sixth Avenue, San Diego, California,* dated July 10, 2008 (Project No. 07980-52-01).

18. Kennedy, M. P,. and S. S. Tan, *Geologic Map of the San Diego 30'x60' Quadrangle, California,* USGS Regional Map Series Map No. 3, Scale 1:100,000, 2008.

19. Legg, M. R., J. C. Borrero, and C. E. Synolakis, *Evaluation of Tsunami Risk to Southern California Coastal Cities*, 2002 NEHRP Professional Fellowship Report, dated January 2002.

20. Lindvall, S. C., T. K. Rockwell, and C. E. Lindvall, *The Seismic Hazard of San Diego Revised: New Evidence for Magnitude 6+ Holocene* E*arthquakes on the Rose Canyon Fault Zone*: Proceedings of the Fourth U.S. National Conference on Earthquake Engineering, 1990, 11 p.

21. Risk Engineering, *EZ-FRISK,* 2015.

22. Rockwell, T. K., S. C. Lindvall, C. K. Haraden, K. Hirabayashi, and E. Baker, *Minimum Holocene Slip Rate for the Rose Canyon Fault in San Diego, California,* in Environmental Perils, San Diego Region, ed. P. L. Abbott and W. J. Elliot, 1991, p. 37-46., San Diego Association of Geologists.

23. Streiff, D., D. Elder-Mills, E. A. Artim, 1982, *The Rose Canyon Fault Zone, Downtown San Diego and Coronado*, Geologic Studies in San Diego, San Diego Association of Geologists Field Trip Guide, April 1982.

24. Treiman, Jerome A., *The Rose Canyon Fault Zone Southern California,* California Division of Mines and Geology Publication, 1993.

25. United States Geological Survey computer program, *Seismic Hazard Curves and Uniform Hazard Response Spectra.* http://geohazards.usgs.gov/designmaps/us/application.php

26. Unpublished Geotechnical Reports and Information, Geocon Incorporated.

27. USGS computer program, *Seismic Hazard Curves and Uniform Hazard Response Spectra,* http://earthquake.usgs.gov/research/hazmaps/design/.