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GEOTECHNICAL REPORT

PLANNED DEVELOPMENT

1093 ~~3848~~ San Bruno Avenue,
Brisbane, California
Project No. 22-0AT15

Prepared by: Modern Technology Resources, Inc.
September 15th, 2017





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Registered Geotechnical Engineer No. 2873

Note: Correct address is 1093 San Bruno Ave.

INTRODUCTION

Modern Technology Resources, Inc. has been retained to serve as a geotechnical engineer of record at the project 3848 San Bruno Avenue, Brisbane, California. In this report, we present the results of our soil and rock investigation for the proposed development to be constructed at 3848 San Bruno Avenue, Brisbane, California. The purpose of this investigation was to provide recommendations concerning the soil and geotechnical engineering of the proposed development. This investigation was conducted using available plans and data, including: published maps, U.S.G.S. seismic design parameters, exploratory boreholes excavation data. Also, as a part of our service, we will review structural engineering plans for conformance with geotechnical recommendations.

The scope of our work included a site reconnaissance by the undersigned, subsurface exploration, engineering analyses of the available data, and the preparation of this report, which includes geotechnical engineering recommendations for the proposed residence.

PLANNED CONSTRUCTION

Proposed development will consist of the single family conventional wood frame residential construction at the subject site 3848 San Bruno Avenue, Brisbane, California

INFORMATION REVIEWED

An initial site reconnaissance and following site subsurface exploration was performed by Igor Kleyner (GE2873, CE68621). The site work included excavation of the six exploratory boreholes using a manually operated and power portable augers at the subject property. We encountered one to 4-feet of brown sandy silty soil with gravel and bedrock debris inclusions before encountering bedrock material. Bedrock is hard, dark grayish sandstone inter bedded with thin shale layers. The degree of weathering is generally reduced with depth. Approximate location of the boreholes, logs of the exploratory boreholes, and details regarding soil classification and bedrock hardness are including in Figures 6 - Figure12.

1. SITE INVESTIGATION

SITE DETAILS

The subject property consists of an existing building at Parcel # 007553160 located in Brisbane, CA. The site is on a northwest-southeast trending street bound by Joy Avenue to the north and McLain Road Street to the south. Elevation above sea level at the site is approximately 65.9 feet at the center of the parcel (Figure 1). Topography at the site is moderate to steeply sloping. The main geotechnical engineering issue connected with construction at the subject property is a moderate downhill sloping with steep downhill slope near the property line.

The following geotechnical investigation addresses this issue.

a. Geologic.

The subject property is mapped on Quaternary-age slope debris (Figure 2). The material is described as yellowish-orange to medium gray silty to sandy clay with rock fragments (Bonilla, 1998). It may locally contain silty to clayey sand or gravel and be unstratified or poorly stratified. Rocks fragments in the debris likely consist of the sandstone and shale bedrock located just upslope of the site. The bedrock, which forms the adjacent San Bruno Mountains, consists of interbedded sandstone and shale which may be fresh to highly weathered. Locally, bedrock dips northeast in the downslope direction.

b. Surficial Soils.

Personnel from this office visited the subject property on September 6th, 2017 to explore subsurface conditions and collect samples for laboratory analysis. Six boreholes were excavated.

Excavations revealed approximately 4 feet of very dense, brown, well-graded, fine silty sand with bedrock debris. This layer was underlain by hard sandstone bedrock where practical refusal was met. Ground water was not observed in any borehole. Excavated material is consistent with what is expected from the geologic map.

c. Groundwater.

Recent excavations conducted on September 6th, 2017 did not encounter ground water. We acknowledge that fluctuations in groundwater level may occur due to variations in rainfall, landscaping, and other natural and human factors which may not have been evident during our measurements.

d. Laboratory Testing.

Tests were performed as per procedures in ASTM D2487-11. A majority of grains were retained in the No.200 and up sieves indicating mostly fine sand. Material is generally well-graded and a range of grain sizes are represented.

Table 1. Borehole-1 Sample 1, 1' depth.

Sieve #	Opening (mm)	Grain Size	% of Total	Cumm. %	% Finer
No. 4	4.75	Fine gravel	0.00	0.00	100.00
No. 10	2.00	Very coarse sand	2.03	2.03	97.97
No. 20	0.85	Coarse sand	11.50	13.53	86.47
No. 40	0.425	Medium sand	24.04	37.57	62.43
No.200	0.075	Fine sand	45.66	83.23	16.77
Pan	0.01	Fines	16.77	100.00	0.00

Table 2. Borehole-1 Sample 2, 2' depth.

Sieve #	Opening (mm)	Grain Size	% of Total	Cumm. %	% Finer
No. 4	4.75	Fine gravel	0.00	0.00	100.00
No. 10	2.00	Very coarse sand	3.94	3.94	96.06
No. 20	0.85	Coarse sand	7.45	11.40	88.60
No. 40	0.425	Medium sand	22.15	33.55	66.45
No.200	0.075	Fine sand	48.10	81.65	18.35
Pan	0.01	Fines	18.35	100.00	0.00

Table 3. Borehole-1 Sample 3, 3' depth.

Sieve #	Opening (mm)	Grain Size	% of Total	Cumm. %	% Finer
No. 4	4.75	Fine gravel	0.00	0.00	100.00
No. 10	2.00	Very coarse sand	2.44	2.44	97.56
No. 20	0.85	Coarse sand	12.19	14.63	85.37
No. 40	0.425	Medium sand	19.05	33.68	66.32
No.200	0.075	Fine sand	49.96	83.64	16.36
Pan	0.01	Fines	16.20	99.84	0.00

Table 4. Borehole-3 Sample 1, 1' depth.

Sieve #	Opening (mm)	Grain Size	% of Total	Cumm. %	% Finer
No. 4	4.75	Fine gravel	24.19	24.19	75.81
No. 10	2.00	Very coarse sand	4.60	28.79	71.21
No. 20	0.85	Coarse sand	9.45	38.24	61.76
No. 40	0.425	Medium sand	20.57	58.81	41.19
No.200	0.075	Fine sand	30.30	89.12	10.88
Pan	0.01	Fines	10.88	100.00	0.00

Table 5. Borehole-4 Sample 1, 6" depth.

Sieve #	Opening (mm)	Grain Size	% of Total	Cumm. %	% Finer
No. 4	4.75	Fine gravel	15.35	15.35	84.65
No. 10	2.00	Very coarse sand	5.75	21.10	78.90
No. 20	0.85	Coarse sand	10.29	31.39	68.61
No. 40	0.425	Medium sand	14.37	45.75	54.25
No.200	0.075	Fine sand	49.01	94.76	5.24
Pan	0.01	Fines	5.25	100.00	0.00

2. GEOLOGIC HAZARDS

The San Francisco Bay Area is in a seismically active region of the United States. Fault movement can be frequent and unpredictable; therefore, the Bay Area is highly susceptible to the effects of earthquakes, landslides, and other geologic hazards. Understanding the geological conditions at the project site significantly aids in building construction and other civil engineering projects. The following hazards with potential to impact the proposed construction are summarized below.

2.1 Landsliding.

No known active landslides are mapped at the project site according to the San Francisco South geologic map (*Figure 2*). During reconnaissance, evidence of landsliding at the site was not observed. Topography at the site is moderate to steeply sloping. As mentioned above the main geotechnical engineering issue connected with construction at the subject property is a steep downhill slope near the property line. To mitigate this potential hazard we suggest to use drilled concrete piers to support gravity and lateral loads. The stiffened interconnected spread footing may be utilized if footings set back far enough from steep slope. As recommended below. We will observe slope stability during excavation and the following construction and will suggest slope stability measures if it will be required.

2.2 Seismic Ground Shaking.

Intensities of an earthquake are strongly dependent on the distance from the ruptured fault and geological character of the ground. Earthquake effects must be considered due to the site's proximity to several active faults in the Bay Area. Any improvements should be designed according to current earthquake standards. The USGS standard design parameters with 2015 NEHRP Provisions are included in this report in *Appendix A*.

2.3 Fault-Rupture & The Alquist-Priolo Act.

The project site at 3848 San Bruno Avenue is not within an Alquist-Priolo Earthquake Fault Zone, according to the Earthquake Zones of Required Investigation for the San Francisco South Quadrangle (*Figure 3*). The Alquist-Priolo Earthquake Fault Zoning Act was passed in 1972 to prevent the construction of buildings used for human occupancy on the surface trace of active faults (*CGS, 2002*). Because no known active faults directly intersect the project site, the overall risk for surface fault-rupture is low.

2.4 Liquefaction.

The subject property is mapped in an area of very low liquefaction susceptibility (*Figure 4*). Material underlying the subject property is very dense to hard, decreasing the risk of liquefaction during an earthquake event. Locally, liquefiable material is found at elevations below the subject property.

2.5. Soil Erosion Control.

Exposed soil slopes are subject to varying degrees of erosion from surface water, and in some degree, from wind scour. The quantity of soil eroded from a slope will normally decrease over time, as the easily transported soil particles are moved early in the life of the slope. For steep slopes, the initial stages of erosion are rapid and extensive, and are usually accompanied by sloughing and even sliding. Standard methods of soil erosion control should be employed at the site during and after the construction of the building.

3. SEISMICITY

3.1 Regional Tectonics.

The San Francisco Bay Area is seismically active due to the region's proximity to a major plate boundary. The Pacific Plate, located west of the project site, moves northwest relative to the North American Plate at a rate of about two inches per year. Movement between the plates is accommodated by right-lateral slip on fault systems, most notably along the San Andreas Fault Zone which has slipped approximately 200 miles over 23 million years.

The San Andreas Fault is a complex system of parallel and interconnecting faults, and fault activity varies along sections of the fault. Related active faults of interest in the Bay Area include the major San Gregorio, Hayward, and Calaveras Fault Zones, and other fault splinters of lesser activity such as the Rodgers Creek Fault, Concord-Green Valley Fault, and the Greenville Fault (*Figure 5*) (*Jennings, 2010*). Faults are all roughly parallel to one another, and exhibit primarily right-lateral motion. Historical large magnitude earthquakes include the M7.9 Great San Francisco Earthquake in 1906 and the M6.9 Loma Prieta Earthquake in 1989.

3.2 Fault Distances.

According to the California Geological Survey, an active fault is defined as causing surface displacement within Holocene time (the last ~11,000 years). The locations of the five nearest active faults are described below. All faults are generally northwest trending.

Table 6. Active fault distances from project site.

Active Fault	Approximate Distance from Project Site		Orientation from Project Site
San Andreas	4.5 mi	7.2 km	West
San Gregorio	10 mi	16.1 km	West to southwest
Hayward	14 mi	22.5 km	East
Calaveras	23 mi	37 km	East to southeast
Mt. Diablo	24 mi	38.6 km	East to northeast

3.3. Earthquake Probabilities.

In a 2008 collaborative study, the U.S. Geological Survey, California Geological Survey, and the Southern California Earthquake Center analyzed the probabilities of earthquake occurrence in the Bay Area. Geologists and seismologists concluded that there is a 63% chance that a 6.7 magnitude or greater earthquake will occur on a Bay Area fault before the year 2038. The Hayward and San Andreas faults are predicted to be the most likely to produce a 6.7 magnitude or greater earthquake, at 31% and 21% chance, respectively. *For a detailed map of Bay Area fault locations, earthquake probabilities, and proximity to the project site, see Figure 5.*

3.4. Seismic Design Parameters.

The U.S. Geological Survey calculates gridded values of seismic design parameters. The tool is built in accordance with design code procedures and earthquake hazard information. It should be noted that these parameters will not prevent damage to existing structures; their main purpose is to prevent catastrophic collapse. The complete report is found in *Appendix A*. If the improvements on the site are designed using the 2015 NEHRP Provisions, the following seismic design parameter values apply:

Table 7. Seismic design parameters*.	
Categorization	Design Value
S_s	1.695
S_1	0.689
F_a	1.2
F_v	1.7
S_{MS}	2.035
S_{M1}	1.172
S_{Ds}	1.356
S_{D1}	0.781
PGA	0.732
PGA_M	0.878
F_{PGA}	1.2

*Based on Site Class D (default): Stiff Soil and Risk Category I/II/III, using 2015 NEHRP Provisions.

4. CONCLUSIONS AND RECOMMENDATIONS

Thus, we conclude that the site is suitable for proposed residential development from a geotechnical and geo-hazards standpoint. However, our office observation and testing during earthworks and foundation installation will be required to confirm subsoil investigation information.

Topography at the site is moderate to steeply sloping. The main geotechnical engineering issue connected with construction at the subject property is a steep downhill slope near the property line. The following geotechnical recommendations address this issue.

The geotechnical recommendations satisfy the intent of California Building Code Section 1806-5-6. Modern Technology Resources, Inc. recommends that the proposed structure may be supported on a drilled piers. Drilled piers may be also used for temporary shoring and as foundations of retaining walls. The stiffened interconnected spread footing may be utilized if footings set back far enough from steep slope.

GEOTECHNICAL RECOMMENDATIONS AND DESIGN CRITERIA

Detailed soil and foundation engineering recommendations are presented in the subsequent sections of this report. All conclusions and recommendations presented in this report are contingent upon our office being retained to:

- observe excavations and grading,
- observe foundation installation,
- observe and test the over-excavated and re-compacted soils under the slabs-on-grade,
- observe the installation of the pier foundations,
- observe the installation of drains behind the retaining walls,
- observe the installation of recommended surface drainage facilities.

4.1. EARTHWORK OPERATIONS

a. Demolition and Clearing.

The area of the proposed improvement should be cleared of all obstructions including natural debris and existing improvements. After removal of the debris, areas to be graded should be cleared of debris, deleterious materials, and vegetation, and then stripped of the upper soils containing roots and organic matter. We anticipate that the required depth of stripping will generally be several inches. Deeper stripping may be required to remove localized concentrations of organic matter, such as tree roots. All the cleared and stripped materials should be hauled off the site or may be stockpiled for later use as topsoil in landscaping areas.

b. Excavation.

We expect that excavations in the soil and upper portions of weathered bedrock at the site can be conducted with conventional equipment, although ripping at some locations may be required. Excavations extending deeper into the bedrock may require extra effort, such as heavy ripping, hoe-rams, or jack- hammering. We anticipate that the bedrock will become harder and more massive with increasing depth.

c. Over-excavation.

Loose, porous soils and topsoil, if encountered, should be over excavated in areas designated for placement of future engineered fill or base of improvements. Difficulty in achieving the recommended minimum degree of compaction can be used to identify areas of weak soils

that should be removed and replaced as engineered fill. The depth and extent of excavation should be approved in the field by the geotechnical engineer prior to placement of fill or improvements.

d. Engineering Fill.

On-site soil proposed for use as structural fill should be inorganic, free from deleterious materials. The suitability of existing fill soil for reuse as a structural fill should be determined by a member of our staff at the time of grading. Prior to delivery to the site, proposed import should be tested in our laboratory to verify its suitability for use as structural fill and, if found to be suitable, further tested to estimate the water content and density at which it should be placed. All grading must be observed by a representative of our office. After the site is properly cleared, the excavation operations performed, any required fill may be placed. We recommend that all slab»-on- grade be supported on at least 1-feet of engineered fill materials, select existing on-site materials that are over- excavated and re-compacted or imported fill materials.

The exposed subgrade at the bottom of all excavations should be inspected by a representative of our office. Any detrimental materials exposed at the subgrade level should be removed to depths specified by our field representative and replaced with fill compacted to the requirements given below.

The exposed subgrade in areas to receive fill should be scarified, moisture-conditioned, and compacted to the requirements given below. All structural fill materials placed at the site should not contain rocks or lumps greater than 6-inches in greatest dimension with not more than 15 percent larger than 2½-inches.

All structural fill placed at the site should be compacted to at least 90 percent relative compaction by mechanical means only, as determined by ASTM Test Designation D1337-7 Q. The upper 6-inches of subgrade soil under slabs-on- grade should, however, be compacted to at least 95 percent relative compaction. The fill materials should be placed in lifts not exceeding 8-inches in un-compacted thickness.

e. Excavations Shoring and Underpinning.

During excavations adjacent to existing structures or footings, care should be taken to adequately support the existing structures. When excavating below the level of foundations supporting existing structures, some form of underpinning may be required where excavations extend below an imaginary plane sloping at 1:1 downward and outward from the edge of the existing footings. All temporary underpinning and shoring design and construction are the responsibility of the contractor. We are available to provide consultation regarding underpinning adjacent improvements and shoring.

f. Temporary Slopes.

Temporary slopes will be necessary during the planned site excavations. In order to safely develop the site, temporary slopes will need to be laid back in conformance with OSHA standards at safe inclinations, or temporary shoring will have to be installed. All temporary slopes and shoring design are the responsibility of the contractor. MTR, Inc. is available to provide consultation regarding stability and support of temporary slopes during construction.

We suggest that during the excavations operations, temporary slopes should have a maximum vertical face of 3-feet with temporary cut slopes above the vertical face having a maximum inclination of 1:1 (horizontal to vertical) in approved clayey materials. If poor quality materials or seepage are encountered in the excavations the temporary slopes will have to be appropriately flattened. However, in very competent bedrock materials the inclination of the temporary slopes may be increased. The materials exposed in the excavations should be evaluated by a representative of our office during excavation operations.

g. Finished Slopes.

We recommend that any new cut and fill slopes at the site have a maximum slope of 2:1 (horizontal to vertical). The tops of cut slopes should be rounded and compacted to reduce the risk of erosion. Fill and cut slopes should be planted with vegetation to resist erosion, or protected from erosion by other measures, upon completion of grading. Surface water runoff should be intercepted and diverted away from the tops and toes of cut and fill slopes by using berms or ditches.

All unsupported fill place on the site should be keyed and benched into competent materials; such fill should be thoroughly compacted to the face of the slopes by continually track-rolling the slopes as fill is being placed or by overfilling the slopes by 1- to 2-feet and then cutting back the slopes after the filling operations are completed.

h. Drainage and Erosion Control.

Surface drainage gradients should be planned to prevent ponding and to promote drainage of surface water away from building, slabs, edges of pavements and sidewalks, and towards suitable collection and discharge facilities.

Control of surface water runoff will require the construction of a concrete lined surface ditch to intercept rainwater runoff during periods of heavy precipitation. Surface drainage ditches will control the quantity and velocity of surface water runoff and prevent inundation of the adjacent down-slope lots. The exact location of this drainage facility should be determined in the field by a representative from our office at the time of construction.

The surface drainage ditch should be constructed of reinforced concrete and have a minimum width of 2-feet and a minimum depth of l-foot. The ditch should be sloped to drain toward catch basins and the collected water should be transported through closed pipes to suitable discharge facilities. The ditch and catch basins will require periodic cleaning and maintenance to function properly.

We recommend that all new cut and/or fill slopes and any existing slopes that are disturbed during the construction operations be covered with jute mesh (or the equivalent) and heavily planted. Site irrigation should be done only as required for plant survival. We recommend that rainwater collected on the roof of the building be transported through gutters, down spouts and closed pipes to approved discharge facilities.

5. FOUNDATIONS

General.
The thickness of soil covering of the bedrock varies significantly across the site. Design criteria are provided for foundations based on soil and rock. We recommend using the soil values, unless bedrock has been observed in excavations at the subject site.

It is our opinion that the planned improvements may be supported on drilled piers extending into competent bedrock. Spread footings may also be used if the footings are set back more than 20 feet from ground surface sloping down and away from the footings at an inclination of 1:1 or 15 feet at an inclination 5:1 (horizontal: vertical) or steeper, but less than 2:1. Where the slope is steeper than 1 unit vertical in 1unit horizontal the required setback shall be measured from an imaginary plane 45 degrees to the horizontal, projected upward from the toe of the slope.

Spread footings may also be used within or near sloping portions of the site if bedrock at the site is encountered near the ground surface and the footings are imbedded into competent bedrock. Design criteria for each foundation type are presented below. If it would be required we can carry out slope stability analysis.

5.1. Drilled Piers

Drilled, cast-in-place, straight-shaft reinforced concrete piers may be used as foundation and also for shoring excavations, retaining walls and underpinning adjacent improvements. The piers should be designed to develop their load-carrying capacity through friction between the sides of the piers and the surrounding subsurface materials. Friction piers should have a minimum diameter of 18 inches. Piers designed to resist lateral loads from retaining walls should extend below grade a minimum of 18 feet. The spacing of the piers should be determined by the Structural Engineer, but in no case shall the center-to- center spacing of the piers be closer than three pier diameters. All piers should extend to a minimum depth of

5-feet into approved bedrock materials, or to practical drilling refusal. The actual lengths of the piers can be determined using an allowable skin friction value of 600 pounds per square foot for dead plus live loads with a one-third increase for all loads including wind or seismic. These values can be used starting at a depth of 4-feet below the grade.

The weight of the foundation concrete extending below grade may be disregarded. In portions of the site with an inclination of 5: 1 (horizontal: vertical) or steeper we recommend piers be interconnected with grade or tie beams and the foundation system should have upslope- downslope elements spaced no more than 20 feet apart, where practical. Field adjustment to final pier depths by our office representative should be expected during piers installation.

All drilled piers must be designed to account for potential "creep" type movement of the surface soils and the weathered upper portion of the bedrock materials. We recommend that the piers be designed to resist an uniform lateral pressure of 250 pounds per square foot acting against 2 the projected diameters of the piers to a depth of 4-feet below the grade.

Resistance to lateral displacement of individual piers will be generated primarily by passive earth pressures acting against two pier diameters. Passive pressures should be assumed equivalent to those created by a fluid with weight 300 pcf to a maximum value of 3000 pounds per square foot. Passive pressures should be disregarded for the uppermost 4 feet of pier depth and in areas with less than 15 feet of horizontal confinement. Where the slope is steeper than 1 unit vertical in 1unit horizontal the required setback shall be measured from an imaginary plane 45 degrees to the horizontal, projected upward from the toe of the slope.

The loose materials overlie the competent natural soils that the piers will be embedded into. Therefore, as a result of these surface conditions, caving of loose soils should be anticipated during the drilling of the piers; this may require the use of temporary casing during the installation of the piers. Concrete should be pumped into each hole as soon as practical after the drilling. The concreting operations in each pier should be carefully performed so that the concrete fills the entire excavated hole.
Pier bottoms should be dry and reasonably free of loose cuttings and fall-in prior to installing reinforcing steel and placing concrete. Where groundwater is encountered during pier shaft drilling, it should be removed by pumping from the holes prior to placing steel and concrete; or, concrete could be placed underwater by the tremie method.

If the pier shafts will not stand open, temporary casing may be necessary to support the sides of the pier shafts until concrete is placed. Concrete should not be allowed to fee fall more than 5 feet to avoid segregation of the aggregate.

5.1. Spread Footings.

Spread footings should extend at least 18 inches below lowest adjacent grade. If soft or unstable soil areas are encountered at the bottom of the footings, deepening of the footings will be necessary. Footing depths may be reduced if competent bedrock is exposed in excavations. Footings should be stepped to produce level tops and bottoms and should be deepened as necessary to provide at least 20 feet of horizontal clearance between the portions of footings designed to impose passive pressures and the face of the nearest slope or retaining wall.

Spread footings founded in soil can be designed to impose dead plus code live load bearing pressures and total design load bearing pressures of 2,500 and 3,500 psf, respectively. If foundations are placed in bedrock, the footings may be designed for maximum allowable rock contact pressures of 3500 pounds per square foot (psf) for dead plus sustained live loads, and 4,500 psf for total loads, including wind or seismic forces.

Isolated footing pads should be avoided, where possible. In the areas with an inclination of 5: 1 (horizontal: vertical) or more, we recommend that all new footings be interconnected and the foundation system should have upslope-downslope elements spaced no more than 15 feet apart, where practical.

Resistance to lateral pressures can be obtained from passive earth pressures against the face of the footing and soil friction along the base of footings. A passive pressure equivalent to that obtained using a fluid weight of 250 pounds per cubic foot (pcf) and a friction factor of 0.33 may be used to resist lateral forces and sliding in soil. These values include a safety factor of 1.5 and may be used in combination without reduction. Passive pressures should be neglected within 12 inches of the ground surface in areas not confined by slabs or pavements.

6. SLAB-ON-GRADE

Exterior concrete flatwork constructed on or near sloping portions of the site may experience displacement due to soil creep. To reduce the potential displacement of slabs from soil creep, the slabs may be structurally tied to improvements supported on foundations and/or edge beams extending below the zone of creeping soils may be used along the downslope edge of the flatwork. If selected, edge beams should be at least 6 inches wide and extend at least 30 inches below grade.

As the soil shrinks and swells, light weight improvements supported on the expansive soils may experience movement that can result in cracking and differential displacements. Potential damage to flatwork may be reduced by moisture conditioning the surficial soils to wet of optimum moisture content and maintaining them in this condition until improvements are constructed. Moisture barriers or edge beams around the perimeter of the flatwork may be used to seal the moisture into the soil.

We recommend that concrete slab-on-grade floors and exterior flatwork be at least 4 inches in thickness and reinforced with at least 1/2 -inch-diameter' steel bars placed at no more than 18 inches on center in both directions. Reinforcing steel should extend through joints to provide resistance to differential movement between sections. On exposed slabs and flatwork, crack control joints should be constructed no more than 15 feet apart in both the longitudinal and transverse directions. Closer spacing of crack control joints will reduce the potential for cracking through the middle of slab sections.

The subgrade soil in slab and flatwork areas should be proof rolled to provide a firm, non-yielding surface. If moisture penetration through the slab would be objectionable, slabs should be underlain by a capillary moisture break consisting of at least 4 inches of clean, free-draining crushed rock or gravel graded such that 100 percent will pass the l -inch sieve and none will pass the No. 4 sieve. Further protection against slab moisture penetration can be provided by means of a moisture vapor barrier membrane, placed between the drain rock and the slab. The membrane may be covered with 2 inches of damp, clean sand to protect it during construction.

7. RETAINING WALLS

All retaining walls constructed at the site must be designed to resist lateral earth pressures and any additional lateral pressures that may be caused by surcharge loads applied at the ground surface behind the walls.

Design criteria are provided in this section for retaining walls. We recommend that unrestrained walls with a level surface or with a sloping surface flatter than 4:1 be designed to resist an equivalent fluid pressure of 40 pounds per cubic foot. Where the sloping surface is at an inclination of 2:1 or steeper the unrestrained walls should be designed to resist an equivalent fluid pressure of 65 pounds per cubic foot. For walls with a sloping surface at an inclination between 4:1 and 2: 1, a straight line interpolation between the 40 and 65 pounds per cubic foot may be used.

In areas where bedrock is exposed and backfil1 is placed behind the wall, the structural engineer may use active lateral earth pressures equivalent to those exerted by a fluid weighing 35 pcf where the back slope is level, and 50 pcf for backfill at a 2:1 (horizontal: vertical) slope. If the retaining wall is constructed directly against the bedrock with no backfill the active lateral earth pressures equivalent to those exerted by a fluid weighing 30 pcf where the back slope is level, and 40 pcf for backfill at a 2: 1 (horizontal: vertical) slope may be used. For intermediate slopes, interpolate between these values. We should be consulted to calculate lateral pressures on the tied-back or braced retaining walls.

We recommend that restrained walls be designed to resist the equivalent pressures given above plus an additional uniform lateral pressure of 8H pounds per square foot where H height of backfill above the top of the wall footing in feet.

In addition to lateral earth pressures, retaining walls must be designed to resist horizontal pressures that may be generated by surcharge loads applied at or near the ground surface. Where an imaginary 1:1 (H:V) plane projected downward from the outermost edge of a surcharge load of foundation intersects a retaining wall, that portion of the wall below the intersection should be designed for an additional horizontal thrust from a uniform pressure equivalent to one-third or one-half of the anticipated surcharge load depending on whether the wall is unrestrained or restrained. In some cases, this value yields a conservative estimate of the actual lateral pressure imposed. We should be contacted if a more precise estimate of lateral loading on the retaining wall from surcharge pressures is desired.

The above pressures assume that sufficient drainage will be provided behind the walls to prevent the build-up of hydrostatic pressures from surface and subsurface water infiltration. Adequate drainage may be provided by a sub-drain system consisting of 4-inch diameter perforated pipes bedded in permeable material. The permeable material should consist of clean gravel that is wrapped with a synthetic filter fabric. For either system, the permeable material placed behind the walls should be at least l-foot in width and should extend to within 1.5-feet of finished grade. The upper 1.5-feet of backfill should consist of compacted on-site materials. The sub-drain pipes should be connected to a system of closed pipes that lead to the city storm drainage facilities. Any building walls should be appropriately waterproofed by an approved foundation waterproofing system.

As an alternative to crushed rock Miradrain, Enkadrain, or other geosynthetic drainage panels approved by this office may be used for retaining wall drainage. If used, the drainage panels should extend from a depth of 18 inches below finish grade to the base of the retaining wall. An approved 2-feet section of crushed rock wrapped in filter fabric should be installed around the drainpipe, as discussed previously. Geosynthetic drainage panels should be installed in strict compliance with manufacturer's recommendations with filter fabric against the crushed rock and soil backfill.

Lined surface ditches should be provided behind any retaining walls that will have exposed sloping surfaces draining toward them. These ditches, will collect runoff water from the slopes, should be sloped to drain to discharge facilities. The top of the walls should extend to at least 1- foot above the ditch.

The retaining walls should be supported on pier foundations, or spread footings designed in accordance with the recommendations presented above.

FOLLOW UP GEOTECHNICAL SERVICES

Our recommendations are based on the assumption that Modern Technology Resources, Inc. will be commissioned to perform the following services:

1. Observe, test and advise during grading, excavation and compaction operations.
2. Observe, test and advise during piers installation.
3. Test proposed capillary break material that will be used beneath concrete slabs-on-grade and advise on suitability.
4. Observe and advise during grade beams and retaining walls construction.
5. Observe, test and advise during utility trench backfilling and landscaping.

LIMITATIONS

The recommendations contained in this report are based on the plans and data that have been provided to us. Any change in that plan, information, and data will render our recommendations invalid unless we are commissioned to review the changes and make the necessary modifications and/or additions to our recommendations. The recommendations in this report are contingent on conducting an adequate monitoring program during construction of the proposed development. Unless the construction monitoring and testing program is provided by or coordinated with our firm, Modern Technology Resources, Inc. will not be held responsible for compliance with design recommendations presented in this report.

Our services have been provided in accordance with generally accepted geotechnical engineering practices. No warranties are made, express or implied, as to the professional opinions or advice provided. Recommendations contained in this report are valid for a period of 2 years; after 2 years they must be reviewed by this firm to determine whether or not they still apply.

Should you have any questions related to the contents of this geotechnical report, please do not hesitate to contact our office.

Sincerely,

Igor Gary Kleyner, Ph.D., C.E., G.E.
Principle
Modern Technology Resources, Inc.

September 10, 2017





Kleyner

MTR INC

2.0 REFERENCES

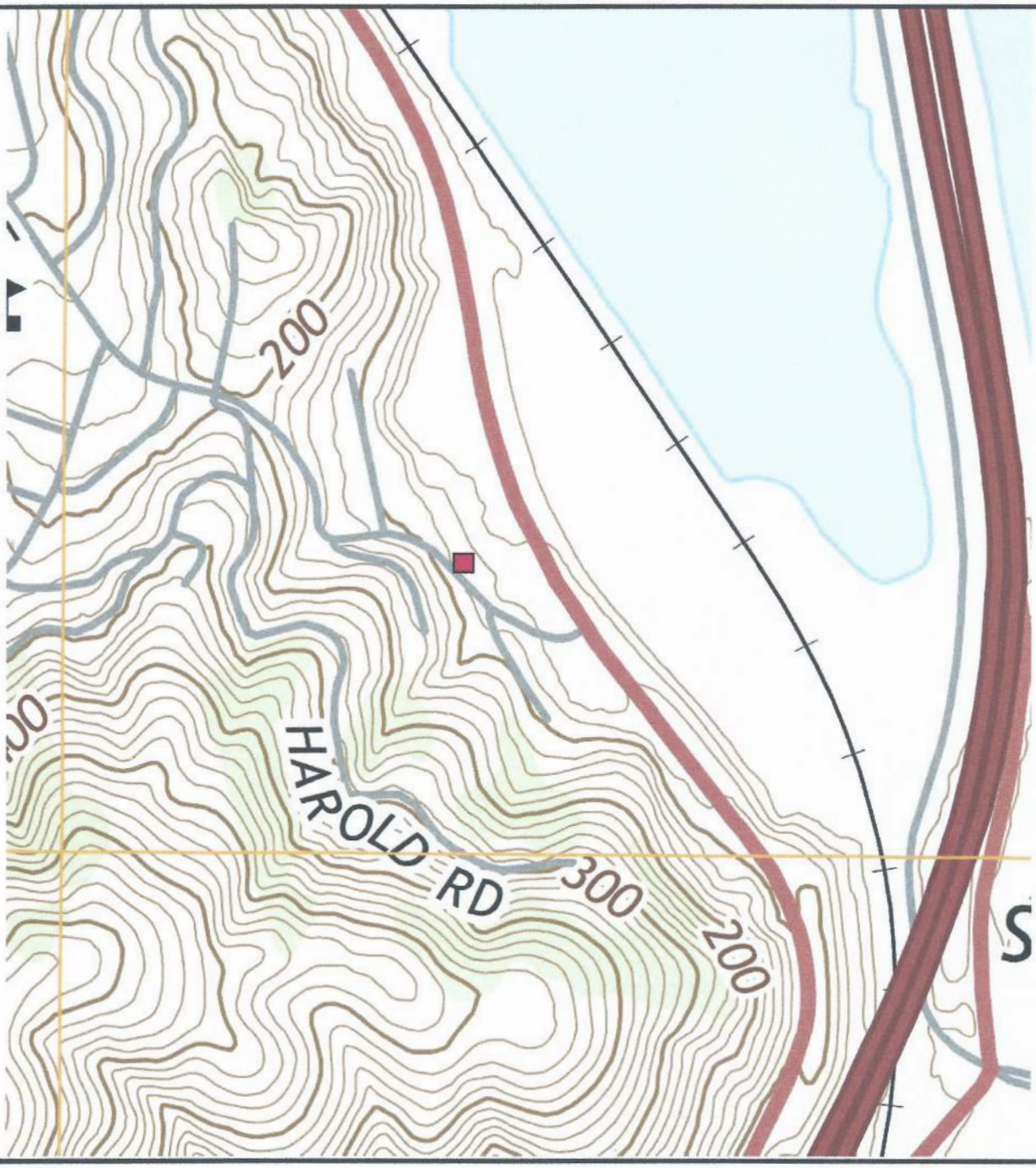
Bonilla, M.G., 1998, Preliminary Geologic Map of the San Francisco South 7.5' Quadrangle and

Part of the Hunter's Point 7.5' Quadrangle, San Francisco Bay Area, California: USGS, U.S. Department of the Interior, Open-File Report 98-354.

California Geological Survey (CGS), 2002, Earthquake Zones of Required Investigation San Francisco South Quadrangle: Department of Conservation, map scale 1:24000.

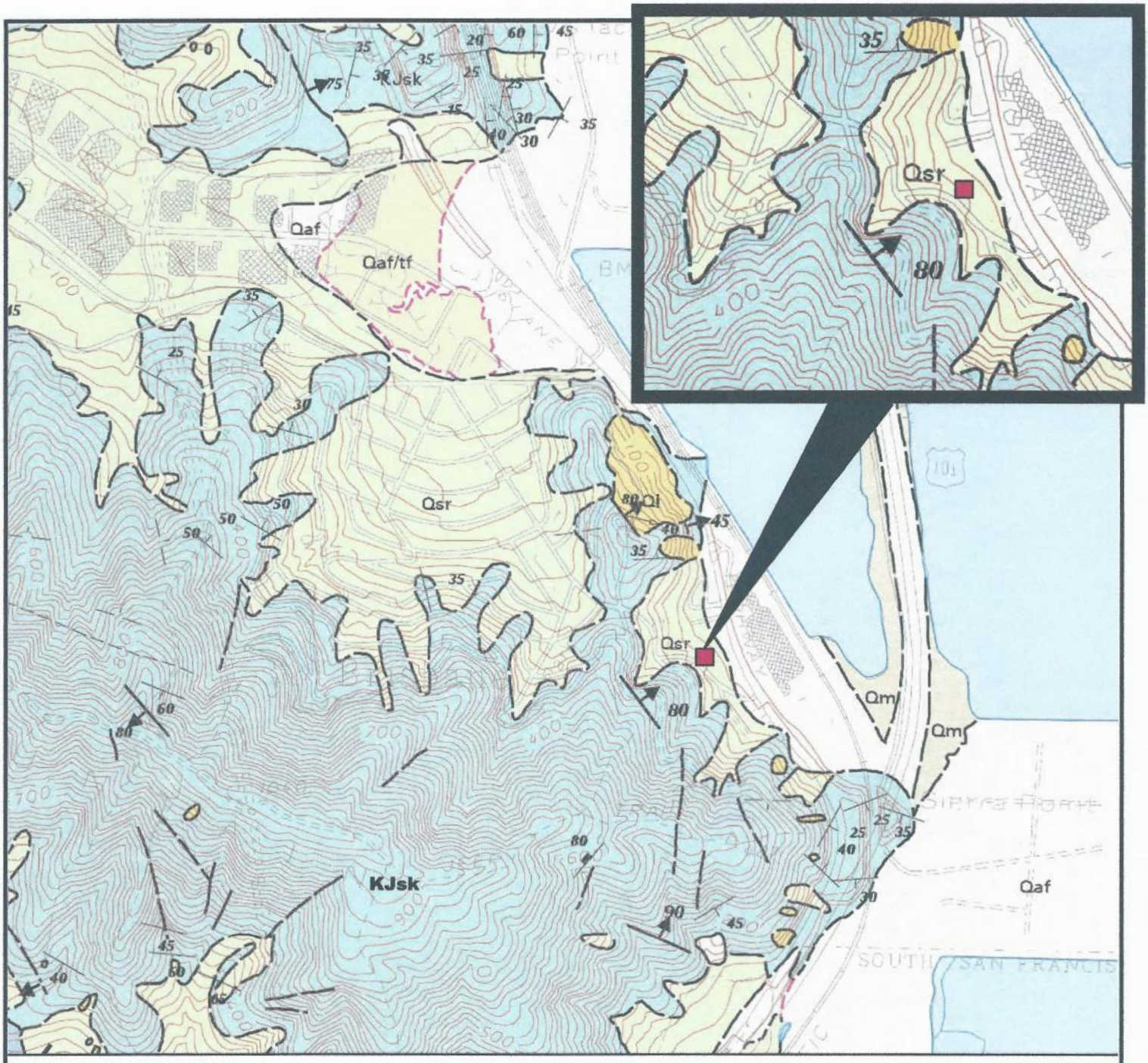
California Department of Conservation, 1982, State of California Special Studies Zones, San Francisco South Quadrangle: California Division of Mines and Geology.

Jennings, C.W., and Bryant, W.A., 2010, Fault activity map of California: California Geological

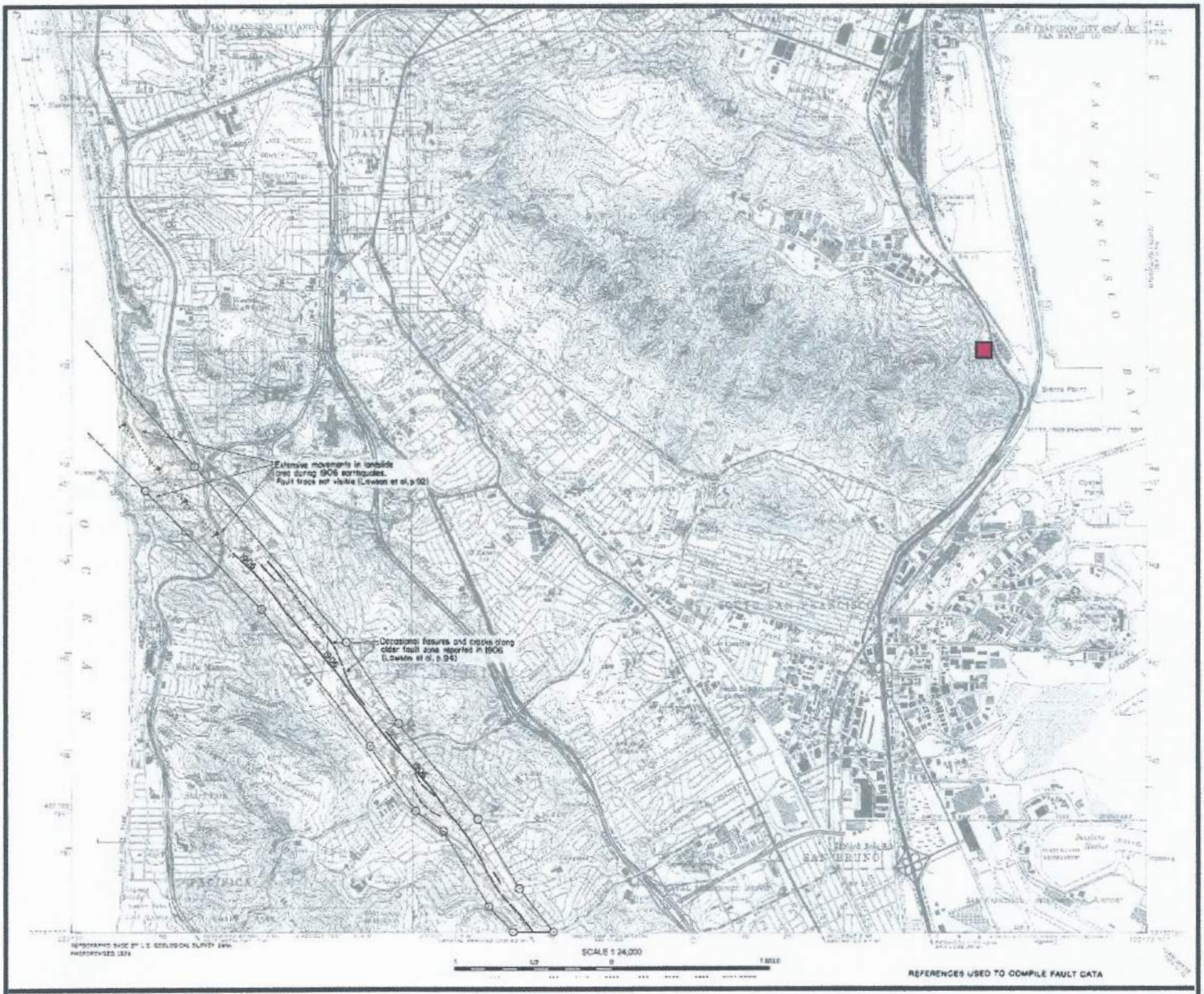


LEGEND: Site location — Contour

PROJECT ADDRESS: 3848 San Bruno Avenue, Brisbane, CA 94005	**DATE:** September 2017	**FIGURE 1**
BASE MAP: USGS Topographic Map, San Francisco South Quadrangle, 7.5-Minute Series, 2015. Scale = 1:24000. Contour Interval = 20 feet. Site elevation approximately 65.9 feet.		**TOPOGRAPHIC MAP**

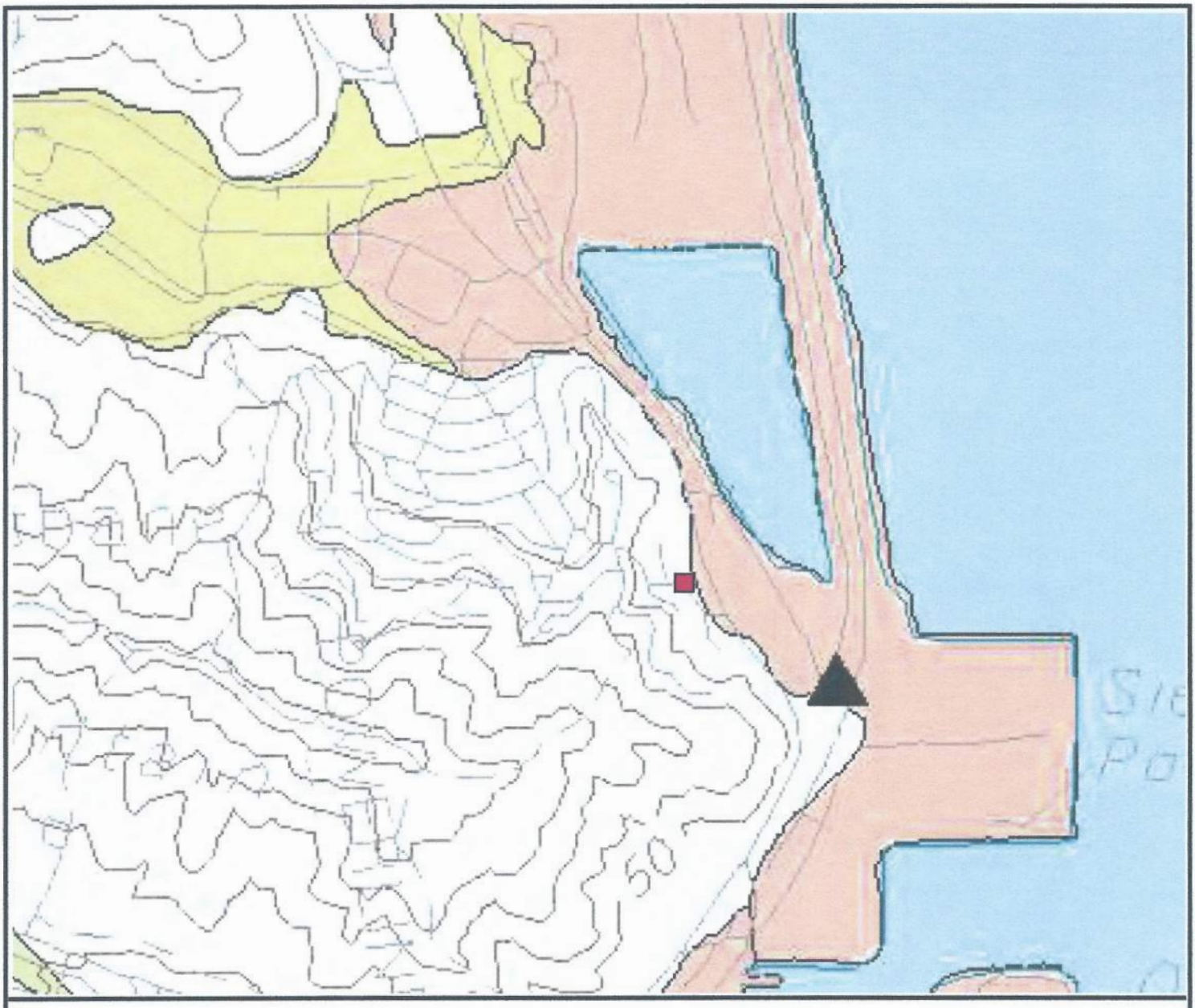


PROJECT ADDRESS: 3848 San Bruno Avenue, Brisbane, CA 94005	**DATE:** September 2017	**FIGURE 2**
BASE MAP: Bonilla, M.G., 1998, Preliminary Geologic Map of the San Francisco South 7.5' Quadrangle and Part of the Hunter's Point 7.5' Quadrangle, San Francisco Bay Area, California: USGS, U.S. Department of the Interior, Open-File Report 98-354.		**GEOLOGIC MAP**



LEGEND: Alquist-Priolo Earthquake Zone; Site location

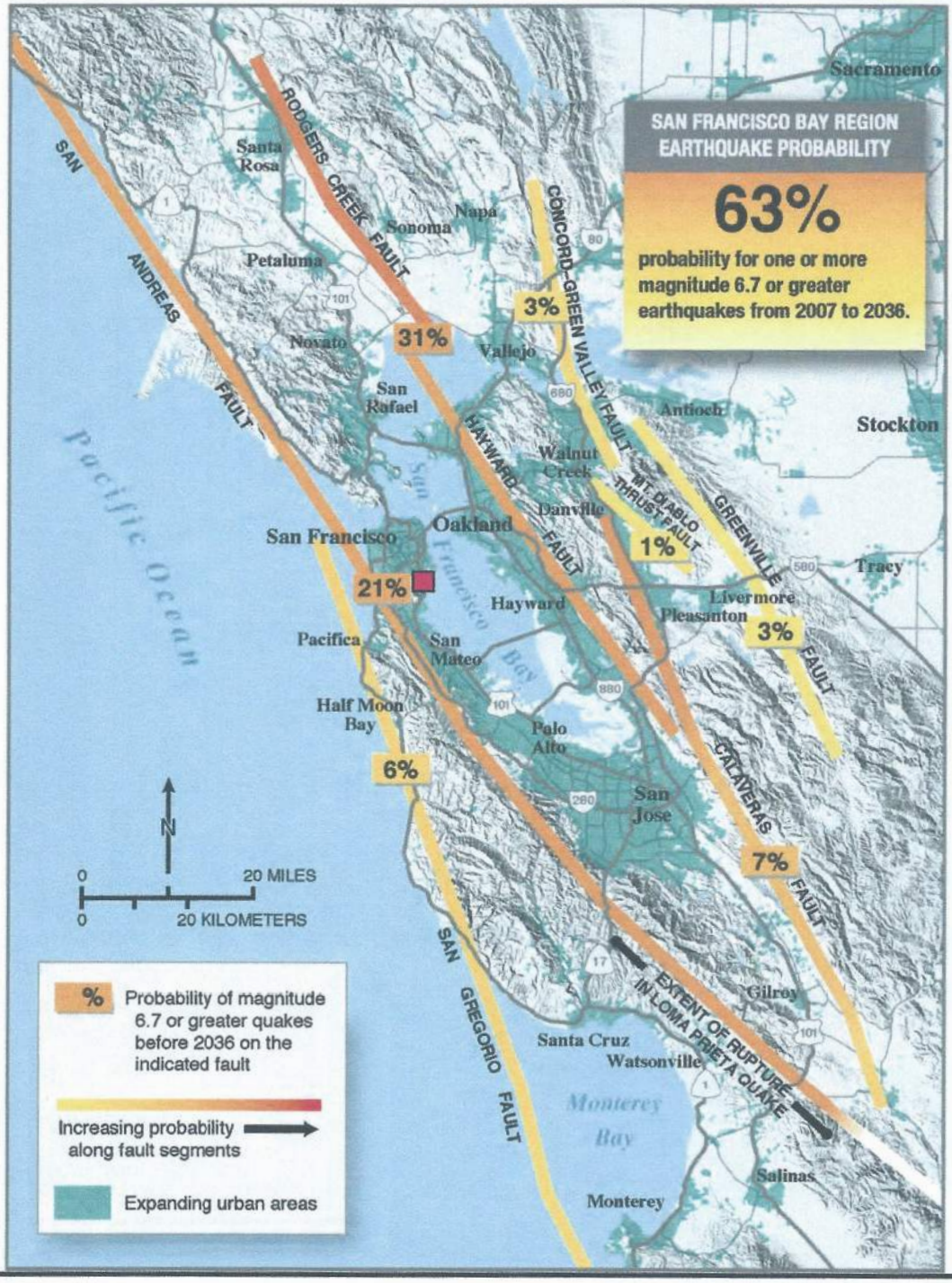
PROJECT ADDRESS: 3848 San Bruno Avenue, Brisbane, CA 94005	**DATE:** September 2017	**FIGURE 3**
BASE MAP: California Department of Conservation, 1982, State of California Special Studies Zones, San Francisco South Quadrangle: California Division of Mines and Geology.		**ALQUIST-PRIOLO MAP**



LEGEND:

- Very low
- Moderate
- Very high
- Site location
- Water
- Historical Liquefaction Event - Ground Settlement

PROJECT ADDRESS: 3848 San Bruno Avenue, Brisbane, CA 94005	**DATE:** September 2017	**FIGURE 4**
BASE MAP: Knudsen, L., et al., 1997, Quaternary Geology and Liquefaction Susceptibility Maps, San Francisco, CA, Open-File Report 97-715: USGS, Department of the Interior.		**LIQUEFACTION SUSCEPTIBILITY MAP**



LEGEND: Site location

PROJECT ADDRESS: 3848 San Bruno Avenue, Brisbane, CA 94005	**DATE:** September 2017	**FIGURE 1**
BASE MAP: The Uniform California Earthquake Rupture Forecast, Version 2 (UCERF 2): U.S. Geological Survey Open-File Report 2007-1437; California Geological Survey Special Report 203214. By: the Working Group on California Earthquake Probabilities (2007), 2008.		**EARTHQUAKE PROBABILITY MAP**

DATE DRILLED	LOGGED/CHECKED BY:	APPROX. SURFACE ELEVATION (FT)
9/6/2017	Igor Kleyner	65′
DRILLING METHOD	**DRILLED BIT SIZE**	**TOTAL DEPTH OF BOREHOLE (FT)**
Power Auger	3″ and 2″	9′
GROUNDWATER LEVEL/DATE MEASURED	**SAMPLING METHOD(S)**	**HAMMER DATA**
No groundwater detected	Disturbed	Dynamic Cone Penetration Test
BOREHOLE BACKFILL	**LOCATION**	
Native soil/cement	3848 San Bruno Avenue, Brisbane, CA 94005	

DEPTH (FT)	SAMPLE TYPE	SAMPLING RESISTANCE (BLOWS/FOOT)	RELATIVE HARDNESS/ CONSISTENCY	SOIL/ROCK TYPE	MOISTURE CONTENT %	MATERIAL DESCRIPTION
1–2		DCPT 45+ SPT 40	Very dense	SM	6.2	6″ topsoil with organics Very dense, well-graded, brown fine silty sand with non-plastic fines and bedrock debris
3–4		DCPT 45+ SPT 40	Very dense	SM	5.7	Very dense, well-graded, brown fine silty sand with low plastic fines and bedrock debris
5–8						Practical refusal met @ 5′6″ Free groundwater not detected
9–16		SPT 50+	Hard	KJsk	–	Sandstone bedrock Excavated with rotary hammer 1.5″ bit

PROJECT ADDRESS:	3848 San Bruno Avenue, Brisbane, CA 94005	DATE: September 2017	FIGURE 6	LOG OF BORING BOREHOLE-1

DATE DRILLED	LOGGED/CHECKED BY:	APPROX. SURFACE ELEVATION (FT)
9/6/2017	Igor Kleyner	65′
DRILLING METHOD	**DRILLED BIT SIZE**	**TOTAL DEPTH OF BOREHOLE (FT)**
Power Auger	3″ and 2″	5′6″
GROUNDWATER LEVEL/DATE MEASURED	**SAMPLING METHOD(S)**	**HAMMER DATA**
No groundwater detected	Disturbed	Dynamic Cone Penetration Test
BOREHOLE BACKFILL	**LOCATION**	
Native soil/cement	3848 San Bruno Avenue, Brisbane, CA 94005	

DEPTH (FT)	SAMPLE TYPE	SAMPLING RESISTANCE (BLOWS/FOOT)	RELATIVE HARDNESS/ CONSISTENCY	SOIL/ROCK TYPE	MOISTURE CONTENT %	MATERIAL DESCRIPTION
1–2		DCPT 40+ SPT 35	Very dense	SM	-	6″ topsoil with organics Very dense, well-graded, brown fine silty sand with non-plastic fines and bedrock debris
3–4		DCPT 50 SPT 40+	Very dense	SM	-	Very dense, well-graded, brown fine silty sand with low plastic fines and bedrock debris
5						Practical refusal met @ 4′ Free groundwater not detected
6		SPT 50+	Hard	KJsk	-	Sandstone bedrock
7–16						

PROJECT ADDRESS:	3848 San Bruno Avenue, Brisbane, CA 94005	DATE: September 2017	FIGURE 7	LOG OF BORING BOREHOLE-2

DATE DRILLED	LOGGED/CHECKED BY:	APPROX. SURFACE ELEVATION (FT)
9/6/2017	Igor Kleyner	65′
DRILLING METHOD	**DRILLED BIT SIZE**	**TOTAL DEPTH OF BOREHOLE (FT)**
Power Auger	3″ and 2″	4′6″
GROUNDWATER LEVEL/DATE MEASURED	**SAMPLING METHOD(S)**	**HAMMER DATA**
No groundwater detected	Disturbed	Dynamic Cone Penetration Test
BOREHOLE BACKFILL	**LOCATION**	
Native soil/cement	3848 San Bruno Avenue, Brisbane, CA 94005	

DEPTH (FT)	SAMPLE TYPE	SAMPLING RESISTANCE (BLOWS/FOOT)	RELATIVE HARDNESS/ CONSISTENCY	SOIL/ROCK TYPE	MOISTURE CONTENT %	MATERIAL DESCRIPTION
1–2		DCPT 35+ SPT 30	Dense	SM	3.8	6″ topsoil with organics Dense, well-graded, brown silty sand with low plastic fines and bedrock debris
3–4		DCPT 40 SPT 45	Very dense	SM	-	Very dense, well-graded, brown silty sand with low plastic fines and sandstone debris
5						Practical refusal met @ 4′ Free groundwater not detected
6–16		SPT 50+	Hard	KJsk	-	Sandstone bedrock

PROJECT ADDRESS:	3848 San Bruno Avenue, Brisbane, CA 94005	DATE: September 2017	FIGURE 8	LOG OF BORING BOREHOLE-3

DATE DRILLED	LOGGED/CHECKED BY:	APPROX. SURFACE ELEVATION (FT)
9/6/2017	Igor Kleyner	65′
DRILLING METHOD	**DRILLED BIT SIZE**	**TOTAL DEPTH OF BOREHOLE (FT)**
Power Auger	3″ and 2″	4′
GROUNDWATER LEVEL/DATE MEASURED	**SAMPLING METHOD(S)**	**HAMMER DATA**
No groundwater detected	Disturbed	Dynamic Cone Penetration Test
BOREHOLE BACKFILL	**LOCATION**	
Native soil/cement	3848 San Bruno Avenue, Brisbane, CA 94005	

DEPTH (FT)	SAMPLE TYPE	SAMPLING RESISTANCE (BLOWS/FOOT)	RELATIVE HARDNESS/ CONSISTENCY	SOIL/ROCK TYPE	MOISTURE CONTENT %	MATERIAL DESCRIPTION
1		DCPT 40	Very dense	SM	3.8	6″ topsoil with organics Very dense, well-graded, brown silty sand with low plastic fines and bedrock debris
2						
3		DCPT 40 SPT 45	Very dense	-	-	Very dense, well-graded, brown silty sand with low plastic fines and sandstone debris
4						
5				KJsk		Practical refusal met @ 4′ Free groundwater not detected Weathered sandstone bedrock
6						
7						
8						
9						
10						
11						
12						
13						
14						
15						
16						

PROJECT ADDRESS:	3848 San Bruno Avenue, Brisbane, CA 94005	DATE: September 2017	FIGURE 9	LOG OF BORING BOREHOLE-4

DATE DRILLED	LOGGED/CHECKED BY:	APPROX. SURFACE ELEVATION (FT)
9/6/2017	Igor Kleyner	65′
DRILLING METHOD	**DRILLED BIT SIZE**	**TOTAL DEPTH OF BOREHOLE (FT)**
Power Auger	3″ and 2″	4′6″
GROUNDWATER LEVEL/DATE MEASURED	**SAMPLING METHOD(S)**	**HAMMER DATA**
No groundwater detected	Disturbed	Dynamic Cone Penetration Test
BOREHOLE BACKFILL	**LOCATION**	
Native soil/cement	3848 San Bruno Avenue, Brisbane, CA 94005	

DEPTH (FT)	SAMPLE TYPE	SAMPLING RESISTANCE (BLOWS/FOOT)	RELATIVE HARDNESS/ CONSISTENCY	SOIL/ROCK TYPE	MOISTURE CONTENT %	MATERIAL DESCRIPTION
1–2		DCPT 25 SPT 20	Dense	SM	-	8″ topsoil with organics Dense, well-graded, brown silty sand with low plastic fines and bedrock debris
3–4		DCPT 30 SPT 25	Dense to very dense	SM	-	Dense to very dense, well-graded, brown silty sand with low plastic fines and sandstone debris
5–6				KJsk		Practical refusal met @ 4′ Free groundwater not detected Sandstone bedrock

PROJECT ADDRESS:	3848 San Bruno Avenue, Brisbane, CA 94005	DATE: September 2017	FIGURE 10	LOG OF BORING BOREHOLE-5

DATE DRILLED	LOGGED/CHECKED BY:	APPROX. SURFACE ELEVATION (FT)
9/6/2017	Igor Kleyner	65′
DRILLING METHOD	**DRILLED BIT SIZE**	**TOTAL DEPTH OF BOREHOLE (FT)**
Power Auger	3″ and 2″	3′6″
GROUNDWATER LEVEL/DATE MEASURED	**SAMPLING METHOD(S)**	**HAMMER DATA**
No groundwater detected	Disturbed	Dynamic Cone Penetration Test
BOREHOLE BACKFILL	**LOCATION**	
Native soil/cement	3848 San Bruno Avenue, Brisbane, CA 94005	

DEPTH (FT)	SAMPLE TYPE	SAMPLING RESISTANCE (BLOWS/FOOT)	RELATIVE HARDNESS/ CONSISTENCY	SOIL/ROCK TYPE	MOISTURE CONTENT %	MATERIAL DESCRIPTION
1–2		DCPT 25 SPT 20	Dense	SM	-	8″ topsoil with organics Dense, well-graded, brown silty sand with low plastic fines and bedrock debris
3–4		DCPT 30 SPT 25	Dense to very dense	SM	-	Dense to very dense, well-graded, brown silty sand with low plastic fines and sandstone debris
5–6				KJsk		Practical refusal met @ 3′6″ Free groundwater not detected Sandstone bedrock
7–16						

PROJECT ADDRESS:	3848 San Bruno Avenue, Brisbane, CA 94005	DATE: September 2017	FIGURE 11	LOG OF BORING BOREHOLE-6

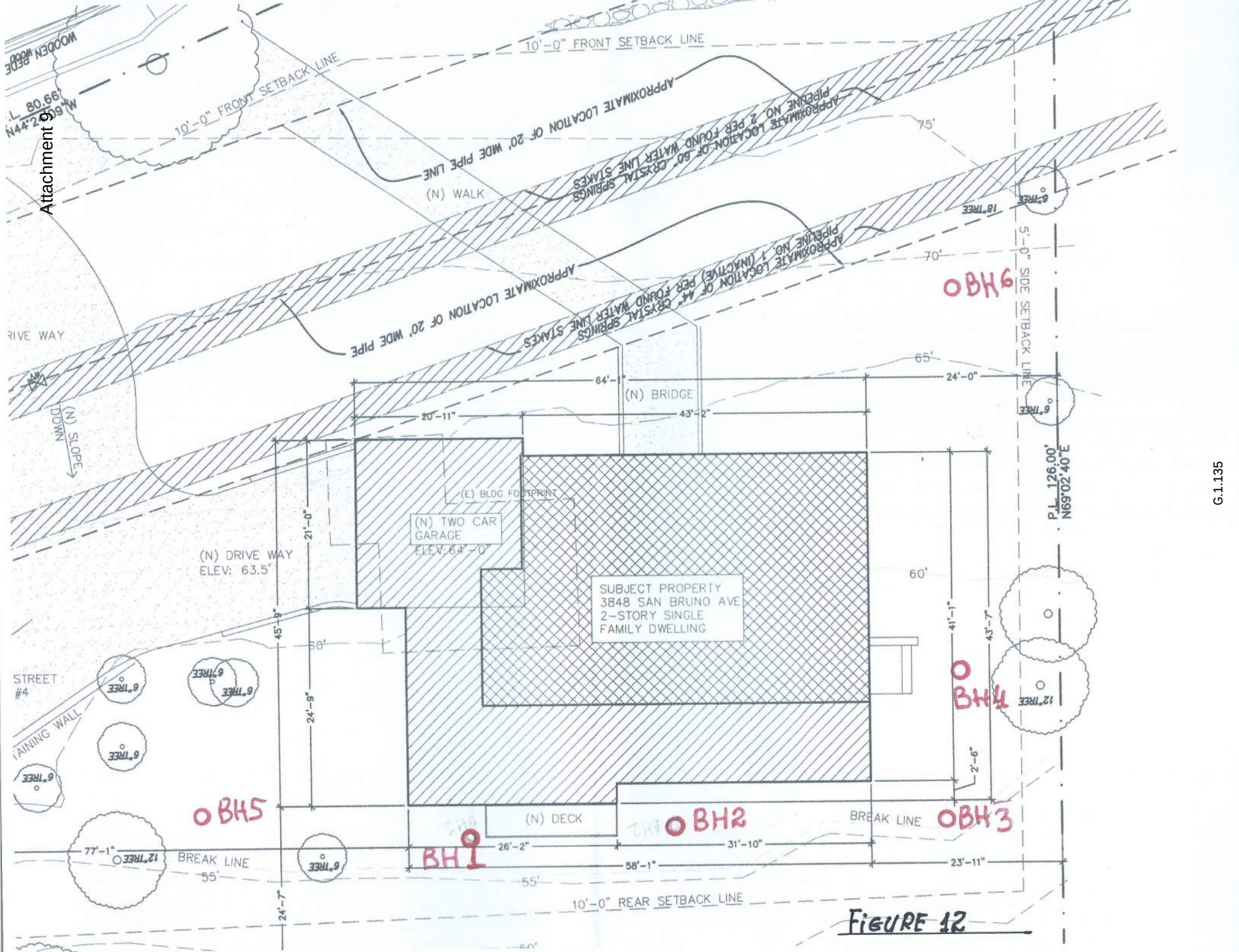

10'-0" FRONT SETBACK LINE
10'-0" FRONT SETBACK LINE
APPROXIMATE LOCATION OF 20' WIDE PIPE LINE
APPROXIMATE LOCATION OF 60" CRYSTAL SPRINGS PIPELINE NO. 2 PER FOUND WATER LINE STAKES
APPROXIMATE LOCATION OF 20' WIDE PIPE
APPROXIMATE LOCATION OF 44" CRYSTAL SPRINGS PIPELINE NO. 1 (INACTIVE) PER FOUND WATER LINE STAKES
(N) WALK
(N) BRIDGE
(N) SLOPE DOWN
(E) BLDG FOOTPRINT
(N) TWO CAR GARAGE
ELEV: 64'-0"
(N) DRIVE WAY
ELEV: 63.5'
SUBJECT PROPERTY
3848 SAN BRUNO AVE
2-STORY SINGLE
FAMILY DWELLING
(N) DECK
BREAK LINE
BREAK LINE
10'-0" REAR SETBACK LINE
5'-0" SIDE SETBACK LINE
P.L. 126.00'
N69°02'40"E
STREET #4
RIVE WAY
WOODEN PEDE
BH1
BH2
BH3
BH4
BH5
BH6
64'-1"
24'-0"
20'-11"
43'-2"
21'-0"
45'-9"
24'-9"
41'-1"
43'-7"
2'-6"
77'-1"
26'-2"
31'-10"
58'-1"
23'-11"
24'-7"
75'
70'
65'
60'
55'

FIGURE 12

APPENDIX A

U.S. Geological Survey - Earthquake Hazards Program

3848 SAN BRUNO AVE, BRISBANE, CA

Latitude = 37.678°N, Longitude = 122.395°W

Location



Reference Document

2015 NEHRP Provisions

Site Class

D (default): Stiff Soil

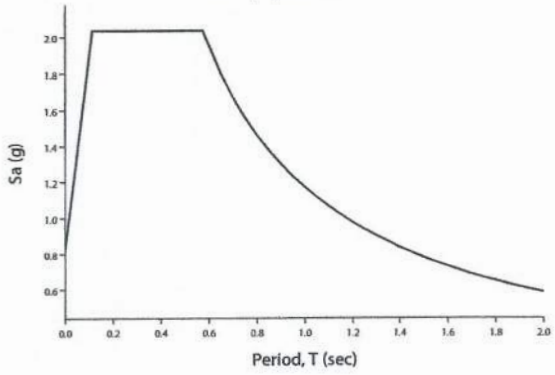
Risk Category

I or II or III

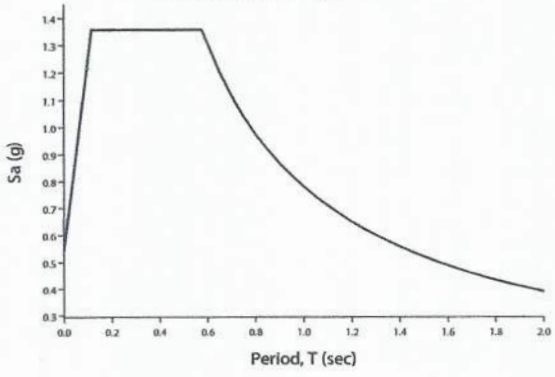
S_S =	1.695 g	S_{MS} =	2.035 g	S_{DS} =	1.356 g
S_1 =	0.689 g	S_{M1} =	1.172 g[1]	S_{D1} =	0.781 g[1]

[1] Since the Site Class is D and $S_1 \geq 0.2$ g, site-specific ground motions might be required. See Section 11.4.7 of the 2015 NEHRP Provisions.

MCE_R Spectrum



Design Response Spectrum



Mapped Acceleration Parameters, Long-Period Transition Periods, and Risk Coefficients

Note: The S_S and S_1 ground motion maps provided below are for the direction of maximmum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) 1.3 (to obtain S_1).

- FIGURE 22-1 S_S Risk-Targeted Maximum Considered Earthquake (MCE_R) Ground Motion Parameter for the Conterminous United States for 0.2 s Spectral Response Acceleration (5% of Critical Damping), Site Class B
- FIGURE 22-2 S_1 Risk-Targeted Maximum Considered Earthquake (MCE_R) Ground Motion Parameter for the Conterminous United States for 1.0 s Spectral Response Acceleration (5% of Critical Damping), Site Class B
- FIGURE 22-9 Maximum Considered Earthquake Geometric Mean (MCE_G) PGA, %g, Site Class B for the Conterminous United States
- FIGURE 22-14 Mapped Long-Period Transition Period, T_L (s), for the Conterminous United States
- FIGURE 22-18 Mapped Risk Coefficient at 0.2 s Spectral Response Period, C_{RS}
- FIGURE 22-19 Mapped Risk Coefficient at 1.0 s Spectral Response Period, C_{R1}

Site Class

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

Table 20.3-1 Site Classification

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

For SI: 1ft/s = 0.3048 m/s 1lb/ft^2 = 0.0479 kN/m^2

Site Coefficients and Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Risk-targeted Ground Motion (0.2 s)

$C_{RS}S_{SUH} = 0.903 \times 2.282 = 2.061$ g

Deterministic Ground Motion (0.2 s)

$S_{SD} = 1.695$ g

$S_S \equiv$ "Lesser of $C_{RS}S_{SUH}$ and S_{SD}" $= 1.695$ g

Risk-targeted Ground Motion (1.0 s)

$C_{R1}S_{1UH} = 0.888 \times 0.949 = 0.843$ g

Deterministic Ground Motion (1.0 s)

$S_{1D} = 0.689$ g

$S_1 \equiv$ "Lesser of $C_{R1}S_{1UH}$ and S_{1D}" $= 0.689$ g

Table 11.4-1: Site Coefficient F_a

	Spectral Reponse Acceleration Parameter at Short Period					
Site Class	$S_S \leq 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S = 1.25$	$S_S \geq 1.50$
A	0.8	0.8	0.8	0.8	0.8	0.8
B (measured)	0.9	0.9	0.9	0.9	0.9	0.9
B (unmeasured)	1.0	1.0	1.0	1.0	1.0	1.0
C	1.3	1.3	1.2	1.2	1.2	1.2
D (determined)	1.6	1.4	1.2	1.1	1.0	1.0
D (default)	1.6	1.4	1.2	1.2	1.2	1.2
E	2.4	1.7	1.3	1.2*	1.2*	1.2*
F	See Section 11.4.7					

* For Site Class E and $S_S \geq 1.0$ g, see the requirements for site-specific ground motions in Section 11.4.7 of the 2015 NEHRP Provisions. Here the exception to those requirements allowing F_a to be taken as equal to that of Site Class C has been invoked.

Note: Use straight-line interpolation for intermediate values of S_S.

Note: Where Site Class B is selected, but site-specific velocity measurements are not made, the value of F_a shall be taken as 1.0 per Section 11.4.2.

Note: Where Site Class D is selected as the default site class per Section 11.4.2, the value of F_a shall not be less than 1.2 per Section 11.4.3.

For Site Class = D (default) and S_S = 1.695 g, F_a = 1.200

Table 11.4-2: Site Coefficient F_v

Site Class	Spectral Response Acceleration Parameter at 1-Second Period					
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 = 0.50$	$S_1 \geq 0.60$
A	0.8	0.8	0.8	0.8	0.8	0.8
B (measured)	0.8	0.8	0.8	0.8	0.8	0.8
B (unmeasured)	1.0	1.0	1.0	1.0	1.0	1.0
C	1.5	1.5	1.5	1.5	1.5	1.4
D (determined)	2.4	2.2 [1]	2.0 [1]	1.9 [1]	1.8 [1]	1.7 [1]
D (default)	2.4	2.2 [1]	2.0 [1]	1.9 [1]	1.8 [1]	1.7 [1]
E	4.2	3.3 [1]	2.8 [1]	2.4 [1]	2.2 [1]	2.0 [1]
F	See Section 11.4.7					

[1] For Site Class D or E and $S_1 \geq 0.2$ g, site-specific ground motions might be required. See Section 11.4.7 of the 2015 NEHRP Provisions.

Note: Use straight-line interpolation for intermediate values of S_1.

Note: Where Site Class B is selected, but site-specific velocity measurements are not made, the value of F_v shall be taken as 1.0 per Section 11.4.2.

For Site Class = D (default) and $S_1 = 0.689$ g, $F_v = 1.700$

Site-adjusted MCE_R (0.2 s)

$$S_{MS} = F_a S_S = 1.200 \times 1.695 = 2.035 \text{ g}$$

Site-adjusted MCE_R (1.0 s)

$$S_{M1} = F_v S_1 = 1.700 \times 0.689 = 1.172 \text{ g}$$

Design Spectral Acceleration Parameters

Design Ground Motion (0.2 s)

$$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3} \times 2.035 = 1.356 \text{ g}$$

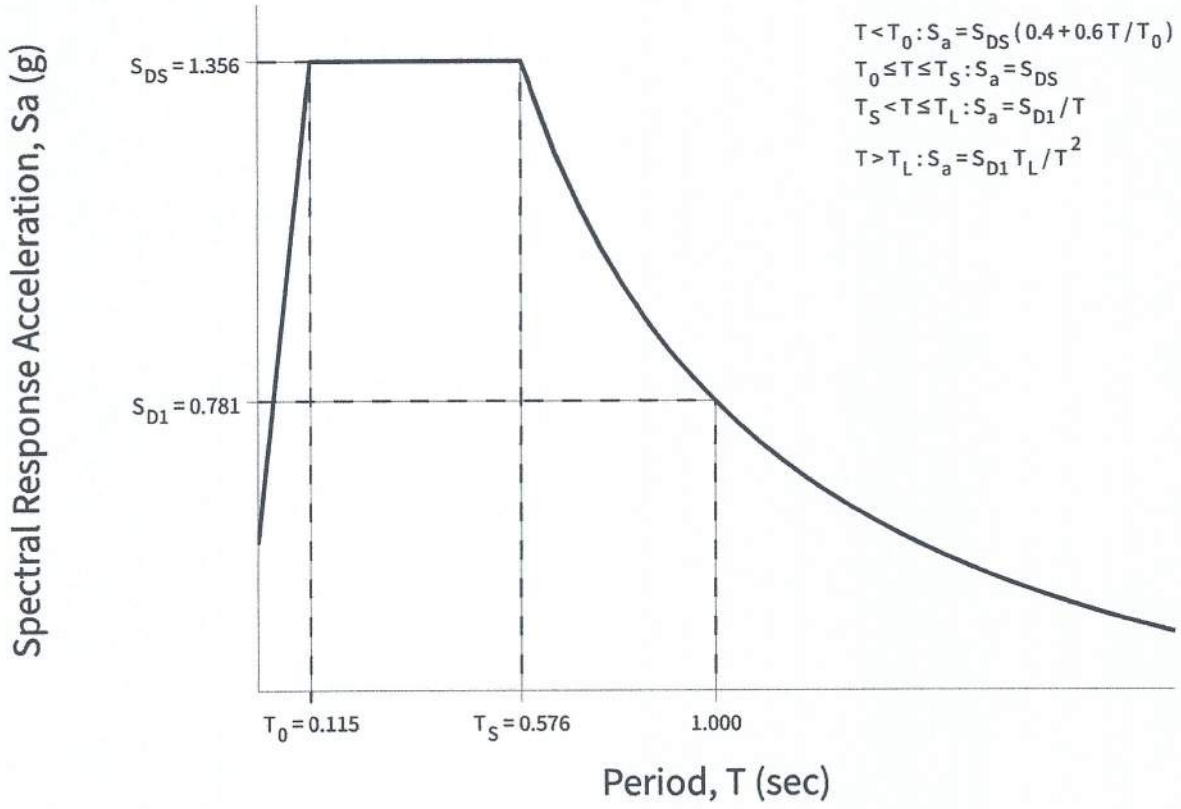
Design Ground Motion (1.0 s)

$$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3} \times 1.172 = 0.781 \text{ g}$$

Design Response Spectrum

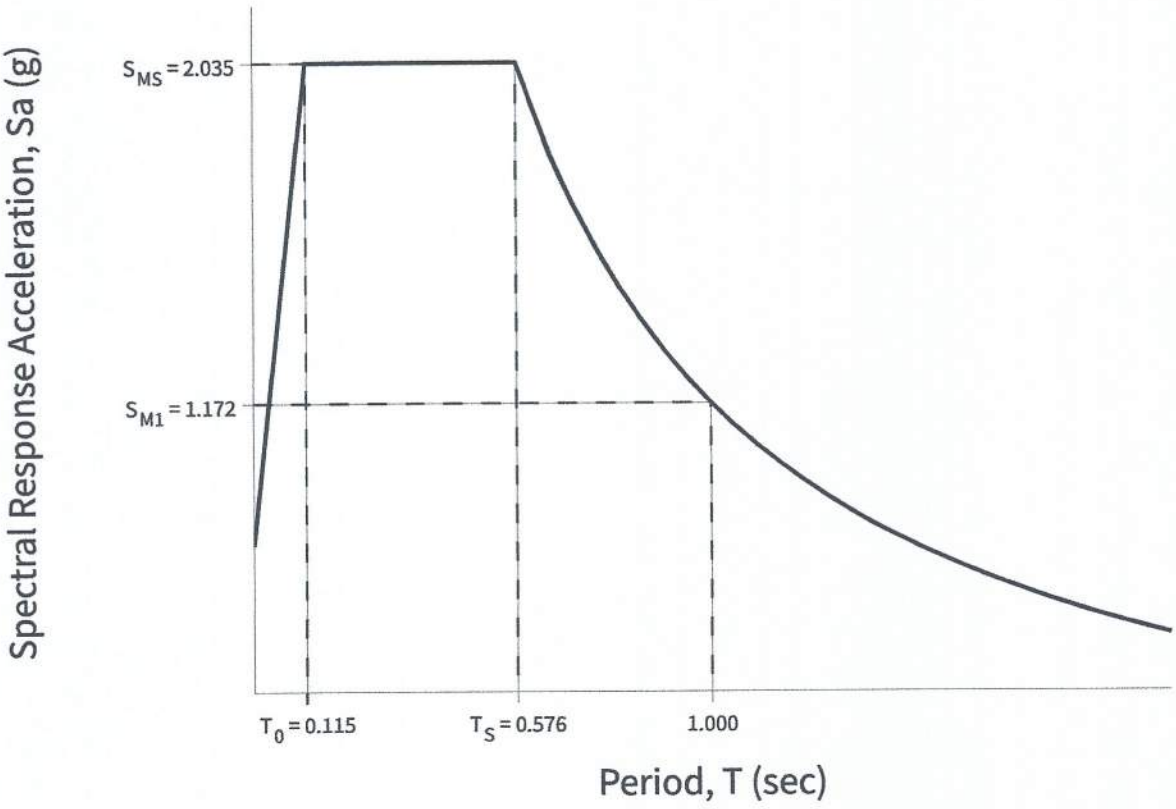
Long-Period Transition Period = T_L = 12 s

Figure 11.4-1: Design Response Spectrum



MCE_R Response Spectrum

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



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

Table 11.8-1: Site Coefficient for F_{PGA}

Site Class	Mapped MCE Geometric Mean (MCE_G) Peak Ground Acceleration					
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA = 0.50	PGA ≥ 0.60
A	0.8	0.8	0.8	0.8	0.8	0.8
B (measured)	0.9	0.9	0.9	0.9	0.9	0.9
B (unmeasured)	1.0	1.0	1.0	1.0	1.0	1.0
C	1.3	1.2	1.2	1.2	1.2	1.2
D (determined)	1.6	1.4	1.3	1.2	1.1	1.1
D (default)	1.6	1.4	1.3	1.2	1.2	1.2
E	2.4	1.9	1.6	1.4	1.2	1.1
F	See Section 11.4.7					

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

Note: Where Site Class D is selected as the default site class per Section 11.4.2, the value of F_{pga} shall not be less than 1.2.

For Site Class = D (default) and PGA = 0.732 g, $F_{PGA} = 1.200$

Mapped MCE_G

PGA = 0.732 g

Site-adjusted MCE_G

$PGA_M = F_{PGA}PGA = 1.200 \times 0.732 = 0.878$ g

WEATHERING

FRESH
ROCK FRESH, CRYSTALS BRIGHT, FEW JOINTS MAY SHOW SLIGHT STAINING. ROCK RINGS UNDER HAMMER IF CRYSTALLINE.

VERY SLIGHT
ROCK GENERALLY FRESH, JOINTS STAINED, SOME JOINTS MAY SHOW THIN CLAY COATINGS, CRYSTALS IN BROKEN FACE SHOW BRIGHT. ROCK RINGS UNDER HAMMER IF CRYSTALLINE.

SLIGHT
ROCK GENERALLY FRESH, JOINTS STAINED, AND DISCOLORATION EXTENDS INTO ROCK UP TO 1 INCH. JOINTS MAY CONTAIN CLAY. IN GRANITOID ROCKS SOME OCCASIONAL FELDSPAR CRYSTALS ARE DULL AND DISCOLORED. CRYSTALLINE ROCKS RING UNDER HAMMER.

MODERATE
SIGNIFICANT PORTIONS OF ROCK SHOW DISCOLORATION AND WEATHERING EFFECTS. IN GRANITOID ROCKS, MOST FELDSPARS ARE DULL AND DISCOLORED; SOME ARE CLAYEY. ROCK HAS DULL SOUND UNDER HAMMER AND SHOWS SIGNIFICANT LOSS OF STRENGTH AS COMPARED WITH FRESH ROCK.

MODERATELY SEVERE
ALL ROCK EXCEPT QUARTZ DISCOLORED OR STAINED. IN GRAINED ROCKS, ALL FELDSPARS DULL AND DISCOLORED AND MAJORITY SHOW KAOLINIZATION. ROCK SHOWS SEVERE LOSS OF STRENGTH AND CAN BE EXCAVATED WITH GEOLOGIST'S PICK. ROCK GOES "CLUNK" WHEN STRUCK.

SEVERE
ALL ROCK EXCEPT QUARTZ DISCOLORED OR STAINED. ROCK "FABRIC" CLEAR AND EVIDENT BUT REDUCED IN STRENGTH TO STRONG SOIL. IN GRANITOID ROCKS, ALL FELDSPARS KAOLINIZED TO SOME EXTENT. SOME FRAGMENTS OF STRONG ROCK USUALLY LEFT.

VERY SEVERE
ALL ROCK EXCEPT QUARTS DISCOLORED AND STAINED. ROCK "FABRIC" DISCERNIBLE, BUT MASS EFFECTIVELY REDUCED TO "SOIL" WITH ONLY FRAGMENTS OF STRONG ROCK REMAINING.

COMPLETE
ROCK REDUCED TO "SOIL". ROCK FABRIC NOT DISCERNIBLE OR DISCERNIBLE ONLY IN SMALL SCATTERED LOCATIONS. QUARTZ MAY BE PRESENT AS DIKES OR STRINGERS.

HARDNESS

VERY HARD
CANNOT BE SCRATCHED WITH KNIFE OR SHARP PICK. HAND SPECIMENS REQUIRES SEVERAL HARD BLOWS OF GEOLOGIST'S HAMMER.

HARD
CAN BE SCRATCHED WITH KNIFE OR PICK ONLY WITH DIFFICULTY. HARD BLOW OF HAMMER REQUIRED TO DETACH HAND SPECIMEN.

MODERATELY HARD
CAN BE SCRATCHED WITH KNIFE OR PICK. GOUGES OR GROOVES TO 1/4 INCH DEEP CAN BE EXCAVATED BY HARD BLOW OF POINT OF A GEOLOGIST'S PICK. HARD SPECIMEN CAN BE DETACHED BY MODERATE BLOW.

MEDIUM
CAN BE GROOVED OR GOUGED 1/16 INCH DEEP BY FIRM PRESSURE ON KNIFE OR PICK POINT. CAN BE EXCAVATED IN SMALL CHIPS TO PIECES ABOUT 1 INCH MAXIMUM SIZE BY HARD BLOWS OF THE POINT OF A GEOLOGIST'S PICK.

SOFT
CAN BE GOUGED OR GROOVED READILY WITH KNIFE OR PICK POINT. CAN BE EXCAVATED IN CHIPS TO PIECES SEVERAL INCHES IN SIZE BY MODERATE BLOWS OF A PICK POINT. SMALL THINK PIECES CAN BE BROKEN BY FINGER PRESSURE.

VERY SOFT
CAN BE CARVED WITH KNIFE. CAN BE EXCAVATED READILY WITH POINT OF PICK. PIECES 1 INCH OR MORE IN THICKNESS CAN BE BROKEN WITH FINGER PRESSURE. CAN BE SCRATCHED READILY BY FINGERNAIL.

JOINT BEDDING AND FOLIATION SPACING

SPACING	JOINTS	BEDDING & FOLIATION
LESS THAN 2 IN.	VERY CLOSE	VERY THIN
2 IN. TO 1 FT.	CLOSE	THIN
1 FT. TO 3 FT.	MODERATELY CLOSE	MEDIUM
3 FT. TO 10 FT.	WIDE	THICK
MORE THAN 10 FT.	VERY WIDE	VERY THICK

ROCK QUALITY DESIGNATOR (RQD)

RQD, AS A PERCENTAGE	DESCRIPTOR
EXCEEDING 90	EXCELLENT
90 TO 75	GOOD
75 TO 50	FAIR
50 TO 25	POOR
LESS THAN 25	VERY POOR

PROJECT ADDRESS: 3848 San Bruno Avenue, Brisbane, CA 94005	**DATE:** September 2017	**APPENDIX B**	**KEY TO BEDROCK DESCRIPTIONS**

PRIMARY DIVISIONS			SOIL TYPE	SECONDARY DIVISIONS
COARSE GRAINED SOILS (<50% FINES)	GRAVEL	CLEAN GRAVEL (<5% FINES)	GW	Well graded gravel, gravel-sand mixtures, little or no fines.
			GP	Poorly graded gravel or gravel-sand mixtures, little or no fines.
		GRAVEL W/ FINES	GM	Silty gravels, gravel-sand silt mixtures, non-plastic fines.
			GC	Clayey gravels, gravel-sand clay mixturesm plastic fines.
	SAND	CLEAN SAND (<5% FINES)	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		SAND W/ FINES	SM	Silty sands, sand-silt mixtures, non-plastic fines.
			SC	Clayey sands, sand-clay mixtures, plastic fines.
FINE GRAINED SOILS (>50% FINES)	SILT AND CLAY LIQUID LIMIT < 50%		ML	Inorganic silts and very fine sands, with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, lean clays.
			OL	Organic silts and organic clays of low plasticity.
	SILT AND CLAY LIQUID LIMIT > 50%		MH	Inorganic Silt, micaceous or diatomaceous fine sandy or silty soil.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils.

RELATIVE DENSITY

SAND & GRAVEL	BLOWS/FOOT*
VERY LOOSE	0 TO 4
LOOSE	4 TO 10
MEDIUM DENSE	10 TO 30
DENSE	30 TO 50
VERY DENSE	OVER 50

CONSISTENCY

SILT & CLAY	STRENGTH^	BLOWS/FOOT*
VERY SOFT	0 TO 0.25	0 TO 2
SOFT	0.25 TO 0.5	2 TO 4
FIRM	0.5 TO 1	4 TO 8
STIFF	1 TO 2	8 TO 16
VERY STIFF	2 TO 4	16 TO 32
HARD	OVER 4	OVER 32

GRAIN SIZES

BOULDERS	COBBLES	GRAVEL		SAND			SILT & CLAY
		COARSE	FINE	COARSE	MEDIUM	FINE	
	12"	3"	3/4"	4	10	40	200
	SIEVE OPENINGS			U.S. STANDARD SERIES SIEVE			

CLASSIFICATION IS BASED ON THE UNIFIED SOIL CLASSIFICATION SYSTEM; FINES REFER TO SOIL PASSING A NO. 200 SEIVE.

*STANDARD PENETRATION TEST (SPT) RESISTANCE, USING A 140 POUND HAMMER FALLING 30 INCHES ON A 2 INCH OD SPLIT SPOON SAMPLER; BLOW COUNTS FOR COARSE-GRAINED SOILS HAVE BEEN STANDARDIZED TO SPT COUNTS BY FACTORS OF 0.8 AND0.7 FOR THE 2.5-INCH OD AND 3.0-INCH OD SAMPLERS, RESPECTIVELY.

^SHEAR STRENGTH IN TONS/SQ. FT. AS ESTIMATED BY SPT RESISTANCE, FIELD AND LABORATORY TESTS, AND/OR VISUAL OBSERVATION.

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