APPENDIX C
Geotechnical Investigation

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GEOTECHNICAL INVESTIGATION REPORT
MULT-FAMILY CONDOMINIUM COMPLEX
1122 N. BEWLEY STREET, CITY OF SANTA ANA, CA 92703
(APN 198-101-07)

Prepared for:

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ZS Engineering #200101

January 29, 2020

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Mai-Lan Thi Nguyen
946 S. Emerald Street
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Subject: **Geotechnical Investigation Report**
Multi-Family Condominium Complex
1122 N. Bewley Street, City of Santa Ana, CA 92703
(APN 198-101-07)

Dear Ms. Nguyen:

Pursuant to our proposal, dated January 7, 2020, which was authorized by you, ZS Engineering has prepared this geotechnical investigation report for the proposed condominium complex of four (4) 2-story buildings and associated site work at the subject residential lot, located at 1122 N. Bewley Street in the City of Santa Ana, California. Purposes of this investigation were to evaluate the subsurface geotechnical conditions of the site; to assess geological hazards, liquefaction potential and other seismic hazards for the project site; and to provide geotechnical design parameters, grading recommendations for design and construction of this condominium complex and associated site improvements.

The subject property is located within a developed area on t he south side of Westminster Avenue, on the east side of Harbor Blvd., within northwestern portion of the City of Santa Ana, California. A golf course and the Santa Ana River trail are located to the west of the site. Developments surrounding the site is mostly residential and some commercial. Topography within the site and its surroundings is fairly flat.

This property is a rectangular shaped parcel of 37,800 sq. feet (~0.87 acre) gross lot area with dimensions about 315 feet along the east-west and about 120 feet along the north-south direction. During this investigation, the site was found to be vacant of any structures. It was covered with grass, weeds, vegetation, multiple trees, and an entry driveway at the northeast corner, which remained from previous construction. Based on the project plans, prepared by YNG Architects, (see References), we understand that the site will be developed for a new multi-family condominium complex. It will have total four (4) above grade, 2-story buildings that will house total ten (10) residential units. These units will have 2 to 4 bedrooms. Each unit will have a private open space and an attached 2-car garage at the ground floor level, and a balcony off the second floor. Floor areas of these units will vary from 1,618 to 1,946 sq. feet. Size of private space and balcony at each unit will vary from 250 to 1,000 sq. feet and from 30 to 250 sq. feet, respectively. Garage at each unit will be over 400 sq. feet floor area. Footprint areas of new four (4) buildings will vary from 2,960 to 4,000 sq. feet. Besides buildings, site improvements will include driveways, parking lots, open spaces, trash enclosures, a picnic shelter, landscaping, perimeter walls, etc.

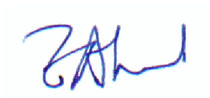
Subsurface soils at the site, as encountered within the drilled holes, consist of fill soils up to a depth about 3 feet, followed by native alluvial soils up to the maximum explored depth of 50 feet. A broad plain surrounding the site is covered with young alluvial fan deposits (Qyf) of Holocene to late Pleistocene age. Fill soils are silty, clayey fine sand to silty fine sand with little clay. Underneath the fill, alluvial soils up to a depth about 15 feet are primarily fine sand with few to little silts. Soils within depths from about 15 to 25 feet are silty, clayey soils with high fine contents, little to some fine sands. Soils below 25 feet and up to the maximum explored of 50 feet comprise of sand to silty sand with few to little clay, sand grains varying from fine to coarse. Fine contents (silt, clay) of sandy layers vary from about 8 to 36 percent while the same for the silty, clayey soils within depths from 15 to 25 feet vary within a range about 55 to 72 percent. Subsurface geologic profiles are found to be fairly consistent across the site.

Historic shallow groundwater level at the project site is within the contours of 5 and 10 feet below grade as shown in the state's seismic hazard zones report (CGS, 1997). Groundwater was encountered at depth varying from about 19 to 20 feet below the existing grade during this field exploration. Due to shallow historic groundwater, a broad plain surrounding this site is mapped within a liquefaction hazard zone as delineated in the state's seismic hazard zones map (CGS, 1998). Our liquefaction potential analysis results indicate a potentially liquefiable layer, about 2 feet thick, within depths from about 29 to 31 feet. Due to this relatively small zone of liquefiable soils and its depth below the existing grade, surface manifestation (such as sand boiling, ground fissure, etc.) causing loss of bearing capacity of the foundation subgrade soils is not anticipated in the event of a major earthquake. Moreover, our analysis considered concurrence of the historical shallow groundwater level during a major earthquake, which is very unlikely.

Proposed new 2-story condominium building structures can be founded on shallow spread footings (wall and/or column) with conventional slab-on-grade. Recommendations for site grading; geotechnical design parameters for building foundations, floor slab, garage floor, pavements, exterior flatwork; and other relevant design parameters, construction consideration for this project are presented in this report. Subsurface soils will provide adequate bearing, lateral resistance, friction and support for the proposed structures and site improvements provided that design and construction of this project adhere to the recommendations in this report. Based on our findings, there are no geotechnical constraints, geologic hazards at the subject site that would adversely impact this condominium buildings project.

We appreciate this opportunity of service. If there are any questions regarding this report, please contact our office.

Respectfully submitted,
ZS ENGINEERING



Zafar Ahmed, PE, GE
Geotechnical Engineer

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1.0 INTRODUCTION

1.1 Purpose and Scope

This report presents the findings, conclusions and recommendations from our geotechnical investigation for the proposed condominium complex of four (4) 2-story buildings and associated site work at the subject residential lot, located at 1122 N . Bewley Street in the City of Santa Ana, California. Purposes of this investigation were to evaluate the subsurface geotechnical conditions of the site; to assess geological hazards, liquefaction potential and other seismic hazards for the project site; and to provide geotechnical design parameters, grading recommendations for design and construction of this condominium complex and site improvements.

In preparation of this report, we conducted the following scope of work:

- Review of published and unpublished reports and maps pertinent to seismic hazards, local and regional geology within and adjacent to the site that could impact the proposed developments.
- Conduct subsurface exploration consisting of three (3) 8-inch diameter hollow stem auger (HSA) bore holes, depths varying from 25 to 50 feet below grade, within the project area. Drilling was done utilizing a truck mounted drilling rig. Subsurface geologic profiles were logged during field exploration and representative soil samples (bulk and ring) were taken at selected depths.
- Conduct necessary laboratory tests in order to characterize the subsurface soils and obtain geotechnical design parameters.
- Conduct geotechnical evaluations, liquefaction potential assessment and other engineering analyses from the collected data and the laboratory test results. Recommendations for grading; geotechnical design parameters for foundations, floor slabs, pavements, site work; and other relevant design parameters, construction and construction guidelines for the proposed condominium complex, as presented in this report, are determined from engineering evaluations and analyses.
- Preparation of this report summarizing our findings, conclusions, and recommendations.

1.2 Site Description and Proposed Developments

The subject property is located within a developed area on the south side of Westminster Avenue, on the east side of Harbor Blvd., within northwestern portion of the City of Santa Ana, California. A golf course and the Santa Ana River trail are located to the west of the site. Site location and its vicinities are shown in the attached Figure 1, *Site Location Map*. Developments surrounding the site is mostly residential and some commercial. Topography within the site and its surroundings is fairly flat. Existing elevations within the perimeters of the site vary from 79.6 to 81.2 feet (above mean sea level) from the west to the east side.

This property is a rectangular shaped parcel of 37,800 sq. feet (~0.87 acre) gross lot area with dimensions about 315 feet along the east-west and about 120 feet along the north-south direction. During this investigation, the site was found to be vacant of any structures. It was covered with grass, weeds, vegetation, multiple trees, and an entry driveway at the northeast corner, which remained from previous construction. Based on the project plans, prepared by YNG Architects, (see References), we understand that the site will be developed for a new multi-family condominium complex. It will have total four (4) above grade, 2-story buildings that will house total ten (10) residential units. These units will have 2 to 4 bedrooms. Each unit will have a private open space and an attached 2-car garage at the ground floor level, and a balcony off the second floor. Floor areas of these units will vary from 1,618 to 1,946 sq. feet. Size of private space and balcony at each unit will vary from 250 to 1,000 sq. feet and from 30 to 250 sq. feet, respectively. Garage at each unit will be over 400 sq. feet floor area. Footprint areas of new four (4) buildings will vary from 2,960 to 4,000 sq. feet. A layout of the proposed building footprints is shown in Figure 2, *Site Plan and Exploration Map*. Besides buildings, site improvements will include driveways, parking lots, open spaces, trash enclosures, a picnic shelter, landscaping, perimeter walls, etc.

New buildings will maintain the following minimum setbacks to the property limits: 10 feet at the north and south sides; 20 feet to the east side (along Bewley Street); and 15 feet on the west side. Separation among the building structures will vary from 23 to 28 feet.

1.3 Field Exploration

Field exploration at the site was conducted on January 18, 2020, which consisted of three (3) 8-inch diameter bore holes, B-1 to B-3, within the project area, drilled to depths varying from 25 to 50 feet below the existing grade. Drilling of these holes was done utilizing a truck mounted CME-75 drilling rig, which was equipped with an automatic trip hammer and hollow stem augers (HSA). Drilling equipment and crew were provided by Advanced Drilling & Sampling, LLC, whom we retained for these services. Approximate drilling locations are shown in Figure 2, *Site Plan and Exploration Map.*

During drilling, bulk bag, SPT (Standard Penetration Test), and relatively undisturbed ring samples were taken at selected depth intervals. Bulk bag samples were taken from the soil cuttings at shallow depths (upper 5 feet) that came out to surface as well as were stuck to the auger stem. Ring samples were obtained utilizing a modified California drive sampler, in accordance with ASTM Test Method D3550. This sampler had 2½ inches I.D. (inside diameter) and 3 inches O.D. (outside diameter). It contained 12 rings - each ring 2½ inches in outside diameter, 1 inch in height. Standard Penetration Tests (SPT) were performed using a 24-inch long, 1⅜-inch I.D., and 2-inch O.D. split spoon sampler in accordance with ASTM Test Method D1586. Both the ring and SPT samplers were driven 18 inches at selected depth intervals with an automatic trip hammer weighing 140 pounds and dropping 30 inches The number of blow counts to achieve the last 12 inches of penetration at each sampling depth are recorded in the "blows/foot" column in the field exploration logs (see Appendix A).

Logging and sampling of the above bore holes were conducted by an engineer from our firm. Each of the collected soil samples was inspected and described in general conformance with the Unified Soil Classification System (USCS) as defined in the ASTM Standard D2487. The soil descriptions were entered on the field exploration logs (Appendix A). After logging and sampling, bore holes were backfilled with grout mix at depths below about 18 feet, which were followed by backfill with excavated soil spoils up to the surface. Collected soil samples were properly sealed and transported to the laboratory for further evaluations and geotechnical tests.

1.4 Laboratory Tests

In order to evaluate suitability of the subsurface soils and to obtain necessary geotechnical parameters for the proposed residential construction, we conducted the following laboratory tests on selected soil samples at different depths:

- Field moisture content and dry density (ASTM D2216 and ASTM D7263);
- Percent finer than No. 200 Sieve (ASTM D1140);

- Expansion Index (ASTM D4829);
- Direct Shear (ASTM D3080); and
- Sulfate and chloride contents (California Test Methods 417 and 422).

Brief descriptions of laboratory test procedures and test results are presented in Appendix B of this report.

2.0 GEOLOGIC AND GEOTECHNICAL FINDINGS

2.1 Subsurface Geologic Profile

Subsurface soils, as encountered within the drilled holes, consist of fill soils up to a depth about 3 feet, followed by native alluvial soils up to the maximum explored depth of 50 feet. A broad plain surrounding the site is covered with young alluvial fan deposits (Qyf) of Holocene to late Pleistocene age.

Fill soils are grayish brown color silty, clayey fine sand to silty fine sand with little clay. Underneath the fill, alluvial soils up to a depth about 15 feet are primarily sand - light gray to light brown color, mottled, fine sand with few to little silts. Soils within depths from about 15 to 25 feet are light brown to light olive gray color silty, clayey soils with high fine contents, little to some fine sands. Soils below 25 feet and up to the maximum explored of 50 feet comprise of light grayish brown to light gray color sand to silty sand with few to little clay, sand grains varying from fine to coarse. Fine contents (silt, clay) of sandy layers vary from about 8 to 36 percent while the same for the silty, clayey soils within depths from 15 to 25 feet vary within a range about 55 to 72 percent.

Subsurface geologic profiles are found to be fairly consistent across the site. Descriptions of subsurface soils are presented in the field exploration logs (Appendix A). Important geotechnical characteristics of the subsurface soils that are relevant for the proposed developments are discussed briefly in the following subsections.

2.1.1 Field Moisture and Density

Near surface soils (upper about 5 feet) are relatively loose to medium dense; underlying soils are medium dense to dense, stiff to very stiff - gradually denser, stiffer with depth. Field dry densities within upper 10 feet vary from 92.6 to 102.1 pcf. Field moistures vary over a wide range - from 1.8 to 17.4 percent - within upper 10 feet. This wide variation of moisture may be attributed to varying fine contents (silt. clay) in these soils as well as retention of surface runoff within the upper fill soils.

2.1.2 Expansion Potential

Subsurface soils at shallow depths (upper 5 feet) are sand to silty sand with little clay. Laboratory test results indicated very low expansion potential (per ASTM

D4829) for a representative soil sample, taken from upper 5 feet, with tested Expansion Index (EI) value of 11. These soils are considered non-expansive per Section 1803.5.3 of the 2019 CBC (California Building Code).

2.1.3 Shear Strength Parameters

Shear strength properties of subsurface soils near the intended foundation bottom were evaluated from laboratory direct shear tests on representative ring samples taken from a depth of 5 feet. Laboratory test results of the shear strength parameters (cohesion 115 psf and friction angle 30.5°) are within the typical range of values for fine sand with few to little silts, which are tested. These shear parameters will support the foundation design parameters - vertical and lateral bearing, frictional resistance - as recommended in this report.

2.1.4 Excavatability

Based on our observation during field exploration, subsurface soils within the anticipated depth of grading are expected to be readily excavatable by conventional earthmoving and trenching equipment in good working condition.

2.1.5 Corrosion Potentials

In general, soil environments that are detrimental to concrete have high concentrations of soluble sulfates and chlorides. Section 1904 of the 2019 CBC refers to the ACI 318 code for durability requirements of concrete. Section 19.3.2 of ACI 318-14 provides guidelines for the concrete mix designs for various exposure levels from soluble sulfate and chloride ions. There are specific requirements on the mix design when the soluble sulfate content of the soil exceeds 0.1 percent by weight or 1,000 parts per million (ppm). As a general practice (e.g., Caltrans guidelines), a threshold limit of chloride ions in the soil environment that may be considered as an external source of chloride to buried concrete is 500 ppm.

For screening purpose, one (1) representative bulk soil sample at shallow depth (within upper 5 feet) was tested for sulfate and chloride contents. The test results are summarized in Table 1 below and also, presented in Appendix B. These test results indicate that the subsurface soils have low soluble sulfate and chloride contents (Exposure Classes S0 and C1 per Section 19.3.1 of ACI 318-14). These soils are not considered corrosive to buried concrete, which will be in direct contact with soils (e.g., foundation).

Table 1 – Sulfate, Chloride Contents of Onsite Soils

Sample Location	Soil Descriptions	Sulfate (ppm)	Chloride (ppm)
B-2 @ 0 – 5 ft.	Silty fine Sand (SM) w/ little clay	134	115

2.2 Groundwater

Groundwater was encountered at depths varying from about 19 to 20 feet below the existing grade during this field exploration. Historic shallow groundwater level at the project site is within the contours of 5 and 10 feet below grade as shown in the state's seismic hazard zones report (CGS, 1997).

Fluctuations in the level of the groundwater may occur due to variations in rainfall and other factors not evident at the time of this field exploration Depth of excavation for the proposed new building structures will be on the order of 3.5 feet below the existing grade. Consequently, grading and construction will not be impacted by groundwater.

3.0 FAULTING, SEISMICITY AND SEISMIC HAZARDS

3.1 Faulting and Primary Seismic Hazards

Surface ground rupture along active fault zones and ground shaking represent primary or direct seismic hazards to structures. There are no known active or potentially active faults trending toward or through the site and the site is not within any currently designated State of California Alquist-Priolo Special Studies Zone. However, the project site is located in the highly seismic Southern California region within the influence of several faults that are considered to be active or potentially active. An active fault is defined by the State of California as a "sufficiently active and well defined fault" that has exhibited surface displacement within the Holocene time (about the last 11,000 years). A potentially active fault is defined by the State as a fault with a history of movement within Pleistocene time (between 11,000 and 1.6 million years ago).

Nearby known active and potentially active faults for the project site include the following: Newport-Inglewood-Rose Canyon Fault Zone (south Los Angeles Basin section) at about 8.6 km to the southwest; Peralta Hills Fault at about 11.7 km to the northeast; THUMS-Huntington Beach Fault (offshore) at about 13.4 km to the southwest; north terminus of Pelican Hill Fault at about 13.6 km to the south; and southeast terminus of Los Alamitos Fault at about 14.5 km to the northwest of the site.

With consideration of proximity of the above active and potentially active faults, moderate to high ground shaking can be expected at the site during the design lifetime of the proposed residential buildings. Peak ground acceleration at this site is evaluated 0.367g for 10 percent probability in 50 years (475 years return period) based on the Probabilistic Seismic Hazards Assessment Model (CGS, 2008b).

3.2 Secondary Seismic Hazards

Secondary seismic hazards for this site, generally associated with severe ground shaking, include liquefaction, seismic settlement, lateral spreading, landslide, tsunamis and seiches. Potentials for these seismic hazards are briefly discussed below.

3.2.1 Liquefaction

Liquefaction is the loss of soil strength due to a buildup of pore-water pressure during severe ground shaking. Liquefaction is associated primarily with loose (low density), saturated, fine- to medium-grained, clean cohesionless soils. Liquefaction must have all three of the following to occur simultaneously:

- Strong ground shaking,
- Shallow groundwater, and
- Loose relatively clean sands.

Historic shallow groundwater level at the project site is within the contours of 5 and 10 feet below grade as shown in the state's seismic hazard zones report (CGS, 1997). Due to shallow historic groundwater, a broad plain surrounding this site is mapped within a liquefaction hazard zone as shown in Figure 3, *Seismic Hazard Zones Map*, which is excerpted from the state's seismic hazard zones map (CGS, 1998). During our field exploration, groundwater was encountered at depths varying from about 19 to 20 feet below the existing grade.

We conducted an analysis for liquefaction potential at the project site in the event of a major earthquake. Our analysis results (see Appendix C) indicate a potentially liquefiable soil layer, about 2 feet thick, within depths from about 29 to 31 feet. Due to this relatively small zone of liquefiable soils and its depth below the existing grade, surface manifestation (such as sand boiling, ground fissure, etc.) causing loss of bearing capacity of the foundation subgrade soils is not anticipated in the event of a major earthquake. Moreover, our analysis considered concurrence of the historical shallow groundwater level during a major earthquake, which is very unlikely.

3.2.2 Seismic Settlement

During a strong seismic event, seismically induced settlement can occur within loose to moderately dense, unsaturated granular soils. Settlement caused by ground shaking is often non-uniformly distributed, which can result in differential settlement.

Seismicity level at the subject site is relatively low (Peak ground Acceleration 0.367g for 10 percent probability in 50 years). Subsurface soils are found to be medium dense, stiff. These soils are not likely to experience damaging settlement during a major seismic event. Our estimate of maximum dynamic settlement at this site is about 0.29 inch (see Appendix C). Integrity of the proposed new 2-story condominium building structures will not be adversely impacted if the settlement estimate (static and seismic), as discussed in Section 4.4, is considered in design.

3.2.3 Lateral Spreading

Lateral spreading typically occurs as a form of horizontal displacement of relatively flat-lying alluvial material toward an open or “free” face such as an open body of water, channel, or excavation. In soils this movement is generally due to failure along a weak plane, and may often be associated with liquefaction. As cracks develop within the weakened material, blocks of soil displace laterally towards the open face. Cracking and lateral movement may gradually propagate away from the face as blocks continue to break free.

Generally, failure in this mode is analytically unpredictable, since it is difficult to determine where the first tension crack will occur. The subject site is relatively far away from a "free" face. The nearest free face of the San Ana River channel is about 1 km to the southeast of this site. A potentially liquefiable subsurface soil layer is found to be too deep (at ~29 feet) to cause any surface manifestation (see Section 3.2.1). With consideration of these factors, probability of lateral spreading occurring at the site during a seismic event is considered to be very low.

3.2.4 Landslides

Topography within this site and the surrounding general area is fairly flat. The site is not mapped within any landside hazard area as shown in Figure 3, *Seismic Hazard Zones Map*. No upsloping hill side grade exists within close vicinities of the site. Consequently, potential for seismically-induced landslides, or debris flows does not exist for this site.

3.2.5 Tsunamis and Seiches

Tsunamis are tidal waves, which are generated by fault displacement or major ground movement. The site is far inland from the Pacific Coast; so the hazard from tsunamis is non-existent.

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. At the present time, no water storage reservoirs are located at high elevation within the immediate vicinity of the site. Therefore, hazards from seiches do not exist for this site.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 General

Based on our geotechnical investigation findings, it is our opinion that the subsurface soils are suitable to support the proposed new 2-story condominium buildings and associated site improvements provided that the geotechnical design parameters and recommendations in this report are taken into account during design and construction of this project. Presented hereafter are our recommendations for grading; geotechnical design parameters for foundations, floor slab, driveway, parking lots, exterior flatwork; and other relevant design parameters, construction considerations for this project.

4.2 Earthwork

Earthwork for this residential construction project will consist of site clearing; excavation and preparation of the building pads, footings; and grading for various site improvements. Recommendations for site earthwork are provided in the following paragraphs.

4.2.1 Site Preparation

Prior to the grading, the site shall be cleared of any weeds, debris, vegetation, topsoils, and any remnants from previous construction. Any existing utility lines, buried abandoned utilities or objects shall be removed and/or rerouted if they interfere with the proposed construction. The cavities resulting from removal of utility lines and any buried obstructions shall be properly backfilled and compacted as recommended in Section 4.2.3 of this report.

4.2.2 Excavation

Building Pads - Excavations within the building pad areas shall extend minimum 3.5 feet below the existing grade or 2 feet below the bottom of the footings, whichever is deeper. Laterally, excavation limits shall extend minimum 2 feet from the outer edges of the footings.

Perimeter Wall, Picnic Shelter, Minor Footings - Excavations for footings for the perimeter block walls, picnic shelter, and minor site improvements (e.g., planter wall, seat wall, etc.), excavations shall extend minimum 2.5 f eet below the existing grade or 12 inches below the bottom of the footings, whichever is deeper.

Laterally, excavation limits shall extend minimum 18 inches from the outer edges of the footings wherever not constrained by the property limits or any structural elements.

Pavements, Flatwork Areas - For driveways, parking lots, and exterior flatwork areas (such as walkway, patio, etc.), excavation shall extend minimum 18 inches below the existing grade or 12 inches below the final soil subgrade (underNeath the base layer), whichever is lower. Lateral limits of overexcavtion shall extend minimum 18 i nches beyond the edges of these improvements wherever not constrained by the property limits or any structural elements.

After excavations as recommended above, if loose, organic, yielding (pumping) or otherwise unsuitable soils are exposed, excavations shall extend deeper until competent bottom soils are reached. Competent removal bottoms shall be unyielding (not penetrating more than 2 i nches) by hand probing with a cone tipped steel probing rod.

4.2.3 Fill Placement and Compaction

After excavations as recommended above and prior to placement of any fill materials, subgrade soils at the removal bottoms shall be scarified, moisture conditioned (adding water as needed) to within 2 pe rcent of the optimum moisture, and recompacted in place to minimum 90 percent (ASTM D1557).

If soils with high moisture contents (moisture exceeding more than 6 percent of the optimum moisture) are encountered at the excavation bottoms, these soils will need to be aerated and/or blended with open-graded gravel (½- to ¾-inch size) prior to recompaction. Same processing will be required for reusing excavated high moisture soils as compacted fill.

Fill soils shall be placed in thin lifts - loose lift thickness not exceeding 8 inches - moisture-conditioned (adding water as needed) to within 2 p ercent of the optimum moisture, and compacted to minimum 90 percent (ASTM D1557). Base materials underneath the driveway, garage floor, car port slab, flatwork and wherever else used, shall be placed at minimum 95 percent compaction (ASTM D1557) with moisture content within 2 percent of the optimum moisture.

During grading, field density tests shall be taken for the graded fill soils, base materials, and asphalt concrete at the following schedule:

- Minimum one (1) field test over 1,500 square feet area of the building pad and site improvements for each one-foot lift of fill and at the final grade.

- Minimum one (1) field test for every 500 square feet area for each lift of asphalt concrete course.

- Minimum one field test for each 50 linear feet of trench backfill for each one-foot lift of fill and at the final grade.

- Minimum one field test for each 50 linear feet of compacted foundation bottoms.

Field density tests may be taken by utilizing a Nuclear Gauge (ASTM D6938) or a combination of both Nuclear Gauge and Sand Cone (ASTM D1556) methods.

4.2.4 Trench Backfill

Utility trenches shall be backfilled with compacted fill in accordance with Section 306-12 of the *Standard Specifications for Public Works Construction,* ("Greenbook"), 2018 Edition.

Utility trenches can be backfilled with onsite or import soils that meet the fill soils criteria in Section 4.2.5. Prior to backfilling the trenches, pipes shall be bedded in and covered with import granular material that has a minimum Sand Equivalent (SE) value of 40 (per ASTM D2419). Bedding sands shall be placed by mechanical compaction; jetting is not recommended. Native soil backfill over the pipe bedding zone shall be placed in thin lifts, moisture conditioned to within 2 percent of the optimum moisture and mechanically compacted to minimum 90 percent compaction (ASTM D1557).

Wherever mechanical compaction as recommended above is not practical due to narrow trenches (width 10 inches or less), alternative backfill method such as placement of pea gravel (size up to 1/2") or sand-cement slurry (minimum 2 sacks of cement for 1 cubic yard mix) may be considered for backfill of utility trenches.

4.2.5 Fill Materials

Onsite soils that are free of organics, debris and oversize particles (e.g., cobbles, rubble, etc. that are greater than 3 inches in the largest dimension) are suitable to be used as fill. Import soils, if used, shall be free of organics, non-expansive (Expansion Index less than 20 per ASTM D4829), and shall have no corrosion impacts on buried metals and concrete.

Base materials underneath driveway, parking lots, and exterior flatwork (such as patio, walkway) may consist of crushed miscellaneous base in conformance to Section 200-2.4 of the *Standard Specifications for Public Works Construction*, (“Greenbook”), 2018 Edition.

Prior to import, geotechnical consultant shall evaluate and test the import soils and base materials in order to confirm the quality of the materials.

4.2.6 Temporary Excavation

Temporary excavations during grading, away from the influence zone (1:1 projection downward and outward from the footing bottoms) of any existing foundations such as the perimeter wall footings, may be constructed according to the slope ratios presented in Table 2 below.

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Table 2 – Slope Ratio for Temporary Excavation

Maximum Depth of Cut (feet)	Maximum Slope Ratio* (horizontal:vertical)
0 - 4	Vertical
4 - 10	1:1

*Slope ratio assumed to be uniform from top to toe of slope.

In order to retard raveling and sloughing, surfaces exposed in the excavation faces shall be kept moist but not saturated. Adequate provisions shall be made to protect the slopes from erosion during periods of rainfall.

Excavated soil spoils, any construction debris, and construction materials shall not be stockpiled and any heavy construction equipment shall not be placed within a distance H (ft) from the top of unsupported excavation/trench edge, where H is the depth of the excavation/trench in feet. Height of stockpiles of construction materials, debris shall not exceed 6 feet.

During grading, all applicable requirements in Article 6, Section 1541.1 of the State of California Construction Safety Order (CAL/OSHA) shall be met for protection of the construction workers working inside the excavations.

4.2.7 Shrinkage and Subsidence

Volume change of the onsite soils from cut to fill conditions is anticipated during grading in order to prepare a level surface for the proposed building pad. Assuming the fill will be compacted to an average relative compaction of 92 percent, cut-fill shrinkage of 15 to 20 percent may be considered for the onsite soils. Further volume loss will occur through subsidence during preparation of the soil subgrade. Although the contractor's method and equipment utilized during grading will have a significant effect on the amount of ground subsidence, our experience indicates as much as 2 inches of subsidence may occur in areas that will be prepared to receive fill. These values are exclusive of losses due to stripping or removal of subsurface obstructions.

4.3 Seismic Design Parameters

Based on our investigation findings, subsurface soil profile at this site may be characterized within the category of Site Class D ("Stiff Soil") according to Chapter 20 of ASCE/SEI 7-10 as referred in Section 1613.2.2 of the 2019 CBC. Based on the nature of occupancy, existing building and the proposed additions will fall into Risk Category II (per Table 1604.5 of the 2019 CBC). A site-specific ground motion hazard analysis was not required for this site with Site Class D provided that the value of seismic response coefficient C_S conforms with the conditions of exception as outlined in Section 11.4.8 of ASCE/SEI 7-16. Consequently, seismic design parameters are determined from the general ground motion analysis in accordance with Section 1613.2 of the 2019 CBC. Seismic design parameters for above soil profile, Risk Category, and site location (Latitude: 33.7553°N; Longitude: 117.9185°W at the center of the site) are determined in accordance with Section 1613.2 of the 2019 CBC. See these parameter in Table 3 below. These are derived from risk-targeted Maximum Considered Earthquake (MCE_R) based spectral response analysis.

Table 3 – Seismic Design Parameters

Categorization/Seismic Parameters	Design Value
Site Class	D
Mapped MCE Spectral Acceleration for Short (0.2 Second) Period, S_S	1.327g

Categorization/Seismic Parameters	Design Value
Mapped MCE Spectral Acceleration for a 1-Second Period, S_1	0.472g
Short Period (0.2 Second) Site Coefficient, F_a	1.0
Long Period (1 Second) Site Coefficient, F_v	1.828
Adjusted Spectral Response Acceleration at 0.2-Second Period, S_{MS}	1.327g
Adjusted Spectral Response Acceleration at 1-Second Period, S_{M1}	0.863g
Design (5% damped) Spectral Response Acceleration for Short (0.2 Second) Period, S_{DS}	0.885g
Design (5% damped) Spectral Response Acceleration for a 1-Second Period, S_{D1}	0.575g
Seismic Design Category	D

Proposed condominium buildings and any structural improvements at this site shall be designed for the above seismic parameters.

4.4 Foundation Design

Proposed 2-story condominium buildings, perimeter block walls, picnic shelter, and other site improvements such as seat wall, planter wall, etc., can be supported on shallow spread foundations (wall and/or column) bearing on properly compacted soil subgrade, prepared as recommended in Sections 4.2.2 and 4.2.3. Geotechnical design parameters for spread foundations are described in the following subsections.

Footing Dimensions and Embedments – Minimum dimensions and embedment for various foundations are listed below:

Table 4 – Foundation Dimensions and Depths

Foundation Type	Minimum Dimension	Minimum Embedment
2-story condominium buildings	18 inches wide wall footings; 18"X18" column footings	18 inches below the lowest adjacent grade [1]

Foundation Type	Minimum Dimension	Minimum Embedment
Perimeter block wall	18 inches wide wall footings	18 inches below the lowest adjacent grade [1]
Picnic shelter	18"X18" column footings	18 inches below the lowest adjacent grade [1]
Minor foundations (e.g., seat wall, planter wall, barbeque pit wall, etc.).	12 inches wide wall footings	12 inches into exterior soil subgrade (excl. topsoils)

[1] Lowest adjacent grade is considered as the top of interior slab-on-grade for the interior footings or the final exterior soil grade (excluding landscape topsoil) for the perimeter footings.

Footings located adjacent to utility trenches or buried vaults shall be embedded below an imaginary 1:1 (horizontal: vertical) plane projected upward and outward from the bottom edge of the trench or vault, up towards the footing.

Vertical Bearing - For footings with the minimum embedment of 12 inches as described in Table 4 above and bearing on properly compacted soil subgrade, allowable vertical bearing capacity of 1,500 psf may be considered for design, which may be increased by 500 psf for each additional foot of embedment up to a maximum value of 2,500 psf. These bearing values may be increased by one-third for short-term loads (e.g., seismic, wind loads).

Lateral Bearing - Lateral loads are resisted by friction at the footing bottoms, between concrete and the supporting soil subgrade, as well as by the passive resistance of the soils from foundation embedment. An allowable frictional resistance of 0.3 may be used for design of concrete foundations poured on properly compacted soil subgrade. Allowable passive resistance of the soils may be considered 200 psf/ft of footing embedment if the foundation concrete is poured neat against properly compacted fill soils without leaving any void pockets. These friction and passive values have already been reduced by a factor-of-safety of 1.5. When frictional and passive resistances are combined to compute the total lateral resistance, no reduction is needed to any of these two components. One-third increase of the soil's passive resistance is allowed for short-term seismic or wind loads.

Settlement Estimates – The static settlement of the proposed 2-story building structures will depend on the actual footing dimensions and the imposed vertical loads. With consideration of the subsurface geologic profile and the allowable bearing capacity as presented above, maximum static settlement on the order of 0.75 inch may be considered. Due to sandy nature of the subgrade soils within foundation surcharge influence zones (up to the depth where surcharge load from foundation is 10% of the foundation load), majority of static settlement will likely occur immediately after construction.

Post-construction maximum seismic settlement is evaluated to be 0.29 inch (see Appendix C). Due to fairly uniform subsurface geologic profile, potential of different seismic settlement will be greatly diminished across the building pads. Total differential settlement (static and seismic combined) across the building pads may be considered on the order of 0.5 inch over a horizontal distance of 40 feet.

4.5 Interior Slab-on-Grade

Floor slab-on-grade inside the building (excluding garage floor; see Section 4.6 for garage floor) shall be placed on properly compacted subgrade, prepared as recommended in Section 4.2.3 of this report. As a minimum, floor slab shall be 4 inches thick and be reinforced with No. 3 rebars at 15 inches on-center each way at mid-depth throughout the slab. The above minimum rebars will not prevent the development of slab cracks but will aid in keeping joints relatively tight and reduce the potential for differential movement between adjacent panels. Care shall be taken to avoid slab curling if slabs are poured in hot weather. Prior to the slab pour, all utility trenches shall be properly backfilled and compacted as outlined in Section 4.2.4.

In areas where a moisture-sensitive floor covering (such as vinyl, tile, or carpet) will be used, a moisture retarder (minimum 10-mil thick Visqueen or equivalent) shall be placed inside sand, between the slab and compacted soil subgrade. The moisture retarder shall be protected with 2 inches of sand above as well as 2 inches of sand below in order to aid in the concrete cure and to prevent punctures to membrane, respectively. Underslab import sand shall have a minimum Sand Equivalent value (per ASTM D2419) of 40. Sand above the membrane needs to be kept lightly moist prior to placement of concrete. Moisture retarder seams shall be overlapped minimum 6 inches and taped or otherwise sealed.

4.6 Garage Floor, Exterior Flatwork

Minimum concrete section, underlying base thickness, concrete strengths, and minimum reinforcements for garage floor and exterior flatwork (such as walkway, patio, trash enclosure pad, etc.) are presented in Table 5 below. Appropriate joints and saw cuts should be provided for all the concrete slabs in accordance with either Portland Cement Association (PCA) or American Concrete Institute (ACI) guidelines.

Table 5 – Garage Floor, Exterior Flatwork

Proposed Improvements	Min. Slab Thickness (inch)	Min. Base Thickness (inch)	Min. Concrete Strength (psi)	Minimum Reinforcement [1]
Garage floor, trash enclosure pad	6.0	4.0	2,500	#4 rebars @ 18" o/c, both ways
Walkway, patio, other exterior flatwork (non vehicular)	4.0	4.0	2,500	#3 rebars @ 15" o/c, both ways

[1] Rebars shall be placed at mid-depth of the slab, flatwork concrete section.

If paver blocks are used for exterior flatwork, total thickness of the paver blocks and underlying concrete shall be minimum 4 inches and minimum reinforcement for concrete slab underneath paver blocks shall be No. 3 rebars at 18 inches on-center each way at mid-depth throughout the slab.

Specifications for base materials are outlined in Section 4.2.5, *Fill Materials*. Soil subgrade underneath the concrete and base layers shall be prepared and compacted in accordance with Section 4.2.3 of this report.

4.7 Pavements

Driveways, parking lots inside the complex may be asphalt or concrete paved. Driveway approaches to the site from Bewley Street will be concrete paved. Preliminary design parameters for both the pavement types, materials specifications and compaction requirements are presented in the following subsections.

4.7.1 Asphalt Pavement

With consideration of subsurface soil conditions at shallow depths, we assumed an R-value of 30 for the pavement subgrade soils for preliminary design purpose. Considering this R-value for soil subgrade, asphalt pavement sections are evaluated for Traffic Index (TI) values from 4.5 through 6.0 following the

Caltrans design guidelines (Caltrans, 2009). These sections are listed in Table 6 below. Appropriate TI values for parking lots, driveway shall be selected by the project civil engineer and appropriate R-value of the subgrade soils shall need to be determined after completion of rough grading to finalize the pavement design.

Where asphalt pavements meet concrete or existing pavements, the concrete and/or asphalt should be sprayed with an SS-1 or CSS-1 emulsion. Proper asphalt compaction next to concrete pavements, curbs, and existing pavements is important to provide a relatively impermeable contact between the two materials.

Table 6 - Asphalt Concrete Pavement Sections

Traffic Index (TI)	Minimum Asphalt Thickness (inches)	Minimum Base[1] Thickness (inches)
4.5	3.0	5.0
5.0	3.0	6.0
5.5	3.5	7.0
6.0	4.0	7.0

[1] Minimum design R-value of the base materials is 78.

Use of concrete cutoff or edge barriers shall be considered at the perimeter of the common parking or driveway areas when they are adjacent to either open (unfinished) or landscaped areas.

Soil subgrade, base and asphalt layers shall be prepared and compacted as recommended in Section 4.2.3. S pecifications for base materials and asphalt concrete are provided in Section 4.2.5.

4.7.2 Concrete Pavement

Pavement sections that are subject to heavy traffic, load from truck wheels such as driveway approaches to the site, driveway for trucks (trash truck, fire truck) may be concrete paved with minimum 6 inches thick concrete overlying a minimum 4-inch thick base layer considering a design Traffic Index value of 6.0 and an R-value of 30 for the soil subgrade. Appropriate TI value for truck traffic shall be selected by the project civil engineer and appropriate R-value of the subgrade soils shall need to be determined after completion of rough grading to finalize the concrete pavement design.

All concrete pavements shall have a minimum 28-day concrete compressive strength of 3,500 psi and have appropriate joints and saw cuts in accordance with either Portland Cement Association (PCA) or American Concrete Institute (ACI) guidelines. Minimum reinforcement for pavement slab for shall be No. 4 rebars at 18 inches on-center each way at mid-depth throughout the slab.

Soil subgrade and the base layer shall be prepared and compacted as recommended in Section 4.2.3. Specifications for base materials are provided in Section 4.2.5.

4.8 Lateral Earth Pressures

No above grade retaining wall is shown in the site plan. However, short retaining walls (aesthetic grade separation, planter wall, etc.) may be incorporated as a part of future site improvements. Any retaining wall for this project shall be designed for the lateral earth pressures presented in Table 7 below. These pressure values are expressed as equivalent fluid unit weight (in pcf). Backfill for the retaining walls may consist of onsite or import non-expansive soils (Expansion Index less than 20 per ASTM D4829). Backside of the retaining walls (within retained height) shall be waterproofed and appropriate drainage (such as weep holes or French drain) shall be installed behind the walls so that any hydrostatic pressure cannot develop.

Lateral pressure values (active and at-rest) in Table 7 do not contain any factor of safety. Structural design needs to take into consideration applicable factors of safety and/or load factors for these lateral pressures. However, the passive resistance values in Table 7 are allowable values, already reduced by a factor of safety 1.5.

Table 7 – Lateral Earth Pressures

Loading Condition	Equivalent Fluid Unit Weight for Level Backfill (pcf)
Active	35
At-Rest	55
Passive	200

If the wall can yield enough to mobilize full shear strength of backfill soils, then the wall can be designed for "active" pressure. If the wall is not allowed to yield under the applied load, the shear strength of the soil cannot be mobilized and the earth pressure will be higher. Such walls shall be designed for "at-rest" conditions.

In addition to the above lateral pressures from retained earth, lateral pressures from other superimposed loads such as load from any adjacent structures shall be added, if those loads fall within a 1:1 projection of wall foundations.

4.9 Impact on Adjacent Structures

Existing topography within the subject lot and neighboring properties on the north, south, and west sides are at about the same elevation. East boundary of the property is along Bewley Street. As discussed in Section 1.2, new buildings will maintain adequate setbacks to the property limits and these buildings will have adequate separation from one another. Surcharge loads from foundations of the proposed new buildings, within 1:1 projection downward and outward from the new foundation edges will not encroach into the existing footings of the perimeter walls and building structures on t he neighboring lots. Accordingly, there will not be any impact from the proposed construction to the adjacent structures.

4.10 Cement Type and Concrete Properties

Laboratory test results indicate that the soluble sulfate and chloride contents of subsurface soils at shallow depth (upper 5 feet) are low (Exposure Classes S0 and C1 per Section 19.3.1 of ACI 318-14). These soils are not considered corrosive to buried concrete, which will be in direct contact with soil (e.g., foundations). As a result, there is no restriction on the type of cement and minimum concrete strength from the durability standpoint. Conventional Type II cement (ASTM C150) may be used for concrete for this project. Minimum 28-day compressive strength (ASTM C39) of structural concrete shall be 2,500 psi unless specified otherwise in this report. Water-soluble chloride ion content in the concrete (per ASTM C1218) shall not exceed 0.3 percent of the cement content (by weight).

4.11 Corrosion Measures for Buried Metals

Non-metal underground pipes (e.g., PVC) shall be used instead of metal pipes. If ferrous metal components (e.g., underground pipes, anchor hold down, metal straps for foundation) are planned to be buried with direct contact with subsurface soils, the following corrosion mitigation measures shall be implemented for this project:

- Below-grade ferrous metals shall be given a high-quality protective coating, such as 20-mil thick plastic tape, extruded polyethylene, coal-tar enamel, or Portland Cement mortar.
- Below-grade ferrous metals shall be electrically insulated (isolated) from above-grade ferrous metals and other dissimilar metals by means of dielectric fittings in utilities and exposed metal structures breaking grade.
- Reinforcements (rebar, wire mesh) within concrete that will be in direct contact with the site soils shall have at least 3 inches of concrete cover.

4.12 Surface Drainage

In order to prevent ponding and intrusion of surface runoff into foundation subgrade, positive drainage shall be provided around the perimeters of the proposed new building and any structural footings. In compliance with Section 1804.4 of the 2019 CBC, the ground immediately adjacent to the proposed new foundations shall be sloped away from the building at a gradient not less than 5 percent for a minimum distance of 10 feet from face of the building walls. If any physical obstructions or lot lines prohibit 10 feet of horizontal distance, a 5 percent gradient shall be provided to an approved alternative method of diverting water away from the foundation. Swales used for this purpose shall be sloped at minimum 2 percent wherever located within 10 feet of the building foundation. Impervious surfaces (such as concrete flatwork) within 10 feet of the new foundations shall be sloped at minimum 2 percent away from the building walls.

For area drains collecting surface run-off within a flat area, finish grades surrounding the drains shall maintain the following minimum gradient - 2 percent for dirt, landscaped surfaces and 1 percent for paved surfaces (e.g., concrete, paver blocks).

4.13 Landscape Considerations

The potential for undesired foundation and slab movements may be reduced or minimized by following certain landscape practices. The main goal for proper landscape design should be to minimize fluctuations in the moisture content of the soils surrounding the structures. In addition to maintaining positive drainage, appropriate plant/tree selections and sprinkler/irrigation practices are extremely important to the long-term performance of the foundations and slabs. As a guideline to landscaping practices, we recommend the following measures:

- Planting flowers or shrubs within 5 feet of any perimeter wall or column foundation should not be allowed.
- Ground cover plants with low water requirements (or drought tolerant) may be acceptable for landscaping near foundations. Ground cover vegetation helps reduce fluctuations in the soil moisture content. Watering should be limited to the minimum needed to maintain the ground cover vegetation near foundations.
- As an alternative to ground cover vegetation, sealed-bottom planter boxes may be considered within 5 feet of building structures.
- Trees should not be planted within a minimum distance of 10 feet from any structural foundations.
- If irrigation/sprinkler systems are to be used, these should be installed all around the structure to provide uniform moisture throughout the year. The irrigation/sprinkler systems should spray no closer than 5 feet from the foundation. The sprinkler system should be checked for leakages once a month. Significant foundation movements can occur if the soils under the foundations are exposed to a source of free water.

4.14 Observation, Tests during Grading

All grading and excavation shall be performed under the observation and testing of the geotechnical consultant at the following stages:

- After completion of site clearing and excavation to the recommended depths;
- During grading for the building pads, driveways, and other site improvements;
- After excavation of all footings and prior to placement of concrete;
- During backfill for utility trenches; and
- Whenever any unusual or unexpected geotechnical conditions are encountered.

4.15 Limitations

The conclusions and recommendations in this report are based in part upon data that were obtained from a limited number of field explorations, laboratory test results, and limited information on historical events and observations. Subsurface conditions may vary across the site.

This report is not authorized for use by, and is not to be relied upon by any party except the following parties: the current owner(s) of this property; their successors and assignees as the future owner(s) of this property; and the design professionals, contractors for this project. Use of or reliance on this report by any other party is at that party's risk. Unauthorized use of or reliance on this report constitutes an agreement to defend and indemnify ZS Engineering from and against any liability which may arise as a result of such use or reliance.

Geotechnical investigation and relevant engineering evaluations for this project are performed in substantial conformance with the prevailing Building Code (2019 CBC) and general practices of geotechnical engineering in southern California at the time of this report. No other warranty is expressed or implied.

5.0 REFERENCES

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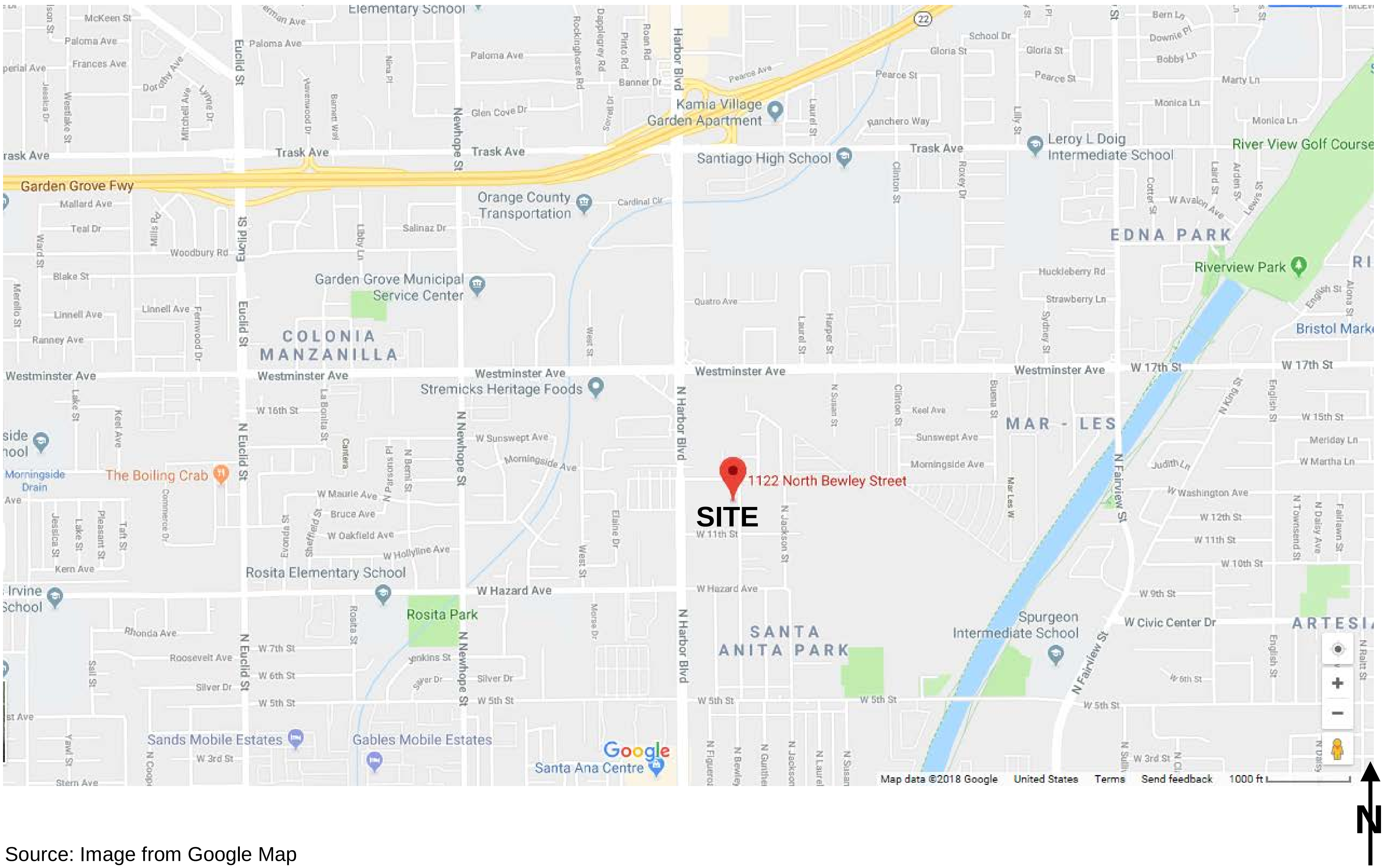


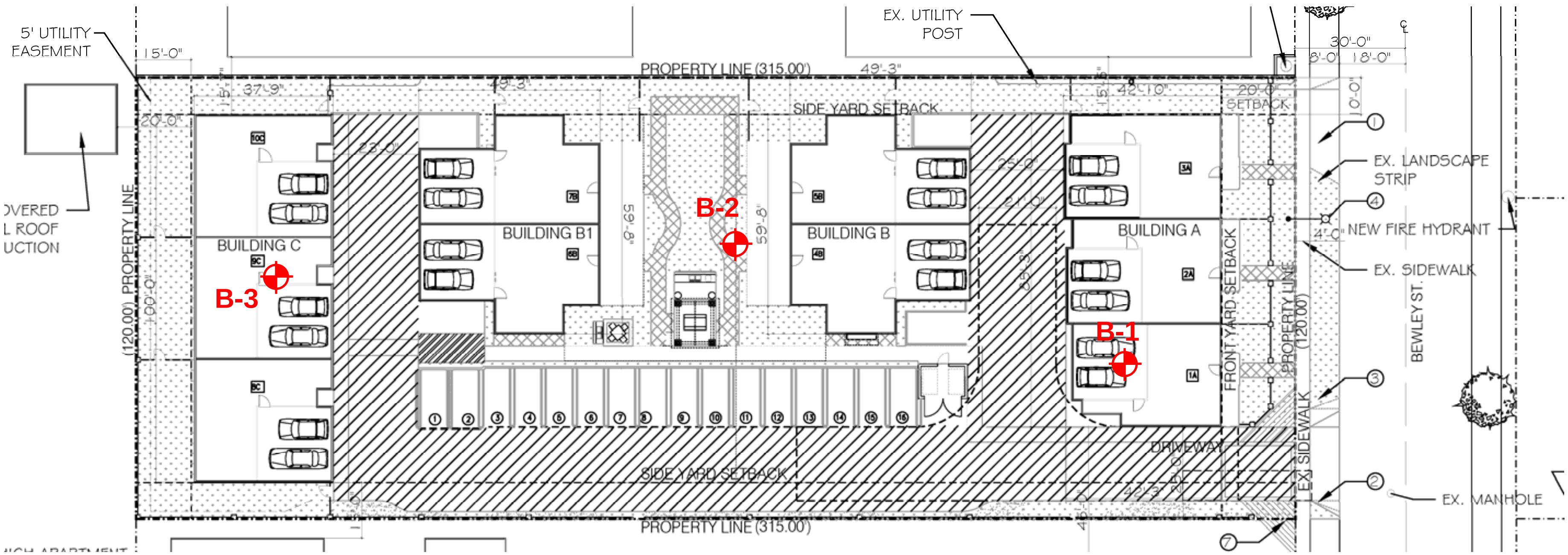
Source: Image from Google Map

Figure 1 – Site Location Map

Residential Lot at 1122 N. Bewley Street, Santa Ana, CA 92703

ZS Engineering

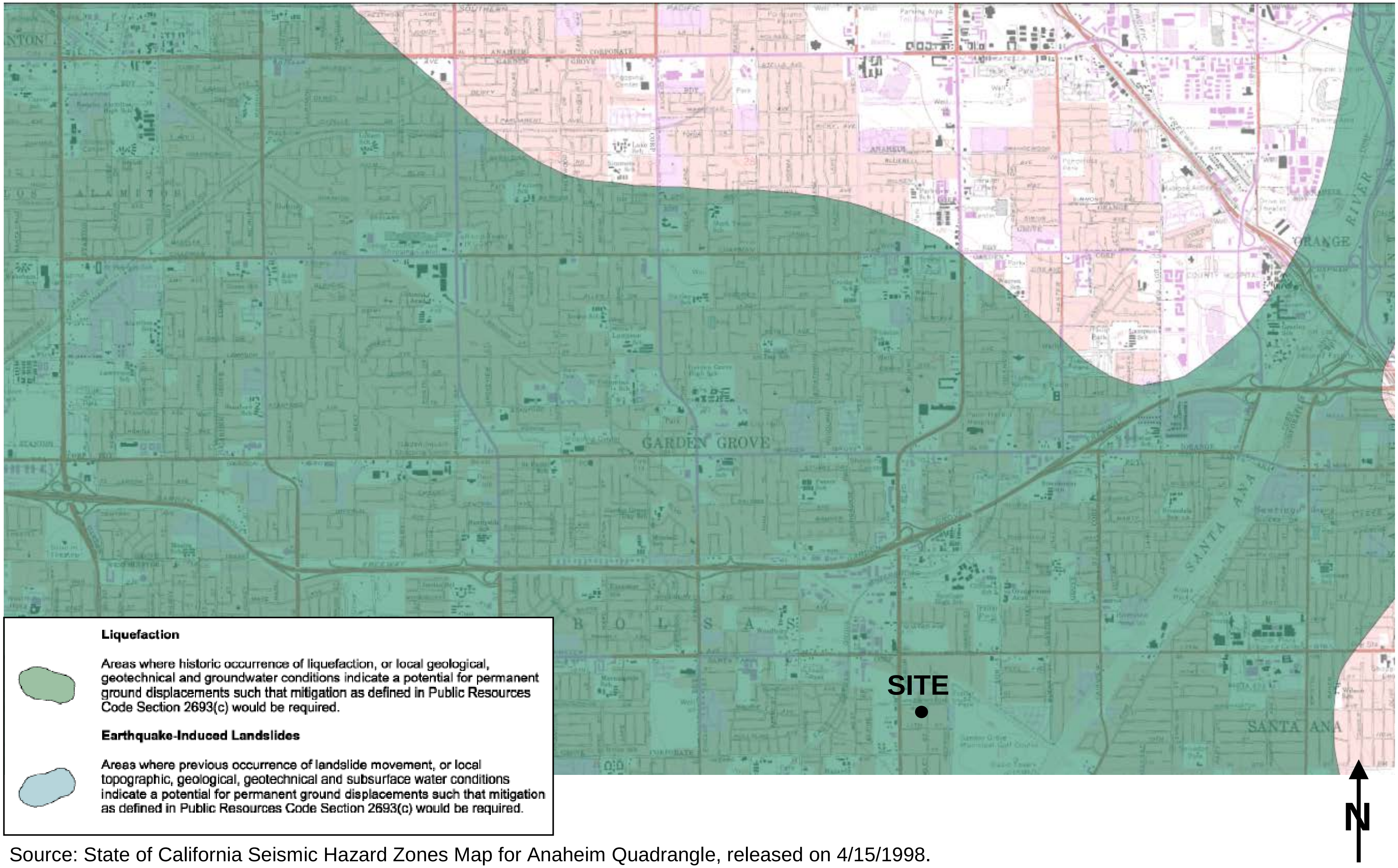
#200101



Legends:

B-3 **Approx. Bore Hole Locations**

	Figure 2 – Site Plan and Exploration Map **Condominium Complex at 1122 N. Bewley Street, Santa Ana, CA 92703**	**ZS Engineering** **#200101**



Source: State of California Seismic Hazard Zones Map for Anaheim Quadrangle, released on 4/15/1998.

Figure 3 – Seismic Hazard Zones Map

Residential Lot at 1122 N. Bewley Street, Santa Ana, CA 92703

ZS Engineering

#200101

APPENDIX A

Field Exploration Logs

ZS Engineering

113 Tomato Springs, Irvine, CA 92618
(949) 331-3232 | info@zs-engineering.com

Date of Drilling: 1/18/20 Method of Drilling: CME-75 Drilling Rig, 8-inch Dia. Hollow Stem Auger

Logged by: ZA Elevation: ~ 81.2 ft. Location: See Fig. 2, Site Plan & Exploration Map

Condominium Complex at a Residential Lot
1122 N. Bewley Street, Santa Ana, CA 92703

Bore Hole No. **B-1**

Depth (ft)	Sample No.	Blows/ft.	Ring Sample	SPT Sample	Bulk Sample	Moisture Content (%)	Dry Density (pcf)	Soil Descriptions	Soil Tests
					X			**Fill (Af):** 0 - 3': Surface covered w/ dirt, weeds.	
2	R-1	15	X		X	10.0	99.9	Grayish brown silty, clayey fine Sand (SC-SM), moist.	
	B-1				X				
					X			**Alluvium (Qyf):** 3' - 15': Lt. gray to lt. brown, mottled,	
5	R-2	20	X		X	2.3	102.1	Sand (SP-SM) to silty Sand (SM), few to little silts, fine grained sand, slightly moist to moist.	
10	R-3	24	X			3.9	95.3		
15	S-1	14		X				@ 15': Lt. brown clayey Silt (ML) w/ some fine sand, moist.	
								- Groundwater at ~19 ft. ▽	
20	S-2	16		X				@ 20': Top 6" - Same as above, wet Btm 6" - Lt. olive gray silty Clay (CL), few fine sand.	
25	S-3	25		X				@ 25': Lt. olive gray sandy Silt (ML) w/ fine sand, little clay, wet.	

- Drilling terminated at 25 feet below the existing grade.
- Groundwater was encountered at ~19 ft below grade.
- After logging & sampling, drilled hole was backfilled w/ grout mix at depths below ~18 ft. Upper ~18 ft was backfilled w/ the soil cuttings.

ZS Engineering

113 Tomato Springs, Irvine, CA 92618
(949) 331-3232 | info@zs-engineering.com

Date of Drilling: 1/18/20 Method of Drilling: CME-75 Drilling Rig, 8-inch Dia. Hollow Stem Auger

Logged by: ZA Elevation: ~ 80.5 ft. Location: See Fig. 2, Site Plan & Exploration Map

Condominium Complex at a Residential Lot
1122 N. Bewley Street, Santa Ana, CA 92703

Bore Hole No. **B-2**

Depth (ft)	Sample No.	Blows/ft.	Ring Sample	SPT Sample	Bulk Sample	Moisture Content (%)	Dry Density (pcf)	Soil Descriptions	Soil Tests
								Fill (Af): 0 - 3': Surface covered w/ dirt, weeds.	Expansion Index;
2	R-1	15	X			17.4	93.3	Grayish brown silty fine Sand (SM) w/ little clay, moist	Sulfate, Chloride
	B-1				X				
								Alluvium (Qyf): 3' - 15': Lt. gray to lt. brown, mottled,	
5	R-2	20	X			8.6	92.6	Sand (SP-SM) to silty Sand (SM), few to less amount of silts. fine grained sand, slightly moist to moist, 8.1% fines at 5 ft.	Direct Shear, Percent Fines
10	R-3	27	X			1.8	93.9		
15	S-1	17		X				@ 15': Lt. brown silty Clay (CL) w/ some fine sand, moist, 64.4% fines.	Percent Fines
20	S-2	14		X				@ 20': Lt. olive gray silty Clay (CL), some fine sand. Groundwater at ~20 ft.	
25	S-3	37		X				@ 25': Lt. olive gray sandy Silt (ML) w/ fine sand, little clay, wet, 55.3% fines.	Percent Fines
30	S-4	10		X				@ 30': Lt. grayish brown silty fine sand (SM), little clay, wet.	
35	S-5	20		X				@ 35': Lt. gray silty fine Sand (SM), trace clay, wet.	Percent Fines
40	S-6	50		X				@ 40': Lt. gray, mottled, fine to coarse Sand (SP-SM), few silts, wet, 9.2% fines.	Percent Fines

ZS ENG. #200101 **LOG OF BORE HOLE B-2**

ZS Engineering
113 Tomato Springs, Irvine, CA 92618
(949) 331-3232 | info@zs-engineering.com

Date of Drilling: 1/18/20 Method of Drilling: CME-75 Drilling Rig, 8-inch Dia. Hollow Stem Auger

Logged by: ZA Elevation: ~ 80.5 ft. Location: See Fig. 2, Site Plan & Exploration Map

Condominium Complex at a Residential Lot
1122 N. Bewley Street, Santa Ana, CA 92703

Bore Hole No. **B-2**

Depth (ft)	Sample No.	Blows/ft.	Ring Sample	SPT Sample	Bulk Sample	Moisture Content (%)	Dry Density (pcf)	Soil Descriptions	Soil Tests
45	S-6	61		X				@ 45': Lt. gray, mottled, Sand w/ few silts (SP-SM), f-m sand, wet.	
50	S-7	32		X				@ 50': Lt. gray silty fine Sand (SM), wet, 36.5% fines.	Percent Fines

- Drilling terminated at 50 feet below the existing grade.
- Groundwater was encountered at ~20 ft below grade.
- After logging & sampling, drilled hole was backfilled w/ grout mix at depths below ~18 ft. Upper ~18 ft was backfilled w/ the soil cuttings.

ZS Engineering

113 Tomato Springs, Irvine, CA 92618
(949) 331-3232 | info@zs-engineering.com

Date of Drilling: 1/18/20 Method of Drilling: CME-75 Drilling Rig, 8-inch Dia. Hollow Stem Auger

Logged by: ZA Elevation: ~ 80 ft. Location: See Fig. 2, Site Plan & Exploration Map

Condominium Complex at a Residential Lot
1122 N. Bewley Street, Santa Ana, CA 92703

Bore Hole No. B-3

Depth (ft)	Sample No.	Blows/ft.	Ring Sample	SPT Sample	Bulk Sample	Moisture Content (%)	Dry Density (pcf)	Soil Descriptions	Soil Tests
								Fill (Af): 0 - 3': Surface covered w/ dirt, weeds.	
2	R-1 B-1	17	X		X	12.3	101.1	Grayish brown silty, clayey fine Sand (SC-SM), moist.	
								Alluvium (Qyf): 3' - 15': Lt. grayish brown to lt. brown,	
5	R-2	22	X		X	3.4	96.5	Sand (SP-SM) to silty Sand (SM), few to less amount of silts, fine grained sand, slightly moist to moist.	
10	R-3	28	X			3.6	99.2		
15	S-1	18		X				@ 15': Lt. brown clayey Silt (ML) w/ some fine sand, moist.	
20	S-2	15		X				@ 20': Lt. brown silty Clay (CL), little fine sand. ▽ Groundwater at ~20 ft.	
25	S-3	31		X				@ 25': Lt. olive gray sandy Silt (ML) w/ fine sand, little clay, wet.	

- Drilling terminated at 25 feet below the existing grade.
- Groundwater was encountered at ~20 ft below grade.
- After logging & sampling, drilled hole was backfilled w/ grout mix at depths below ~18 ft. Upper ~18 ft was backfilled w/ the soil cuttings.

APPENDIX B

Laboratory Test Procedures and Test Results

Laboratory Test Procedures and Test Results

Brief description of the laboratory test procedures and test results are presented hereafter.

Field Moisture and Density: Field moisture contents and dry densities of subsurface soils within upper 10 feet were determined from the collected ring samples. These moisture and density values were determined in accordance with the ASTM Test Methods D2216 and D7263, respectively. Test results are presented in the field exploration logs (see Appendix A).

Percent Fines (< No. 200): S elected soil samples were wash sieved through a No. 200 U .S. Standard brass sieve in accordance with the ASTM Test Method D1140 in order to determine the percent fines (silts and clays). The test data were used to define the Unified Soil Classification for tested soil samples. Test results are summarized in the following table:

Sample Location	Soil Descriptions	Percent Finer than No. 200 Sieve
B-2 @ 5 ft.	Fine Sand w/ few silts (SP-SM)	8.1
B-2 @ 15 ft.	Silty Clay w/ some fine sand (CL)	72.4
B-2 @ 25 ft.	Sandy Silt w/ fine sand, little clay (ML)	55.3
B-2 @ 35 ft.	Silty fine Sand, trace clay (SM)	31.5
B-2 @ 40 ft.	Sand w/ few silts, f-c sand (SP-SM)	9.2
B-2 @ 50 ft.	Silty fine Sand (SM)	36.5

Expansion Index: Expansion Index (EI) test was performed on a representative bulk soil sample, taken from shallow depth, in accordance with the ASTM D4829 Test Method. Test results are summarized in the following table and also, presented in this appendix.

Sample Location	Soil Descriptions	Expansion Index	Expansion Potential
B-2 @ 0 – 5 ft.	Silty fine Sand w/ little clay (SM)	11	Very Low

Direct Shear: Direct shear tests under consolidated drained condition were performed on selected ring samples in general accordance with the ASTM Test Method D3080. The samples were soaked for a minimum of 24 hours under a surcharge equal to the applied normal force during testing. Samples and specimens were then transferred to the shear box, reloaded, and pore pressures set up in the sample (due to transfer) were allowed to dissipate for a period of approximately one-hour. Following pore pressure dissipation, samples were subjected to shearing forces. The samples were tested under various normal loads by a motor-driven, strain-controlled, direct-shear testing apparatus at a strain rate of 0.05 inch per minute. Shear deformation was recorded until about 0.3 inches of shear displacement was achieved. Ultimate shear strengths for different surcharge pressures were selected from the shear-stress deformation data and plotted to determine the shear strength parameters. Test results are presented on the *Direct Shear Test Results* figure in this appendix.

Sulfate and Chloride Contents: We retained service from Anaheim Test Laboratory to perform the corrosion evaluation tests comprising of sulfate and chloride contents. These tests were conducted on a representative bulk soil sample, taken from shallow depth, in general accordance with California Test Methods 417 and 422. The test results are summarized in the following table and also presented in this appendix.

Sample Location	Soil Descriptions	Sulfate (ppm)	Chloride (ppm)
B-2 @ 0 – 5 ft.	Silty fine Sand w/ little clay (SM)	134	115

ZS Engineering
113 Tomato Springs
Irvine, CA 92618
949-331-3232 | info@zs-engineering.com

EXPANSION INDEX TEST RESULTS (ASTM D4829)

Project Name: New Condominium Complex
1122 N. Bewley Street, Santa Ana, CA 92703

Project No: 200101

Sample Date: 1/18/2020

Test Date: 1/21/2020

Test Method: ASTM D4829

Sample Location	Soil Descriptions	Expansion Index	Expansion Potential
B-2 @ 0 - 5 ft.	Silty fine Sand w/ little clay (SM)	11	Very Low

ZS Engineering

113 Tomato Springs, Irvine, CA 92618
(949) 331-3232 | info@zs-engineering.com

DIRECT SHEAR TEST RESULTS (ASTM D3080)

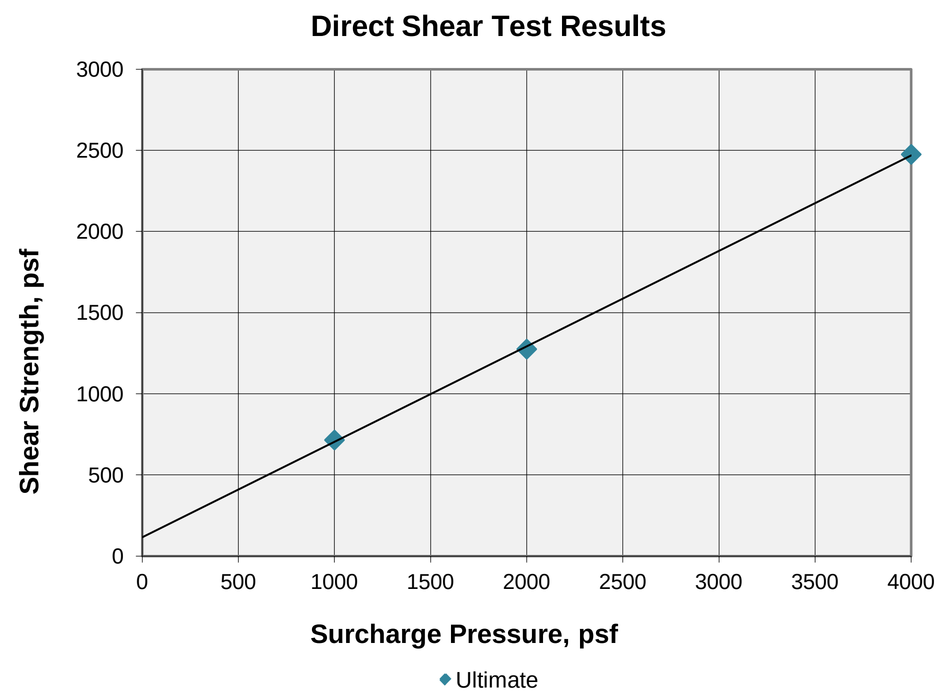
Project: New Condominium Complex
1122 N. Bewley Street, Santa Ana, CA 92703

Test Date: January 20, 2020 **ZS Engineering #:** 200101

Sample Source: B-2 @ 5 ft. **Undisturbed_X__Remolded____**

Visual Classification: Fine Sand w/ few silts (SP-SM) **Initial dry density:** 92.6 pcf

Method of Test: ASTM D3080 **Initial moisture content:** 8.6%



<u>Test Results :</u>

	Ultimate
Friction Angle, ϕ :	30.5 deg
Cohesion, c :	115 psf

ANAHEIM TEST LAB, INC

196 Technology Drive, Unit D
Irvine, CA 92618
Phone (949) 336-6544

DATE: 1/21/20

ZS ENGINEERING
113 TOMATO SPRINGS
IRVINE, CA 92618

P.O. NO. TRANSMITTAL

LAB NO. B-7693

SPECIFICATION: CA-417/422

ATTN: ZAFAR AHMED, P.E.

MATERIAL: Silty Sand, little clay

ZS Engineering #191202

Condominium Complex
1122 N. Bewley Street, Santa Ana, CA
B-2 @ 0 - 5 ft.

ANALYTICAL REPORT

CORROSION SERIES
SUMMARY OF DATA

SOLUBLE SULFATES per CA. 417 ppm	SOLUBLE CHLORIDES per CA. 422 ppm
134	115

RESPECTFULLY SUBMITTED

WES BRIDGER LAB MANAGER

APPENDIX C
Liquefaction and Seismic Settlement Analysis

Liquefaction and Seismic Settlement Analysis

In order to evaluate the of liquefaction (below groundwater) and seismic densification (above groundwater) potential of the subsurface soils, we conducted a settlement analysis utilizing a computer program LiquefyPro (CivilTech, 2009), following the guidelines of the SCEC and CGS Special Publication 117A (SCEC, 1999 and CGS, 2008a). The following input parameters were used in our analysis:

- For depths up to 25 feet, the most conservative field N-values at each depth from bore holes B-1 to B-3 are considered. Beyond 25 feet and up to 50 feet, field SPT N-values are taken from the deep bore hole B-2. Field N-values from modified California sampler within upper 10 feet were correlated with SPT N-values using a conversion factor of 0.63, as suggested in the guidelines (SCEC, 1999).

- Physical properties of subsurface soils (unit weight, fine contents) are obtained from the laboratory test results as well as our assumptions based on visual observation of the soil samples.

- Following correction factors were applied to SPT N-values: hammer energy ratio C_E of 1.3 for calibrated automatic trip hammer; borehole diameter correction factor C_B of 1.15 for 8-inch diameter borehole; and sampling method correction factor C_S of 1.2 for the SPT samplers without any inner liner or groove for liner.

- Maximum Moment, Mw, of 6.8 as shown in the deaggregated magnitude-distance plot in the state's seismic hazard zones report for the Anaheim Quadrangle (CGS, 1997);

- Peak ground acceleration (PGA) of 0.622g per Section 1803.5.12 of the 2019 CBC and Section 11.8.3 of ASCE 7-16; and

- Groundwater was encountered at depths varying from 19 to 20 feet below grade during this field exploration. Historic shallow groundwater level at the project site is within the contours of 5 and 10 feet below grade as shown in the state's seismic hazard zones report (CGS, 1997). As a conservative estimate, groundwater at a historic high level of 5 feet below grade is considered in the analysis.

- Cyclic Stress Ratio (CSR) to represent the anticipated field earthquake excitation (cyclic loading) was determined by Seed's Method (See and Idriss, 1971), which was adopted in the NCEER Proceeding and the subsequent SCEC guidelines (see References). A Factor of Safety value 1.3 is considered for CSR in compliance with the CGS Special Publication 117A.

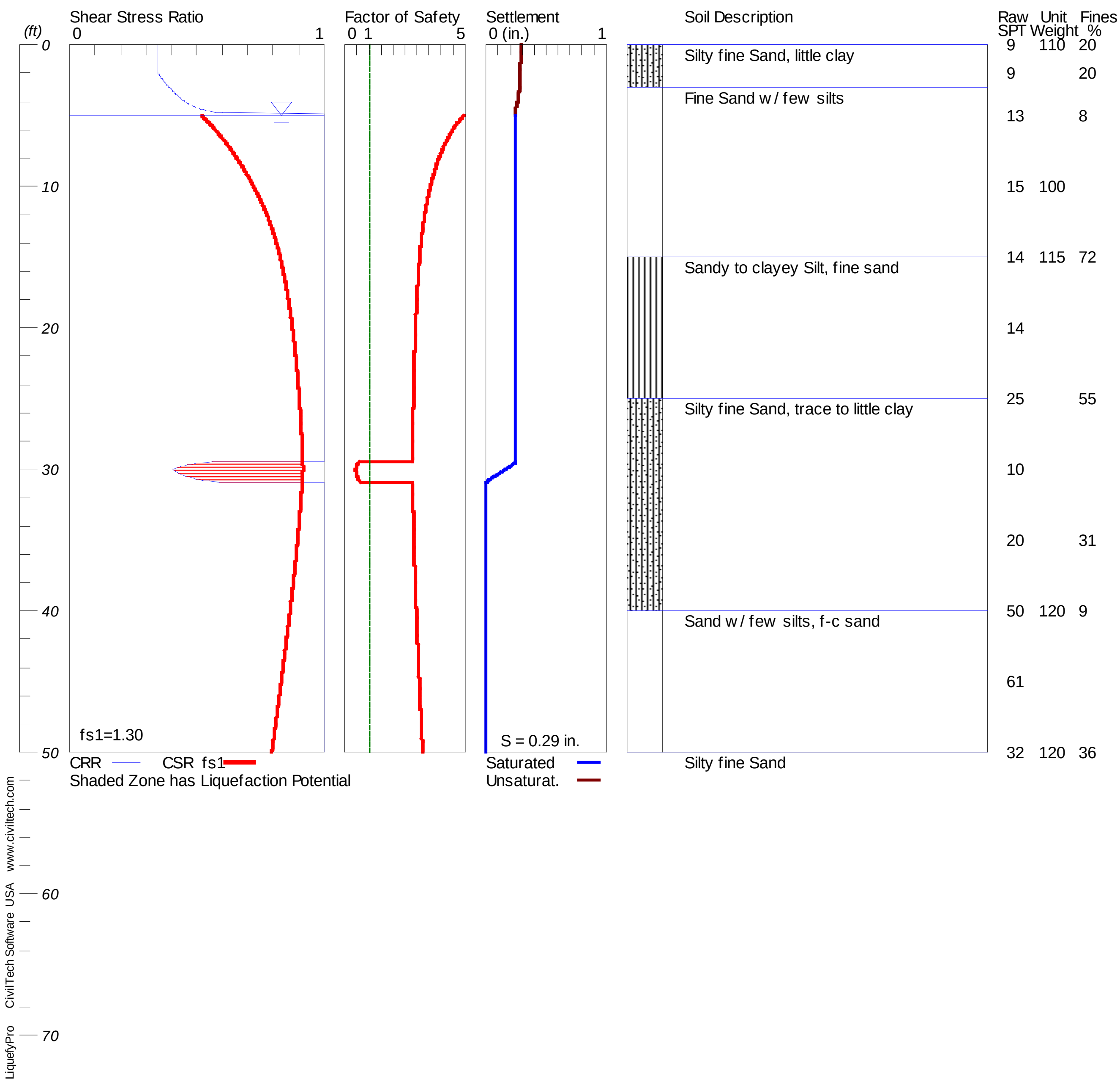
Our evaluation results indicate that maximum seismic settlement at the site will be on the order of 0.28 inch. A potentially liquefiable soil layer, about 2 feet thick, was identified at depths from about 29 to 31 feet below the existing grade. Graphical plot showing seismic settlement profile and the analysis results summary are presented in this appendix.

Liquefaction & Seismic Settlement Analysis

Condominium Complex - 1122 N. Bewley Street, Santa Ana, CA

Hole No.= Water Depth=5 ft Surface Elev.=80 ft

Magnitude=6.8
Acceleration=0.622g



ZS Engineering

****
LIQUEFACTION ANALYSIS SUMMARY

****
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Input File Name: D:\Consulting Projects\200101 - Multi-Family Apt - 1122 Bewley St, Santa Ana\SeismicSettle.liq
Title: Condominium Complex - 1122 N. Bewley Street, Santa Ana, CA
Subtitle:

Surface Elev.=80 ft
Hole No.=
Depth of Hole= 50.00 ft
Water Table during Earthquake= 5.00 ft
Water Table during In-Situ Testing= 5.00 ft
Max. Acceleration= 0.62 g
Earthquake Magnitude= 6.80

Input Data:
Surface Elev.=80 ft
Hole No.=
Depth of Hole=50.00 ft
Water Table during Earthquake= 5.00 ft
Water Table during In-Situ Testing= 5.00 ft
Max. Acceleration=0.62 g
Earthquake Magnitude=6.80
No-Liquefiable Soils: Based on Analysis

1. SPT or BPT Calculation.
2. Settlement Analysis Method: Ishihara / Yoshimine
3. Fines Correction for Liquefaction: Stark/Olson et al.*
4. Fine Correction for Settlement: During Liquefaction*
5. Settlement Calculation in: All zones*
6. Hammer Energy Ratio, Ce = 1.3
7. Borehole Diameter, Cb= 1.15
8. Sampling Method, Cs= 1.2
9. User request factor of safety (apply to CSR) , User= 1.3
   Plot one CSR curve (fs1=User)
10. Use Curve Smoothing: Yes*
* Recommended Options

In-Situ Test Data:

Depth ft	SPT	gamma pcf	Fines %
0.00	9.00	110.00	20.00
2.00	9.00	110.00	20.00
5.00	13.00	110.00	8.00
10.00	15.00	100.00	8.00
15.00	14.00	115.00	72.00
20.00	14.00	115.00	72.00
25.00	25.00	115.00	55.00
30.00	10.00	115.00	55.00
35.00	20.00	115.00	31.00

40.00	50.00	120.00	9.00
45.00	61.00	120.00	9.00
50.00	32.00	120.00	36.00

Output Results:

Settlement of Saturated Sands=0.24 in.
Settlement of Unsaturated Sands=0.04 in.
Total Settlement of Saturated and Unsaturated Sands=0.29 in.
Differential Settlement=0.144 to 0.191 in.

Depth ft	CRRm	CSRfs	F.S.	S_sat. in.	S_dry in.	S_all in.
0.00	0.35	0.53	5.00	0.24	0.04	0.29
0.50	0.35	0.52	5.00	0.24	0.04	0.29
1.00	0.35	0.52	5.00	0.24	0.04	0.29
1.50	0.35	0.52	5.00	0.24	0.04	0.28
2.00	0.35	0.52	5.00	0.24	0.04	0.28
2.50	0.37	0.52	5.00	0.24	0.04	0.28
3.00	0.39	0.52	5.00	0.24	0.03	0.28
3.50	0.42	0.52	5.00	0.24	0.03	0.27
4.00	0.45	0.52	5.00	0.24	0.02	0.26
4.50	0.51	0.52	5.00	0.24	0.00	0.25
5.00	2.57	0.52	4.95	0.24	0.00	0.24
5.50	2.57	0.55	4.70	0.24	0.00	0.24
6.00	2.57	0.57	4.49	0.24	0.00	0.24
6.50	2.57	0.60	4.31	0.24	0.00	0.24
7.00	2.57	0.62	4.16	0.24	0.00	0.24
7.50	2.57	0.64	4.03	0.24	0.00	0.24
8.00	2.57	0.66	3.91	0.24	0.00	0.24
8.50	2.57	0.67	3.81	0.24	0.00	0.24
9.00	2.57	0.69	3.71	0.24	0.00	0.24
9.50	2.57	0.71	3.63	0.24	0.00	0.24
10.00	2.57	0.72	3.55	0.24	0.00	0.24
10.50	2.57	0.74	3.48	0.24	0.00	0.24
11.00	2.57	0.75	3.42	0.24	0.00	0.24
11.50	2.57	0.76	3.36	0.24	0.00	0.24
12.00	2.57	0.78	3.31	0.24	0.00	0.24
12.50	2.57	0.79	3.27	0.24	0.00	0.24
13.00	2.57	0.80	3.23	0.24	0.00	0.24
13.50	2.57	0.80	3.19	0.24	0.00	0.24
14.00	2.57	0.81	3.16	0.24	0.00	0.24
14.50	2.57	0.82	3.13	0.24	0.00	0.24
15.00	2.57	0.83	3.11	0.24	0.00	0.24
15.50	2.57	0.83	3.08	0.24	0.00	0.24
16.00	2.57	0.84	3.06	0.24	0.00	0.24
16.50	2.57	0.85	3.04	0.24	0.00	0.24
17.00	2.57	0.85	3.02	0.24	0.00	0.24
17.50	2.57	0.86	3.00	0.24	0.00	0.24
18.00	2.57	0.86	2.99	0.24	0.00	0.24
18.50	2.57	0.86	2.97	0.24	0.00	0.24
19.00	2.57	0.87	2.96	0.24	0.00	0.24
19.50	2.57	0.87	2.95	0.24	0.00	0.24
20.00	2.57	0.88	2.93	0.24	0.00	0.24
20.50	2.57	0.88	2.92	0.24	0.00	0.24
21.00	2.57	0.88	2.91	0.24	0.00	0.24
21.50	2.57	0.89	2.90	0.24	0.00	0.24
22.00	2.57	0.89	2.89	0.24	0.00	0.24
22.50	2.57	0.89	2.88	0.24	0.00	0.24
23.00	2.57	0.89	2.87	0.24	0.00	0.24
23.50	2.57	0.90	2.87	0.24	0.00	0.24
24.00	2.57	0.90	2.86	0.24	0.00	0.24

24.50	2.57	0.90	2.85	0.24	0.00	0.24
25.00	2.57	0.90	2.85	0.24	0.00	0.24
25.50	2.57	0.90	2.84	0.24	0.00	0.24
26.00	2.57	0.91	2.83	0.24	0.00	0.24
26.50	2.57	0.91	2.83	0.24	0.00	0.24
27.00	2.57	0.91	2.82	0.24	0.00	0.24
27.50	2.57	0.91	2.82	0.24	0.00	0.24
28.00	2.57	0.91	2.81	0.24	0.00	0.24
28.50	2.57	0.91	2.81	0.24	0.00	0.24
29.00	2.57	0.92	2.81	0.24	0.00	0.24
29.50	0.56	0.92	0.61*	0.24	0.00	0.24
30.00	0.40	0.92	0.44*	0.16	0.00	0.16
30.50	0.46	0.92	0.50*	0.07	0.00	0.07
31.00	2.57	0.91	2.81	0.00	0.00	0.00
31.50	2.57	0.91	2.82	0.00	0.00	0.00
32.00	2.57	0.91	2.82	0.00	0.00	0.00
32.50	2.57	0.91	2.83	0.00	0.00	0.00
33.00	2.57	0.91	2.84	0.00	0.00	0.00
33.50	2.57	0.90	2.84	0.00	0.00	0.00
34.00	2.57	0.90	2.85	0.00	0.00	0.00
34.50	2.57	0.90	2.86	0.00	0.00	0.00
35.00	2.57	0.90	2.87	0.00	0.00	0.00
35.50	2.57	0.89	2.88	0.00	0.00	0.00
36.00	2.57	0.89	2.88	0.00	0.00	0.00
36.50	2.57	0.89	2.89	0.00	0.00	0.00
37.00	2.57	0.89	2.90	0.00	0.00	0.00
37.50	2.57	0.88	2.91	0.00	0.00	0.00
38.00	2.57	0.88	2.92	0.00	0.00	0.00
38.50	2.57	0.88	2.93	0.00	0.00	0.00
39.00	2.57	0.87	2.94	0.00	0.00	0.00
39.50	2.57	0.87	2.95	0.00	0.00	0.00
40.00	2.57	0.87	2.96	0.00	0.00	0.00
40.50	2.57	0.86	2.98	0.00	0.00	0.00
41.00	2.57	0.86	2.99	0.00	0.00	0.00
41.50	2.57	0.86	3.00	0.00	0.00	0.00
42.00	2.57	0.85	3.01	0.00	0.00	0.00
42.50	2.57	0.85	3.03	0.00	0.00	0.00
43.00	2.57	0.85	3.04	0.00	0.00	0.00
43.50	2.57	0.84	3.05	0.00	0.00	0.00
44.00	2.57	0.84	3.06	0.00	0.00	0.00
44.50	2.57	0.84	3.08	0.00	0.00	0.00
45.00	2.57	0.83	3.09	0.00	0.00	0.00
45.50	2.57	0.83	3.10	0.00	0.00	0.00
46.00	2.57	0.82	3.12	0.00	0.00	0.00
46.50	2.57	0.82	3.13	0.00	0.00	0.00
47.00	2.57	0.82	3.14	0.00	0.00	0.00
47.50	2.57	0.81	3.16	0.00	0.00	0.00
48.00	2.57	0.81	3.17	0.00	0.00	0.00
48.50	2.57	0.81	3.19	0.00	0.00	0.00
49.00	2.57	0.80	3.20	0.00	0.00	0.00
49.50	2.57	0.80	3.22	0.00	0.00	0.00
50.00	2.57	0.79	3.23	0.00	0.00	0.00

* F.S.<1, Liquefaction Potential Zone
(F.S. is limited to 5, CRR is limited to 2,   CSR is limited to 2)

Units: Unit: qc, fs, Stress or Pressure = atm (1.0581tsf); Unit Weight = pcf; Depth = ft; Settlement = in.

1 atm (atmosphere) = 1 tsf (ton/ft2)

CRRm   Cyclic resistance ratio from soils
CSRsf   Cyclic stress ratio induced by a given earthquake (with user

```
request factor of safety)
        F.S.            Factor of Safety against liquefaction, F.S.=CRRm/CSRsf
        S_sat           Settlement from saturated sands
        S_dry           Settlement from Unsaturated Sands
        S_all           Total Settlement from Saturated and Unsaturated Sands
        NoLiq           No-Liquefy Soils
```