# Geotechnical Investigation

Riverfront Apartments Project
Santa Cruz, California

Report No. 252341 has been prepared for:
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# SANTA CRUZ, CALIFORNIA

## 1.0 INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed Riverfront Apartments project to be constructed in Santa Cruz, California. The site location is shown on the Vicinity Map, Figure 1. The purpose of our investigation was to evaluate the geologic and subsurface conditions and to provide geotechnical recommendations for design of the proposed project. For our use, we received the following:

- Conceptual plans titled, "Riverfront Apartments, Santa Cruz, California," prepared by Jon Worden Architects dated January 6, 2016.

- Site plan titled, "Topographic Map and Boundary Survey," prepared by Bowman & Williams dated February 1, 2016.

### 1.1 Project Description

As currently planned, the project will consist of the construction of a 6 level apartment development with roof deck and one level of partially below grade parking. The nearly 1¼-acre approximately rectangular shaped site is bounded by Front Street to the west, the San Lorenzo River to the east and commercial developments to the north and south. Additional improvements will include pavements, underground utilities, and landscaping. The site is currently occupied by commercial properties. The layout of the proposed development is shown on the Site Plan, Figure 2.

Structural loads have not been provided to us; therefore we assumed that structural loads will be representative for this type of construction.

### 1.2 Scope of Services

Our scope of services was presented in our agreement with you dated February 17, 2016. To accomplish this work, we provided the following services:

- Exploration of subsurface conditions by drilling two borings in the area of the proposed development and retrieving soil samples for observation and laboratory testing. We also advanced three Cone Penetration Tests (CPTs).

- Evaluation of the physical and engineering properties of the subsurface soils by visually classifying the samples and performing various laboratory tests on selected samples.

- Engineering analysis to evaluate structure foundations, site earthwork, slabs-on-grade and pavements.

- Preparation of this report to summarize our findings and to present our conclusions and recommendations.

## 2.0 SITE CONDITIONS

### 2.1 Site Reconnaissance

Our Senior Staff Engineer performed a reconnaissance of the site on March 3, 2016. At the time of the reconnaissance, the site was occupied by existing commercial buildings with asphalt concrete parking lots and appeared relatively flat with minor grade variation for drainage purposes.

### 2.2 Exploration Program

Subsurface exploration was performed on March 3, 2016 using conventional, hollow-stem auger truck-mounted drilling equipment and CPT equipment to investigate, sample, and log subsurface soils. Two hollow-stem auger exploratory borings were drilled to refusal depths of 27 and 41½ feet. Three CPTs were advanced to refusal depths ranging from approximately 19 to 73½ feet. The borings and CPTs were backfilled in accordance with the Santa Cruz County guidelines. The approximate locations of the borings and CPTs are shown on the Site Plan, Figure 2. The logs of the borings and details regarding our field investigation are included in Appendix A; laboratory tests are discussed in Appendix B.

### 2.3 Subsurface Conditions

Borings EB-1 and EB-2 encountered pavement sections consisting of 4½ and 1 inches of asphalt concrete (AC), respectively. The AC in boring EB-1 was underlain by 1½ inches of aggregate base (AB). No AB was encountered below the AC in boring EB-2. Below the pavement section, our borings encountered interbedded layers of loose to medium dense silty sand, and loose to very dense poorly graded sand to a depth of approximately 25 to 26 feet. Below 25 to 26 feet, the borings encountered hard sandy silt with blow counts greater than 50 blows per foot. The hard sandy silt appears to be bedrock of the Purisima Formation. The Purisima Formation is described by Brabb (1997) as yellowish-gray tuffaceous and diatomaceous siltstone containing interbeds of bluish-gray, semifriable, fine grained andesitic sandstone.

The borings encountered refusal at depths of 41½ and 27 feet (EB-1 and EB-2, respectively). Sieve analysis on samples of the silty sand indicated 16 and 19 percent passing the #200 sieve.

Soils encountered in the CPTs were interpreted to include interbedded layers consisting of poorly graded sand, silty sand, and sandy silt with occasional thin interbedded layers of sand, silty clay, clayey silt, and clay. The CPT interpretations were generally consistent with our experience in the area, and the conditions encountered in our borings. Granular layers encountered were interpreted as loose to very dense.

### 2.4 Ground Water

Free ground water was encountered during subsurface exploration at depths of approximately 9½ and 8½ feet (EB-1 and EB-2, respectively). Based on pore pressure dissipation measurements, the CPTs encountered groundwater at depths ranging from approximately 7¾ to 12¼ feet below grade. The ground water depth was measured at the time of drilling and may not reflect a stabilized level. The California Geological Survey (CGS) has not mapped the minimum depth to ground water. Therefore, based on our subsurface exploration, we judge a design ground water depth of 5 feet to be appropriate for liquefaction purposes. All borings were backfilled immediately after drilling. Fluctuations in the level of the ground water may occur due to variations in rainfall, underground drainage patterns, tidal and river levels, and other factors not evident at the time we performed our explorations.

## 3.0 GEOLOGIC HAZARDS

A brief qualitative evaluation of geologic hazards was made during this investigation. Our comments concerning these hazards are presented below.

## 3.1 Fault Rupture

The San Francisco Bay Area is one of the most seismically active regions in the United States. The significant earthquakes that occur in the Bay Area are generally associated with crustal movement along well-defined active fault zones of the San Andreas Fault system, which regionally trend in a northwesterly direction. A Regional Fault Map illustrating known active faults relative to the site is presented in Figure 3. The site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone (known formerly as a Special Studies Zone). A Regional Fault Map illustrating known active faults relative to the site is presented in Figure 3. As shown on Figure 3, no known surface expression of active faults is believed to cross the site. Fault rupture through the site, therefore, is not anticipated.

## 3.2 Maximum Estimated Ground Shaking

Based on Equation 11.8-1 of ASCE 7-10, we judge a maximum considered earthquake geometric mean peak ground acceleration of 0.50g to be appropriate for geotechnical analyses.

## 3.3 Future Earthquake Probabilities

Although research on earthquake prediction has greatly increased in recent years, seismologists cannot predict when or where an earthquake will occur. The U.S. Geological Survey’s Working Group on California Earthquake Probabilities (WGCEP, 2014) estimates there is a 72 percent chance of at least one magnitude 6.7 earthquake occurring in the San Francisco Bay region between 2014 and 2044. This result is an important outcome of WGCEP’s work because any major earthquake can cause damage throughout the region. The 1989 Loma Prieta earthquake demonstrated this potential by causing severe damage in Oakland and San Francisco, more than 50 miles from the fault epicenter.

Although earthquakes can cause damage at a considerable distance, shaking will be very intense near the fault rupture. Therefore, earthquakes located in urbanized areas of the region have the potential to cause much more damage than the 1989 Loma Prieta earthquake.

## 3.4 Liquefaction

### 3.4.1 General Background

The site is not located within an area that has been mapped by the CGS for seismically induced liquefaction hazards. The site is located within an area zoned in the Santa Cruz County Geologic Hazard Maps as having very high susceptibility to soil liquefaction. During cyclic ground shaking, such as earthquakes, cyclically-induced stresses may cause increased pore water pressures within the soil matrix, which results in liquefaction. Liquefied soil may lose shear strength that may lead to large shear deformations and/or flow failure (Youd et al., 2001). Liquefied soil can also settle as pore pressures dissipate following an earthquake. Limited field data is available on this subject; however, settlement on the order of 2 to 3 percent of the thickness of the liquefied zone has been measured in some cases.

Soils most susceptible to liquefaction are loose to moderately dense, saturated, non-cohesive soils with poor drainage, such as sands and silts with interbedded or capping layers of relatively low permeability soil.

### 3.4.2 Analysis and Results

Based on our explorations, a design ground water level at 5 feet below the existing site grade was used for our liquefaction analysis. As discussed in the subsurface description above, several sand and silt layers were encountered below the design ground water depth. These layers were evaluated to assess liquefaction potential and the effects liquefaction may have on the proposed structures. No liquefaction analyses were performed on layers above the design ground water depth. Table 1 includes estimated liquefaction settlement in the upper 50 feet; the current methods for estimating liquefaction settlement are generally applicable for the upper 50 feet and the effects of liquefaction settlement below 50 feet on the proposed structures should be minimal.

Our liquefaction analysis followed the methods presented by the 1998 NCEER Workshops (Youd et al., 2001) in accordance with guidelines set forth in the CGS Special Publication 117A (2008). The NCEER methods for CPT analysis update simplified procedures presented by Seed and Idriss (1971). In broad terms, these methods are used to calculate a factor of safety against liquefaction triggering by comparing the resistance of the soil to cyclic shaking to the seismic demand that can be caused during seismic events.

The resistance to cyclic shaking is quantified by the Cyclic Resistance Ratio (CRR), which is a function of soil density, layer depth, ground water depth, earthquake magnitude, and soil behavior. Our CPT tip pressures were corrected for the overburden and fines content. The CPT method utilizes the soil behavior type index ($I_C$) and the exponential factor “n” applied to the Normalized Cone Resistance “Q” to evaluate how plastic the soil behaves. The Cyclic Stress Ratio (CSR) is used to quantify the stresses that are anticipated to develop during cyclic shaking. The formula for CSR is shown below:

$$CSR = 0.65\ (a_{max}/g)(\sigma_{vo}/\sigma'_{vo})r_d$$

Where $a_{max}$ is the peak horizontal acceleration at the ground surface generated by an earthquake, g is the acceleration of gravity, $\sigma_{vo}$ and $\sigma'_{vo}$ are total and effective overburden stresses, respectively, and $r_d$ is a stress reduction coefficient. We evaluated the liquefaction potential of the medium dense sand and silt strata encountered below the design ground water depth using a peak ground acceleration of 0.50g (based on Equation 11.8-1 of ASCE 7-10) and moment magnitude of 8.0 (USGS 2008).

The factor of safety (FS) against liquefaction can be expressed as the ratio of the CRR to CSR. If the FS for a soil layer is less than 1.0, the soil layer is considered liquefiable during a moderate to large seismic event.

$$FS = CRR/CSR$$

Soils that have $I_c$ greater than 2.6 or CPT tip resistance greater than 160 tons per square foot (tsf) are considered either too plastic or too dense to liquefy, respectively. Such soil layers have been screened out of the analysis and are not presented below. A summary of our CPT analysis is presented in Table 1 below.

Table 1. Results of Liquefaction Analyses – CPT Method

| CPT Number | Depth to Top of Sand/Silt Layer (feet) | Layer Thickness (feet) | $I_C$ | *$q_{C1N}$ (tsf) | Factor of Safety | Potential for Liquefaction | Estimated Total Settlement (in.) |
|---|---|---|---|---|---|---|---|
| CPT-1 | 5.1 | 0.5 | 2.5 | 79.1 | 0.4 | Likely | 0.2 |
| | 5.7 | 3.4 | 2.0 | 91.8 | 0.3 | Likely | 1.1 |
| | 9.7 | 3.6 | 1.7 | 116.1 | 0.4 | Likely | 0.9 |
| | 14.3 | 7.2 | 1.8 | 106.8 | 0.4 | Likely | 2.0 |
| | 22.2 | 1.3 | 1.6 | 144.4 | 0.6 | Likely | 0.3 |
| | 24.6 | 11.3 | 1.8 | 113.7 | 0.4 | Likely | 2.9 |
| | 37.6 | 0.8 | 1.6 | 140.9 | 0.6 | Likely | 0.2 |
| | 40.5 | 1.2 | 1.6 | 140.9 | 0.6 | Likely | 0.2 |
| | 42.2 | 2.0 | 1.7 | 120.5 | 0.4 | Likely | 0.5 |
| | 44.8 | 2.1 | 1.8 | 107.2 | 0.4 | Likely | 0.6 |
| | 47.4 | 1.0 | 2.4 | 67.6 | 0.2 | Likely | 0.4 |
| | 48.9 | 1.2 | 1.9 | 92.0 | 0.3 | Likely | 0.4 |
| | 50.0** | 10.3 | 2.0 | 90.4 | 0.3 | Likely | 3.3 |
| | 62.7** | 6.9 | 2.1 | 99.2 | 0.4 | Likely | 1.9 |
| | 70.2** | 0.7 | 2.4 | 148.1 | 0.7 | Likely | 0.1 |
| | 71.7** | 0.5 | 2.5 | 147.5 | 0.7 | Likely | 0.1 |
| | 73.0** | 0.5 | 2.3 | 113.7 | 0.4 | Likely | 0.1 |
| | | | | | | Total = | 15.2 |
| CPT-2 | 5.1 | 11.8 | 1.9 | 90.4 | 0.3 | Likely | 3.9 |
| | 17.6 | 3.4 | 1.7 | 111.6 | 0.3 | Likely | 0.9 |
| | 21.5 | 2.1 | 1.6 | 134.5 | 0.5 | Likely | 0.5 |
| | 24.4 | 3.9 | 1.6 | 139.2 | 0.5 | Likely | 0.8 |
| | | | | | | Total = | 6.1 |
| CPT-3 | 5.1 | 8.9 | 1.9 | 90.5 | 0.3 | Likely | 2.8 |
| | 14.8 | 6.2 | 1.8 | 120.0 | 0.4 | Likely | 1.5 |
| | 22.8 | 1.8 | 2.4 | 94.6 | 0.3 | Likely | 0.6 |
| | | | | | | Total = | 4.9 |

* CPT tip pressure corrected for overburden and fines content

** Settlements below depths of 50 feet included for deep foundation evaluation, not included as part of total or differential settlements at ground surface for shallow foundations

The current methods for estimating liquefaction settlement are generally applicable for the upper 50 feet. The effects of liquefaction settlement below 50 feet on the proposed structures should be minimal. Settlements below a depth of 50 feet are not included as part of total or differential liquefaction induced settlements at the ground surface for the shallow foundation recommendations.

Our analyses indicate that several sand and silt layers below the design ground water depth may theoretically liquefy, resulting in approximately 5 to 9¾ inches of total settlement for the top 50 feet. Volumetric change and settlement were estimated using the Zhang, Robertson, and Brachman (2002) method. We estimate differential settlements from liquefaction will be on the order of ¾-inch in 50 horizontal feet. A detailed discussion of estimated settlements is presented in the "Foundations" section of this report.

### 3.4.3 Potential for Ground Rupture/Sand Boils

The methods of analysis used to estimate the total liquefaction induced settlement assume that there is no possibility of surface ground rupture. For liquefaction induced sand boils or fissures to occur, the pore water pressure induced within the liquefied strata must be large enough to break through the surface layer.

The bottom of the proposed partially below-grade structure is approximately 7 feet below the ground surface. There will be no non-liquefiable material overlying the potential liquefiable layers at the site after excavation for the parking level has been completed. Based on the work by Youd and Garris (1995), there is not an adequate non-liquefiable material capping for some of the liquefiable layers at the site. If ground rupture and sand venting were to occur, significantly higher ground deformation could occur. Therefore, we recommend the proposed structures be supported on either improved ground (to reduce the liquefaction potential) or deep foundations (to reduce the potential for impact to the structure due to ground rupture). Detailed recommendations are presented in subsequent sections of this report.

### 3.5 Dry Seismic Settlement

If near-surface soils vary in composition both vertically and laterally, strong earthquake shaking can cause non-uniform densification of loose to medium dense cohesionless soil strata. This results in movement of the near-surface soils. The soils above the design groundwater depth of 5 consist of loose silty sand and poorly graded sand. However, we understand the project will include a partially below grade parking level bearing below the design ground water depth of 5 feet. Therefore, we judge the probability of significant differential settlement of non-saturated granular layers at the site to be low.

### 3.6 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.

The San Lorenzo River is located approximately 100 feet east of the site. Loose to medium dense silty sand and poorly graded sand layers were encountered at depths ranging approximately from 5 to 73½ feet below the ground surface. These soil layers have a high potential for liquefaction. If liquefaction were to occur, the potential for lateral spreading would be moderate to high in localized areas. Lateral spreading could damage structures, pavements and underground utilities on the site. As mentioned above, we recommend soil improvement be performed for the area of the proposed structures. However, the recommended of soil improvement may not mitigate the potential for lateral spreading, and damage could occur to site improvements. The magnitude of lateral spreading movement is difficult to estimate accurately, but could be on the order of several feet based on an earthquake of moderate magnitude and duration and is judged to be possible in the upper 25 feet. Detailed recommendations for soil improvement are presented under the “Earthwork” section of this report.

### 3.7 Flooding

The site is located in FEMA flood Zone A99 (FEMA 2012) which is defined as an “areas to be protected from 1% annual chance flood event by a federal flood protection system under construction; No base flood elevations determined.”

## 3.8 Tsunami

The site is located in a recommended tsunami evacuation area (CGS 2009). The land at and around the project may become quickly flooded if there is a tsunami. The nearest high ground (area outside the evacuation area) is approximately 1,200 west of the project site.

## 4.0 CORROSION EVALUATION

To evaluate the corrosion potential of the subsurface soils at the site, we submitted two samples collected during our subsurface investigation to an analytical laboratory for pH, resistivity, soluble sulfate and chloride content testing. The results of these tests are summarized in Table 2 below.

Table 2. Results of Corrosivity Testing

| Sample | Depth (feet) | Chloride (mg/kg) | Sulfate (mg/kg) | pH | Resistivity (ohm-cm) | Estimated Corrosivity Based on Resistivity | Estimated Corrosivity Based on Sulfates |
|---|---|---|---|---|---|---|---|
| EB-1 | 2.5 | <2 | 77 | 7.7 | 9,374 | Mildly | Negligible |
| EB-1 | 4.5 | <2 | 68 | 7.9 | 12,478 | Very Mildly | Negligible |

Notes: 1. mg/kg = milligrams per kilogram.

Many factors can affect the corrosion potential of soil including soil moisture content, resistivity, permeability and pH, as well as chloride and sulfate concentration. In general, soil resistivity, which is a measure of how easily electrical current flows through soils, is the most influential factor. Based on classification developed by William J. Ellis (1978), the approximate relationship between soil corrosiveness was developed as shown in Table 3 below.

Table 3. Relationship Between Soil Resistivity and Soil Corrosivity

| Soil Resistivity (ohm-cm) | Classification of Soil Corrosiveness |
|---|---|
| 0 to 900 | Very Severely Corrosive |
| 900 to 2,300 | Severely Corrosive |
| 2,300 to 5,000 | Moderately Corrosive |
| 5,000 to 10,000 | Mildly Corrosive |
| 10,000 to >100,000 | Very Mildly Corrosive |

Chloride and sulfate ion concentrations and pH appear to play secondary roles in affecting corrosion potential. High chloride levels tend to reduce soil resistivity and break down otherwise protective surface deposits, which can result in corrosion of buried metallic improvements or reinforced concrete structures. Sulfate ions in the soil can lower the soil resistivity and can be highly aggressive to Portland cement concrete (PCC) by combining chemically with certain constituents of the concrete, principally tricalcium aluminate. This reaction is accompanied by expansion and eventual disruption of the concrete matrix. Soils containing high sulfate content could also cause corrosion of the reinforcing steel in concrete. Table 4.2.1 of the American Concrete Institute (ACI, 2008) provides requirements for concrete exposed to sulfate-containing solutions as summarized in Table 4.

Table 4. Relationship Between Sulfate Concentration and Sulfate Exposure

| Water-Soluble Sulfate ($SO_4$) in soil, ppm | Sulfate Exposure |
|---|---|
| 0 to 1,000 | Negligible |
| 1,000 to 2,000 | Moderate[1] |
| 2,000 to 20,000 | Severe |
| over 20,000 | Very Severe |

[1] = seawater

Acidity is an important factor of soil corrosivity. The lower the pH (the more acidic the environment), the higher will the soil corrosivity be with respect to buried metallic structures. As soil pH increases above 7 (the neutral value), the soil is increasingly more alkaline and less corrosive to buried steel structures due to protective surface films which form on steel in high pH environments. A pH between 5 and 8.5 is generally considered relatively passive from a corrosion standpoint.

As shown in Table 2, the soil resistivity results were 9,374 and 12,478 ohm-centimeters. Based on these results and the resistivity correlations presented in Table 3, the corrosion potential to buried metallic improvements may be characterized as moderately to mildly to very mildly corrosive. A corrosion protection engineer may be consulted about appropriate corrosion protection methods for buried metallic materials.

Based on our previous experience and Table 4.2.1 of the ACI, it is our opinion that sulfate exposure to PCC may be considered negligible for the native subsurface materials sampled.

## 5.0 CONCLUSIONS AND RECOMMENDATIONS

From a geotechnical engineering viewpoint, the proposed structure may be constructed as planned, in our opinion, provided the design and construction are performed in accordance with the recommendations presented in this report.

### 5.1 Primary Geotechnical Concerns

The primary geotechnical and geologic concerns at the site are as follows:

- Strong seismic shaking
- Liquefaction, ground rupture, and lateral spreading
- Shallow ground water
- Differential settlement between below-grade and at-grade portions of the structure
- Differential settlement at utilities tie-ins
- Demolition of the existing buildings prior to site development

We have prepared a brief description of the issues and present typical approaches to manage potential concerns associated with the long-term performance of the development.

### 5.1.1 Strong Seismic Shaking

We recommend that, at a minimum, the proposed structure be designed in accordance with the seismic design criteria as discussed in the Maximum Estimated Ground Shaking section above, and the site seismic coefficients presented in Table 6.

### 5.1.2 Liquefaction, Ground Rupture, and Lateral Spreading

Our analyses indicate that several layers theoretically can liquefy, ranging from 5 to 9¾ inches of total settlement in the upper 50 feet, with differential settlements from liquefaction on the order of ¾-inch in a horizontal distance of 50 feet. There is also a high potential for ground rupture to occur as there is not an adequate cap of non-liquefiable material overlying the liquefiable layers at the site. In addition, there is also high potential for liquefaction-induced lateral spreading to occur due to the close proximity of the San Lorenzo River. These phenomena could result in up to several feet of vertical and lateral movement of the ground surface.

We have identified two methods to reduce the potential for ground rupture below the foundations of the proposed structure. The mitigation measures include 1) deep soil cement-treating the upper approximately 30 feet of the on-site soils below the proposed partially one-level below grade excavation depth and 2) ground improvement consisting of rammed aggregate piers (RAPs) or stone columns to a depth of 30 feet. With both mitigation measures, the structure could be supported on reinforced mat foundations overlying the improved soils. It should be noted, however, that the vibration associated with RAPs or stone columns may cause densification (and resulting settlement) of nearby structures and improvements.

The two options will provide a densified block of soil which will reduce the potential for liquefaction and ground rupture, but will likely not totally eliminate it. It will not mitigate the potential for liquefaction-induced settlements below the treated zone which still could be on the order of up to 5 to 6 inches, nor will it reduce the potential for liquefaction-induced lateral spreading. The potential will still exist for differential settlement and significant tilting of the structure depending on the magnitude and duration of earthquake shaking.

Alternatively, the structure could be founded on deep foundations. The deep foundation option would not reduce the magnitude or potential for liquefaction, or ground rupture, but should be designed to resist lateral spreading. We would recommend that the garage slab would consist of a structural slab supported on the piles. Due to the varying depth to non-liquefiable soils below the site, deep foundation elements will be subject to varying magnitudes of downdrag and therefore may need to be differing lengths to achieve the design capacity.

### 5.1.3 Shallow Ground Water

As discussed in Section 2.4, ground water was encountered in our CPTs and exploratory borings at depths ranging from 8 to 12 feet below the existing ground surface. The proposed structure with a partial one-level of below grade parking should be designed to resist hydrostatic uplift pressures. Basement walls should be designed to resist hydrostatic pressure up to the design groundwater of 5 feet. The contractor should be aware that excavations/trenches extending near the ground water level may need to be stabilized and/or dewatered to facilitate placement of structures and/or placement and compaction of fill.

### 5.1.4 Differential Settlement between Below-grade and At-grade portions of the Structure

Based on the plans provided, we understand that the ramps into the partially one-level below-grade garage of the building will be within the footprint of the foundation. However, there may be differential settlement between the structurally supported ramps and walkways and adjacent flatwork. We recommend structurally supporting flatwork adjacent to the building for a span of at least 5 feet laterally from the building. A hinge slab or other method should be used to accommodate portions of the structures that will be supported on different materials.

#### 5.1.5 Differential Settlement for Utilities Tie-ins

The utilities entering the building could experience differential settlement specifically at the tie-in locations. We recommend emergency shut-off valves and flexible utility and piping connections that can accommodate at least two inches of movement.

#### 5.1.6 Demolition Debris

Construction debris is anticipated as a result of the site demolition required prior to site grading. The debris should be either: 1) collected and off-hauled to an appropriate facility prior to beginning the earthwork for the project, or 2) the concrete crushed and re-used as fill at the site. If generated, recycled materials containing AC should not be used below interior floor slabs, therefore if recycled materials are proposed to be re-used beneath interior floor slabs, AC pavements should be segregated from the debris. It has been our experience that some debris will remain in the soil on-site after the demolition contractor has completed their work. Therefore, it should be anticipated that some debris would be encountered in excavations for underground utilities and foundations. Some coordination between the demolition contractor, grading contractor and geotechnical engineer is needed to identify the scope of the excavation backfill and other similar work items. Recommendations for re-use of recycled materials are presented in the Earthwork section of this report.

### 5.2 Plans, Specifications, and Construction Review

We recommend that our firm perform a plan review of the geotechnical aspects of the project design for general conformance with our recommendations. In addition, subsurface materials encountered in the relatively small diameter, widely spaced borings and CPTs may vary significantly from other subsurface materials on the site. Therefore, we also recommend that a representative of our firm observe and confirm the geotechnical specifications of the project construction. This will allow us to form an opinion about the general conformance of the project plans and construction with our recommendations. In addition, our observations during construction will enable us to note subsurface conditions that may vary from the conditions encountered during our investigation and, if needed, provide supplemental recommendations. For the above reasons, our geotechnical recommendations are contingent upon our firm providing geotechnical observation and testing services during construction.

## 6.0 EARTHWORK

### 6.1 Clearing and Site Preparation

The proposed project area should be cleared of all surface and subsurface improvements to be removed and deleterious materials including existing building foundations, slabs, irrigation lines, utilities, fills, pavements, debris, designated trees, shrubs, and associated roots. Abandonment of existing buried utilities is discussed below. Excavations extending below the planned finished site grades should be cleaned and backfilled with suitable material compacted as recommended in the “Compaction” section of this report. We recommend that backfilling of holes or pits resulting from demolition and removal of existing building foundations, buried structures or other improvements be carried out under our observation and that the backfill be observed and tested during placement.

After clearing, any vegetated areas within the proposed improvements should be stripped to sufficient depth to remove all surface vegetation and topsoil containing greater than 3 percent organic matter by weight. The actual stripping depth required depends on site usage prior to construction and should be established in the field by us at the time of construction. The stripped materials should be removed from the site or may be stockpiled for use in landscaped areas, if desired.

## 6.2 Removal of Undocumented Fill

If undocumented fill is encountered, it should be removed down to the native soil. If the fill material meets the requirements in the "Material for Fill" section below, it may be reused an engineered fill. Side slopes of fill removal excavations in building and pavement areas should be sloped at inclinations no steeper than 3:1 (horizontal:vertical) to minimize abrupt variations in fill thickness. All fill should be compacted in accordance with the recommendations for fill presented in the "Compaction" section of this report.

## 6.3 Abandoned Utilities

Abandoned utilities within the proposed building area should be removed in their entirety. Utilities within the proposed building area would only be considered for in-place abandonment provided they do not conflict with new improvements, if the ends and all laterals are located and completely grouted, and the previous fills associated with the utility do not pose a risk to the structure.

Utilities outside the building area should be removed or abandoned in-place by grouting or plugging the ends with concrete. Fills associated with utilities abandoned in-place could pose some risk of settlement; utilities that are plugged could also pose some risk of future collapse or erosion should they leak or become damaged.

## 6.4 Reuse of On-site Recycled Materials

Some asphalt concrete/aggregate base grindings may be generated during removal of any existing pavements. If it is desired to reuse the grindings for new site pavement structural support, we recommend the asphalt concrete be pulverized and mixed with the underlying aggregate base to meet Caltrans Class 2 AB requirements. If laboratory testing of the recycled material indicates that it meets Caltrans Class 2 specifications, it may be used as Class 2 AB beneath pavements and sidewalks. Recycled material containing AC grindings should not be used below building areas. Laboratory testing may be performed on initial grindings generated to evaluate the material further and refine the pavement recommendations.

## 6.5 Ground Improvement Options

### 6.5.1 RAP or Stone Column Option

The proposed structure may be supported on reinforced mat foundations in conjunction with ground improvement consisting of RAPs or stone columns which can be installed to reduce the potential for liquefaction and the potential for ground rupture by densifying the potentially liquefiable soils. The RAPs or stone columns are constructed by pushing a mandrel or vibrator into the ground and withdrawing it as aggregate is placed and compacted in the void. The void is backfilled by compacting well-graded aggregate in lifts using a modified hydraulic hammer. Below the ground water table, where significant seepage occurs, clean crushed rock is used. This method pre-stresses the soil and provides increased soil density. Based on previous discussions with several design build contractors (Farrell Design-Build and Hayward Baker) for a nearby project, the following is an assumption for a probable scope of work. These assumptions need to be confirmed once the contractor is provided a set of project plans and structural loads to review and after discussion of the likely construction schedule with the general contractor. RAPs or stone columns would be installed throughout the foundation area of the proposed structure with an approximate spacing of 5 to 7 feet on center, and to a depth of approximately 30 feet.

The design-build contractor would work with TRC and the project structural engineer in designing the foundations and RAP or stone column layout. Following installation of the RAPs or stone columns, CPT exploration would be performed to confirm that the adjacent soils have been sufficiently densified such that they will not experience liquefaction.

We estimate that this ground improvement method would reduce liquefaction-induced settlements to about 3 inches, and reduce the potential for ground rupture below the reinforced mat foundations. The potential for lateral spreading within the structure would be improved, but the potential for some lateral movement will remain. Ground rupture outside of the treated area would remain likely during strong earthquake shaking.

### 6.5.2 Deep Soil Mixing Option

As an alternate to the RAPs or stone columns, the upper 30 feet below the proposed foundation bearing level could be improved by mixing cement into the soil. The method works by mixing a cementing agent into the existing soils by using radial mixing paddles mounted on the end of a drill string. The cementing agent is pumped as the drill string is withdrawn. The resulting soil-cement columns have increased shear-strength can be used to support building foundations. This will also provide a reinforced soil zone capable of reducing the differential lateral movements resulting from lateral spreading directly on the reinforced mat foundation and allowing the structure additional rigidity against pull apart loading. Column spacing would be determined by the design-build contractor.

We estimate that this ground improvement method would reduce liquefaction-induced settlements to approximately 5 to 6 inches, and reduce the potential for ground rupture directly below the mat foundation, but not entirely eliminate it.

Samples of the cement-treated soils should be obtained and tested to confirm the recommended unconfined compressive strength has been achieved.

## 6.6 Subgrade Preparation

After the site has been properly cleared, stripped and necessary excavations have been made, exposed surface soils in those areas to receive fill or pavements should be scarified to a depth of 6 inches, moisture conditioned, and compacted in accordance with the recommendations for fill presented in the "Compaction" section. The finished compacted subgrade should be firm and non-yielding under the weight of compaction equipment.

## 6.7 Material for Fill

All on-site soils below the stripped layer having an organic content of less than 3 percent by weight are suitable for use as fill at the site. In general, fill material should not contain rocks or lumps larger than 6 inches in greatest dimension, with 15 percent or less larger than 2½ inches in the greatest dimension.

Import fill and non-expansive fill (NEF) material should be inorganic, have a PI of 15 or less and should have sufficient binder to reduce the potential for sidewall caving of foundation and utility trenches. Samples of the proposed import fill should be submitted to us at least 10 working days prior to delivery to the site to allow for visual review and laboratory testing. This will allow us to evaluate the general conformance of the import fill with our recommendations.

Consideration should also be given to the environmental characteristics and corrosion potential of any imported fill. Suitable documentation should be provided for import material. In addition, it may be appropriate to perform laboratory testing of the environmental characteristics and corrosion potential of imported materials. Import soils should not be more corrosive than the on-site native materials, including pH, soluble sulfates, chlorides and resistivity.

### 6.8 Compaction

All fill, as well as scarified surface soils in those areas to receive fill, should be uniformly compacted to at least 90 percent relative compaction as determined by ASTM Test Designation D1557, latest edition, at a moisture content near the laboratory optimum. Fill should be placed in lifts no greater than 8 inches in uncompacted thickness. Each successive lift should be firm and relatively non-yielding under the weight of construction equipment.

In pavement areas, the upper 6 inches of subgrade and full depth of aggregate base should be compacted to at least 95 percent relative compaction (ASTM D1557, latest edition). Aggregate base and all import soils should be compacted at a moisture content near the laboratory optimum moisture content.

### 6.9 Wet Soils and Wet Weather Conditions

Earthwork such as subgrade preparation, fill placement and trench backfill may be difficult for soil containing high moisture content or during wet weather. If the soil is significantly above its optimum moisture content, it will become soft, yielding, and difficult to compact. Based on the results of our laboratory tests, the in-situ moisture contents of the near surface soils are generally near to above optimum moisture contents. If saturated soils are encountered, aerating or blending with drier soils to achieve a workable moisture content may be required. We recommend that earthwork be performed during periods of suitable weather conditions, such as the “summer” construction season.

There are several alternatives to facilitate subgrade preparation, fill placement and trench backfill if the soil is wet or earthwork is performed during the wet winter season.

- Scarify and air dry until the fill materials have a suitable moisture content for compaction,
- Over-excavate the fill and replace with suitable on-site or import materials with an appropriate moisture content,
- Install a layer of geo-synthetic (geotextile or geogrid) to reduce surface yielding and bridge over soft fill,
- Chemically treat the higher moisture content soils with quicklime (CaO), kiln-dust, or cement to reduce the moisture content and increase the strength of the fill.

The implementation of these methods should be reviewed on a case-by-case basis so that a cost effective approach may be used for the specific conditions at the time of construction.

### 6.10 Trench Backfill

Bedding and pipe embedment materials to be used around underground utility pipes should be well graded sand or gravel conforming to the pipe manufacturer's recommendations and should be placed and compacted in accordance with project specifications, local requirements of the governing jurisdiction. General fill to be used above pipe embedment materials should be placed and compacted in accordance with local requirements or the recommendations contained in this section, whichever is more stringent.

On-site soils may be used as general fill above pipe embedment materials provided they meet the requirements of the "Material for Fill" section of this report. General fill should be placed in lifts not exceeding 8 inches in uncompacted thickness and should be compacted to at least 90 percent relative compaction (ASTM D1557, latest edition) by mechanical means only. Water jetting of trench backfill should not be allowed. The upper 6 inches of general fill in all pavement areas subject to wheel loads should be compacted to at least 95 percent relative compaction.

Utility trenches located adjacent to footings should not extend below an imaginary 1:1 (horizontal:vertical) plane projected downward from the footing bearing surface to the bottom edge of the trench. Where utility trenches will cross beneath footing bearing planes, the footing concrete should be deepened to encase the pipe or the utility trench should be backfilled with sand/cement slurry or lean concrete within the foundation-bearing plane.

Where relatively higher permeability sand or gravel backfill is used in trenches through lower permeability soils, we recommend that a cut-off plug of compacted clayey soil or a 2-sack cement/sand slurry be placed where such trenches enter the building and pavement areas. This would reduce the likelihood of water entering the trenches from the landscaped areas and seeping through the trench backfill into the building and pavement areas, and coming into contact with very highly expansive subgrade soils.

### 6.11 Temporary Slopes and Trench Excavations

The contractor should be responsible for all temporary slopes and trenches excavated at the site and design of any required temporary shoring. Shoring, bracing, and benching should be performed by the contractor in accordance with the strictest governing safety standards. On a preliminary basis, site soils can be classified as Type C based on soil classification by OSHA. Therefore a maximum slope 1.5:1 (horizontal:vertical) should be anticipated. A TRC representative should be retained to verify soil conditions in the field at the time of the excavation.

### 6.12 Temporary Shoring Support System

As previously discussed, excavations on the order of approximately 7 feet are planned to construct the proposed partially one-level below-grade parking garage. Loose to medium dense sand and ground water should be expected. The excavations could potentially be temporarily supported by several methods including tiebacks, soil nailing, braced shoring, temporary slopes if space is adequate, or potentially other methods. Where shoring is required, restrained shoring will most likely be necessary to limit deflections and disruption to nearby improvements. It has been our experience that cantilever shoring might be feasible for temporary shoring to a height of about 10 to 13 feet where allowable deflections are limited. The choice of shoring method should be left to the contractor's judgment since economic considerations and/or the individual contractor's construction experience may determine which method is more economical and/or appropriate. However, other factors such as the location of nearby utilities and encroachment on adjacent properties may influence the choice of support.

The temporary shoring should be designed for additional surcharges due to adjacent loads such as from construction vehicles, street traffic and adjacent levee. To prevent excessive surcharging of the walls, we recommend that heavy loads such as construction equipment and stockpiles of materials be kept at least 15 feet from the top of the excavations. If this is not possible, the shoring must be designed to resist the additional anticipated lateral loads. Shoring systems should be designed with sufficient rigidity to prevent detrimental lateral displacements. Minimum geotechnical parameters for design of a temporary shoring system are given in Table 5.

Table 5. Temporary Shoring System Design Parameter

| Design Parameter | Design Value (psf) |
|---|---|
| Minimum Lateral Wall Surcharge[1] | 120 psf |
| Earth Pressure – Cantilever Wall | 40 pcf |
| Earth Pressure – Restrained Wall[2] | |
| From ground surface to H/4 (ft) | Increase from 0 to 25H psf |
| Below H/4 (ft) | Uniform pressure of 25H psf |
| Passive Pressure[3] | 400 pcf up to 2,000 psf max |

Note: 1 For the upper 5 feet (minimum for incidental loading)
2 Where H equals height of excavation
3 Can assume to act over 2 times the diameter of soldier piles, neglecting the upper foot

To limit potential movements of the shoring system, the shoring designer and contractor should consider several design and construction issues. For the movements of shoring to be reduced, the designer will have to provide for a uniform and timely mobilization of the soil pressures. Tiebacks or internal bracing should be loaded to the design loads prior to excavation of the adjacent soil so that load induced strains in the retaining system will not result in the system moving toward the excavation. In addition, a relatively stiff shoring system should be designed to limit deflections under loading. In general, we recommend designing a shoring system to deflect less than 1-inch.

In addition, ground subsidence and deflections can be caused by other factors such as voids created behind the shoring system by over-excavation, soil sloughing, erosion of sand or silt layers due to perched water, etc. All voids behind the shoring system should be filled as soon as feasible by grouting to minimize potential problems during installation of the shoring system.

Since we drilled our borings with hollow-stem auger drilling equipment, we are not able to evaluate the potential for caving of on-site soils, which may become a factor during soldier pile and/or tieback installation. The contractor is responsible for evaluating excavation difficulties prior to construction. Pilot holes using proposed production drilling equipment may be prudent, to evaluate possible excavation difficulties such as caving soils, cobbles, boulders and/or other excavation difficulties.

In conjunction with the shoring installation, a monitoring program should be set up and carried out by the contractor to determine the effects of the construction on the adjacent sound wall, street and other improvements such as sidewalks and utilities. As a minimum, we recommend horizontal and vertical surveying of reference points on the shoring and on the adjacent street, sound wall and other improvements in addition to an initial crack survey. We also recommend that all supported and/or sensitive utilities be located and monitored by the contractor. Reference points should be set up and read prior to the start of construction activities. Points should also be set on the shoring as soon as initial installations are made. Alternatively, inclinometers could be installed by the contractor at critical locations for a more detailed monitoring of shoring deflections. Surveys should be made at least once a week and more frequently during critical construction activities, or if significant deflections are noted. TRC can provide inclinometer materials and has the equipment and software to read and analyze the data quickly.

This report is intended for use by the design team. The contractor should perform additional subsurface exploration and/or geotechnical studies as they deem necessary for the chosen shoring system. The contractor is also responsible for site safety and the means and methods of construction, including temporary shoring. Temporary shoring must be designed by a California licensed Civil or Structural Engineer. Prior to construction, we recommend that the contractor forward his plan for the support system to the structural engineer and geotechnical engineer for preconstruction review.

## 6.13 Temporary Dewatering

As previously discussed, measured ground water elevations levels and design ground water levels are near or above the planned excavation depths; therefore, temporary dewatering will likely be necessary during construction for the proposed structure. Temporary dewatering for construction should be the responsibility of the contractor. The selection of equipment and methods of dewatering should be left up to the contractor and, due to the variable nature of the subsurface conditions in the area, the contractor should be aware that modifications to the dewatering system, such as adding well points or wells, may be required during construction depending on the conditions encountered.

During excavations, we recommend that the ground water level be maintained 5 feet below the bottom of the excavation to help reduce earthwork difficulties. Ground water should not be drawn down deeper than necessary; however, as lowering the ground water table can cause subsidence and settlement of adjacent parcels. Additionally, it may be desired to allow dewatering to take place for some time (1 to 2 weeks) before the excavation begins to allow time for the soil to drain. We should review the dewatering and excavation plans prior to construction.

Should dewatering be temporarily shut down while the excavation is open, it could have considerable affects on the excavations, including flooding, destabilization of the bottom of the excavation, shoring failures, etc. Therefore, we recommend that consideration be given to having the dewatering contractor provide backup power in case of loss of power or other redundancies, as deemed necessary.

Special considerations may be required prior to discharge of ground water from dewatering activities depending on the environmental impacts at the site or at nearby locations. These requirements may include storage and testing under permit prior to discharge. Impacted ground water may require discharge to an offsite facility.

## 6.14 Surface Drainage

Positive surface water drainage gradients, at least 2 percent in landscaping and 0.5 percent in pavement areas, should be provided to direct surface water away from foundations and slabs towards suitable discharge facilities. Ponding of surface water should not be allowed on or adjacent to structures, slabs-on-grade, or pavements. Roof runoff should be directed away from foundation and slabs-on-grade. Downspouts may discharge onto splash-blocks provided the area is covered with concrete slabs or asphalt concrete pavements.

## 6.15 Landscaping Considerations

We recommend restricting the amount of surface water infiltrating these soils near structures and slabs-on-grade. This may be accomplished by:

- Selecting landscaping that requires little or no watering, especially within 3 feet of structures, slabs-on-grade, or pavements,
- Using low flow rate sprinkler heads, or preferably, drip irrigation systems

- Regulating the amount of water distributed to lawn or planter areas by installing timers on the sprinkler system,

- Providing surface grades to drain rainfall or landscape watering to appropriate collection systems and away from structures, slabs-on-grade, or pavements,

- Preventing water from draining toward or ponding near building foundations, slabs-on-grade, or pavements, and

- Avoiding open planting areas within 3 feet of the building perimeters.

We recommend that the landscape architect consider these items when developing the landscaping plans.

### 6.16 Construction Observation

A representative from our company should observe the geotechnical aspects of the grading and earthwork for general conformance with our recommendations including site preparation, selection of fill materials, and the placement and compaction of fill. To facilitate your construction schedule we request sufficient notification (48 hours) for site visits. The project plans and specifications should incorporate all recommendations contained in the text of this report.

## 7.0 FOUNDATIONS

As discussed in the Conclusions and Recommendations section there is a potential for liquefaction induced settlement, ground rupture, and lateral spreading to occur. Provided that the site is prepared in accordance with the “Earthwork” section of this report and the proposed structure can be designed to accommodate the following estimated amounts of settlement, the structure may be supported on reinforced mat foundations bearing on improved ground.

As an alternative to ground improvement, the structure may be supported on deep foundations consisting of augered cast-in-place piles. We do not recommend driven piles for the project due to the potential for vibration induced damage to existing nearby structures and the potential for driving refusal in the bedrock that was encountered at varying depths below the project site. It is our opinion that augercast pile foundations will be able to support the structure with only minor settlements and will provide adequate support during liquefaction and seismic events. Foundation recommendations are discussed in the sections below.

### 7.1 2013 CBC Site Coefficients and Site Seismic Coefficients

Chapter 16 of the 2013 California Building Code (CBC) outlines the procedure for seismic design of structures. Based on our explorations, the site is generally underlain by hard silts, and loose to very dense sands, which correspond to a soil profile type D. Based on the above information and local seismic sources, the site may be characterized for design using the information in Table 6 below.

Table 6. 2013 CBC Site Class and Site Seismic Coefficients

| Latitude: 36.9713 N<br>Longitude: 122.0239 W | CBC Reference | Factor/ Coefficient | Value |
|---|---|---|---|
| Soil Profile Type | Section 1613.3.2 | Site Class | D |
| Mapped Spectral Response Acceleration for MCE at 0.2 second Period | Figure 1613.3.1(1) | $S_s$ | 1.50 |
| Mapped Spectral Response Acceleration for MCE at 1 Second Period | Figure 1613.3.1(2) | $S_1$ | 0.60 |
| Site Coefficient | Table 1613.3.3(1) | $F_a$ | 1.0 |
| Site Coefficient | Table 1613.3.3(2) | $F_v$ | 1.5 |
| Adjusted MCE Spectral Response Parameter | Equation 16-37 | $S_{MS}$ | 1.50 |
| Adjusted MCE Spectral Response Parameter | Equation 16-38 | $S_{M1}$ | 0.90 |
| Design Spectral Response Acceleration Parameter | Equation 16-39 | $S_{DS}$ | 1.00 |
| Design Spectral Response Acceleration Parameter | Equation 16-40 | $S_{D1}$ | 0.60 |

## 7.2 Reinforced Mat Foundations

The proposed structure may be supported on a conventionally reinforced mat foundation bearing on the site soils improved with one of the recommended ground improvement methods discussed in the “Earthwork” section of this report. Based on the subsurface conditions, the mat may be designed for an average allowable bearing pressure of 1,500 pounds per square foot (psf) for dead plus live loads with maximum localized allowable bearing pressures of 3,000 psf at column or wall loads. Allowable bearing pressures may be increased by one-third for all loads including wind or seismic. These allowable bearing pressures are net values; the weight of the mat can be neglected for design purposes.

The mat should be reinforced with top and bottom steel, as appropriate, to provide structural continuity and to permit spanning of local irregularities. These recommendations may be revised depending on the particular design method selected by the structural engineer. It is essential that we observe the subgrade of the mat foundation prior to placement of reinforcing steel.

### 7.2.1 Mat Foundation Settlement

Our calculations for the proposed structure with a reinforced mat foundation designed for an average allowable bearing pressure of 1,500 psf for dead plus sustained live loads indicate total static settlement will be on the order of approximately ½-inch with differential settlement of approximately ¼-inch in 50 horizontal feet for a mat bearing approximately 9 feet below the existing grade. Mat settlements due to liquefaction are summarized for each ground improvement method discussed in the Earthwork section of this report and should be confirmed with the ground improvement contractor.

### 7.2.2 Lateral Loads

Lateral loads may be resisted by friction between the bottom of mats and the supporting subgrade. A maximum allowable frictional resistance of 0.3 may be used for design. In addition, lateral resistance may be provided by passive pressures acting against deepened mat edges poured neat against competent soil. We recommend that an allowable passive pressure based on an equivalent fluid pressure of 300 pounds per cubic foot (pcf) be used in design.

## 7.3 Augercast Piles

Augercast piles have been successfully used for projects throughout the Bay Area in similar soil conditions. Augercast piles are cast-in-place concrete piles that are drilled using a hollow-stem auger and pumping sand-cement grout through the bottom of the auger as the auger is retracted. Three types of augercast piles are available: augercast pressure-grouted (APG) piles, which like piers, remove the soil column and replace it with grout; augercast, pressure-grouted displacement (APGD) piles, which displace the soil prior to grout placement and augercast and pressure-grouted partial-displacement (APGPD) piles, which partially displace the soil prior to grout placement. Augercast piles are a low noise and vibration installation compared to driven piles and would not require pre-drilling. Various types of steel reinforcing including rebar cages or H-piles may be installed into the still wet grout after drilling.

Based on the subsurface conditions encountered, we would recommend the use of APGPD or APGD piles for the project. The displacement action of the auger will densify surrounding soils, reducing the potential for liquefaction and lateral spreading as well as reducing the volume of soil cuttings generated.

### 7.3.1 Vertical Capacities

As with driven piles, augercast piles will develop their vertical capacity predominately from frictional support in the stiff clays and silts and dense sands. We computed allowable downward vertical capacities for 18- and 24-inch diameter APG piles at two locations. A summary of the allowable pile capacities with downdrag forces is presented in Tables 9 and 10 below. In addition, Figure 4 shows the increase in pile capacity with length. The indicated capacities in Table 9 and 10 and Figure 4 are for dead plus live loads. Dead loads should not exceed two-thirds of the computed capacities. Uplift loads should also not exceed two-thirds of the computed downward capacities. The pile capacities may be increased by one-third under transient loading, including wind and seismic. Because downdrag forces due to liquefaction-induced settlements are considered to be a short-term loading condition, we used a factor-of-safety of 1.5 for the allowable skin friction of the soils below the depth of downdrag forces.

Piles extending into the bedrock at the site may increase axial capacity more quickly than shown in Figure 4. We recommended the implementation of a pile test program as discussed in section 7.3.4. Please note that displacement augercast piles may be limited to a total drill depth of 75 to 80 feet below grade or less due to the equipment limitations.

We have assumed a base of pile cap at approximately 12 feet below the existing site grade for our analysis. To effectively minimize pile group effects and reduction in individual pile capacity, piles should be located with a minimum center-to-center spacing of three times the pile diameter.

Table 9. Estimated Allowable Capacities for 18-and 24-inch Augercast Piles
Downdrag to 36 feet

| Pile Size | Length[1] (feet) | Estimated Allowable Capacity (dead plus live loads) (kips) |
|---|---|---|
| 18-inch round | 52 | 50 |
| 18-inch round | 60 | 100 |
| 18-inch round | 67 | 150 |
| 18-inch round | 73 | 200 |
| 24-inch round | 50 | 50 |
| 24-inch round | 56 | 100 |
| 24-inch round | 61 | 150 |
| 24-inch round | 67 | 200 |

Note: 1 – Length from bottom of pile cap to bottom of pile

Table 10. Estimated Allowable Capacities for 18-and 24-inch Augercast Piles
Downdrag to 70 feet

| Pile Size | Length[1] (feet) | Estimated Allowable Capacity (dead plus live loads) (kips) |
|---|---|---|
| 18-inch round | 101 | 50 |
| 18-inch round | 105 | 100 |
| 18-inch round | 110 | 150 |
| 18-inch round | 114 | 200 |
| 24-inch round | 99 | 50 |
| 24-inch round | 103 | 100 |
| 24-inch round | 106 | 150 |
| 24-inch round | 110 | 200 |

Note: 1 – Length from bottom of pile cap to bottom of pile

Based on the maximum allowable loads for a single pile, we estimate total settlements of less than ¾-inch to mobilize allowable static capacities. Therefore, post-construction pile foundation settlements of less than ½-inch should be anticipated.

### 7.3.2 Lateral Loads on Augercast Piles

To estimate lateral capacities of piles, we used a computer program that models the soil response in the form of load-deflection (p-y) curves to estimate the capacity of the piles to resist the expected lateral loads. The lateral load characteristics for 18- and 24-inch diameter, augercast piles with fixed- and free-head conditions are presented in Table 10 below for 5 feet below grade. A 150 kip axial load was used.

Table 10. Estimated Lateral Pile Response – 18- and 24-inch Round Piles

| Pile Size | Head Condition | Deflection (inches) | Maximum Shear Force (kips) | Maximum Moment (ft-kips) | Depth to Maximum Moment (ft) |
|---|---|---|---|---|---|
| 18-inch | Free | ¼ | 25 | 68 | 5.4 |
| | | ½ | 43 | 125 | 5.4 |
| | Fixed | ¼ | 59 | 198 | Top of Pile |
| | | ½ | 101 | 359 | |
| 24-inch | Free | ¼ | 37 | 132 | 6.6 |
| | | ½ | 62 | 240 | 7.2 |
| | Fixed | ¼ | 86 | 376 | Top of Pile |
| | | ½ | 145 | 681 | |

The analysis results represent the probable response of the piles under short-term loading conditions and include no factor-of-safety. Suitable factors-of-safety should be selected on the basis of the type of loading. Pile stiffnesses (EI) of 1.9 x $10^{10}$ lb-in$^2$ and 5.9 x $10^{10}$ lb-in$^2$ have been assumed in our calculations of load deflection for the 18- and 24-inch piles, respectively. We assumed a minimum compressive strength of 4,000 pounds per square inch for concrete modulus calculations. If pile stiffness varies by no more than 20 percent than that reported above, load deflection characteristics can be approximated by multiplying the deflection values by the ratio of the pile stiffness (EI). We should evaluate the response of piles with significantly different stiffness.

The above lateral load characteristics are for single piles and may not be characteristic of the lateral load capacity of piles in a group. Group effects may reduce the allowable lateral load for a given deflection. We recommend that a pile group efficiency of 0.75 be used for pile groups 3-by-3 or smaller. A group reduction would not be necessary for groups of 1 or 2 piles. For pile groups larger than 3-by-3, we recommend that we review the final pile group layout and structural loads to further evaluate the pile group efficiency under lateral loading.

Due to the potential for lateral spreading, the piles in the two rows of pile groups nearest to the San Lorenzo River should be designed for an additional equivalent fluid pressure of 50 pcf acting on the upper 13 feet of the pile (assuming the top of the pile is 12 feet below the existing ground surface). We recommend assuming no passive resistance along the upper 13 feet of the pile. The lateral loading should be assumed to act over two pile diameters, and the piles should be spaced no further apart than three pile diameters, center to center.

### 7.3.3 Passive Resistance against Pile Caps and Grade Beams

If desired, the passive resistance of soil against pile caps and grade beams poured neat against well-compacted engineered fill may be used for lateral resistance. We recommend that an allowable passive pressure based on an equivalent fluid pressure of 300 pounds per cubic foot be used in design.

### 7.3.4 Pile Load Tests

Load testing for augercast pile foundations typically consists of performing one or more full scale static load tests. Static load tests include installing a test pile, with four surrounding anchor piles supporting a load frame to resist jacking against the test pile. The test pile may or may not be installed in a production pile location. During installation of the test piles, the contractor should allow for monitoring pile displacement at the top of pile, 10 feet below top, middle, and pile tip. Monitoring can be by strain gauges or capped conduits placed in the pile, allowing telltales to be placed during testing. This will allow for observation of the loads at which the skin friction is mobilized. A more detailed description of static load tests is presented in ASTM D1143. A member of our staff should be present during installation of the test piles and load testing and have the opportunity to review the test results.

### 7.3.5 Garage Floor Slabs

If the structure is pile supported, the floor slab for the parking garage should be designed to structurally span between pile caps and grade beams to minimize the impact of ground rupture.

## 7.4 Hydrostatic Uplift and Waterproofing

We recommend that the mat or structural slab be designed to withstand hydrostatic uplift pressures to a design ground water of 5 feet below site grades. We also recommend that the mat foundation, all construction joints, and basement walls be waterproofed to limit moisture infiltration. We recommend that a waterproofing specialist design the waterproofing system, including the under-mat waterproofing design and all below-grade walls. A rat slab could be poured over the sugrade to protect the waterproofing as reinforcing steel is placed. The issue of drainage systems above historic high ground water levels and designing for hydrostatic pressures for the basement walls are discussed in subsequent report sections.

# 8.0 BASEMENT WALLS

## 8.1 Lateral Earth Pressures

The basement walls should be designed to resist lateral earth pressures from adjoining natural materials, backfill, and surcharge loads. Provided that adequate drainage is provided as recommended below, we recommend that walls restrained from movement at the top and level upslope conditions be designed to resist an equivalent fluid pressure of 45 pounds per cubic foot (pcf) plus a uniform pressure of 8H pounds per square foot, where H is the distance in feet between the bottom of the mat and the top of the wall. Where the proposed structure abuts the existing levee (2H:1V upslope), we recommend that a restrained wall be designed using and equivalent fluid pressure of 55 pcf. Restrained walls should also be designed to resist an additional uniform pressure equivalent to one-half of any surcharge loads applied at the surface.

Any unrestrained retaining walls with adequate drainage should be designed to resist an equivalent fluid pressure of 45 pcf plus one-third of any surcharge loads. Unrestrained retaining walls abutting the levee should be designed to resist an equivalent fluid pressure of 55 pcf.

The above lateral earth pressures assume level backfill conditions and sufficient drainage behind the walls to prevent build-up of hydrostatic pressure from surface water infiltration and/or a rise in the ground water level. The walls should be designed as undrained below a depth of 5 feet below existing site grades and should have an equivalent fluid pressure of 40 pcf added to the values recommended above. Provided a wall drainage system as described below is included above 5 feet below existing site grades, the walls in the upper 5 feet may be designed based on drained earth pressures. Dampproofing of the walls should be included in areas where wall moisture and efflorescence would be undesirable.

## 8.2 Seismic Lateral Earth Pressures

We understand the basement walls may be designed for seismic lateral loading. For our analysis, we have assumed that the walls will have flat, non-sloping backfill. We used the Mononobe-Okabe approach to approximate the increased earth pressures induced by earthquakes. As discussed in Section 3.2 of our report, a peak ground acceleration of 0.5g is expected at the site. We performed calculations using this ground acceleration, and estimated an additional seismic increment of $8H^2$ for fixed walls. This seismic increment is a resultant applied to the wall in addition to the static lateral earth pressures given in Section 8.1. For fixed walls the additional seismic load would be applied as a uniform pressure with the resultant applied at mid-height.

### 8.3 Drainage

The basement walls should be designed to withstand hydrostatic pressures below a depth of 5 feet below existing site grades. Passive wall drainage should be provided above the design ground water of 5 feet below existing site grades. The passive wall drainage system should consist of a 4-inch minimum diameter perforated pipe placed at 5 feet below the existing site grades (perforations placed downward). The pipe should be bedded and backfilled with Class 2 Permeable Material per Caltrans Standard Specifications, latest edition. The permeable backfill should extend at least 12 inches out from the wall and to within 2 feet of outside finished grade. Alternatively, ½-inch to ¾-inch crushed rock may be used in place of the Class 2 Permeable Material provided the crushed rock and pipe are enclosed in filter fabric, such as Mirafi 140N or equivalent. The upper 2 feet of wall backfill should consist of relatively low permeable compacted on-site clayey soil. The subdrain outlet should be connected to a free-draining outlet or sump.

A suitable prefabricated drainage system designed for this specific use, such as Miradrain, Geotech Drainage Panels, or Enkadrain drainage matting may be used for wall drainage as an alternative to the Class 2 Permeable Material or drain rock backfill. The drainage panel should be connected to the perforated pipe at 5 feet below existing site grades, or to some other closed or through-wall system. Miradrain panels should terminate 18 to 24 inches from final exterior grade. The prefabricated drainage system should be installed against the wall (if excavation is laid back) or shoring system and should be installed in at least 4-foot-wide vertical strips at 8 feet on-center around the basement walls. The Miradrain panel filter fabric should be extended over the top of and behind the panel to protect it from intrusion of the adjacent soil.

We recommend that design details for draining the basement walls above the design ground water level be determined prior to completion of construction documents as this is often a critical feature. A sump will likely be needed for drainage at this elevation unless storm drains are at an elevation that would accept the water by gravity. A horizontal collection system external to the basement should drain to a sump system. Waterproofing should be installed between the drainage system and the basement walls. The project structural engineer should review and approve any notch or penetrations planned in basement walls.

### 8.4 Backfill

Where surface improvements will be located over the retaining wall backfill, backfill placed behind the walls should be compacted to at least 95 percent relative compaction using light compaction equipment. Where no surface improvements are planned, backfill should be compacted to at least 90 percent. If heavy compaction equipment is used, the walls should be temporarily braced.

### 8.5 Foundation

Basement walls may be supported on the mat foundation designed in accordance with the recommendations presented in the “Reinforced Mat Foundation” section of this report. Lateral load resistance for the walls may be developed in accordance with the recommendations in the “Lateral Loads” section.

## 9.0 PAVEMENTS

### 9.1 Asphalt Concrete

We obtained a representative bulk sample of the surface soil from the parking area and performed an R-value test to provide data for pavement design. The results of the test are included in Appendix B and indicate an R-value of 72. We judge an R-value of 50 to be applicable for design based on a subgrade consisting of untreated native soils. Using estimated traffic indices for various pavement-loading requirements and untreated native soils, we developed the following recommended pavement sections based on Procedure 608 of the Caltrans Highway Design Manual, presented in Table 11.

Table 11. Recommended Asphalt Concrete Pavement Design Alternatives
Pavement Components
Design R–Value = 50

| General Traffic Condition | Design Traffic Index | Asphalt Concrete (Inches) | Aggregate Baserock* (Inches) | Total Thickness (Inches) |
|---|---|---|---|---|
| Automobile Parking | 4.0 | 2.5 | 4.0 | 6.5 |
| | 4.5 | 2.5 | 4.0 | 6.5 |
| Automobile Parking Channel | 5.0 | 3.0 | 4.0 | 7.0 |
| | 5.5 | 3.0 | 4.0 | 7.0. |
| Truck Access & Parking Areas | 6.0 | 3.5 | 4.0 | 7.5 |
| | 6.5 | 4.0 | 4.0 | 8.0 |

*Caltrans Class 2 aggregate base; minimum R-value equal to 78.

The traffic indices used in our pavement design are considered reasonable values for the proposed development and should provide a pavement life of approximately 20 years with a normal amount of flexible pavement maintenance. The traffic parameters used for design were selected based on engineering judgment and not on information furnished to us such as an equivalent wheel load analysis or a traffic study. Because of the presence of moderately expansive clay at the site, some increased amount of maintenance should be expected.

Because the full thickness of asphalt concrete is frequently not placed prior to construction traffic being allowed to use the streets (or parking lots), rutting and pavement failures can occur prior to project completion. To reduce this occurrence, we recommend that either the full design pavement section be placed prior to use by construction traffic, or a higher Traffic Index (TI) be specified where construction traffic will use the pavement.

## 9.2 Exterior Portland Cement Concrete (PCC) Pavements

Recommendations for exterior PCC pavements are presented below in Table 12. Since the expected Average Daily Truck Traffic (ADTT) is not known at this time, we have provided alternatives for minimum pavement thickness. An allowable ADTT should be chosen that is greater than expected for the development.

Table 12. Recommended Minimum PCC Pavement Thickness

| Allowable ADTT | Minimum PCC Pavement Thickness (inches) |
|---|---|
| 0.8 | 5 |
| 13 | 5½ |
| 130 | 6 |

Our design is based on an R-value of 10 and a 28-day unconfined compressive strength for concrete of at least 3,500 pounds per square inch. In addition, our design assumes that pavements are restrained laterally by a concrete shoulder or curb and that all PCC pavements are underlain by at least 6 inches of Class 2 aggregate base. We recommend that adequate construction and control joints be used in design of the PCC pavements to control the cracking inherent in this construction.

### 9.3 Pavement Cutoff

Surface water infiltration beneath pavements could significantly reduce the pavement design life. While the amount of reduction in pavement life is difficult to quantify, in our opinion, the normal design life of 20 years may be reduced to less than 10 years. Therefore, long-term maintenance greater than normal may be required.

To limit the need for additional long-term maintenance, it would be beneficial to protect at-grade pavements from landscape water infiltration by means of a concrete cut-off wall, deepened curbs, redwood header, "Deep-Root Moisture Barrier," or equivalent. However, if reduced pavement life and greater than normal pavement maintenance are acceptable, the cutoff barrier may be eliminated. If desired to install pavement cutoff barriers, they should be considered where pavement areas lay downslope of any landscape areas that are to be sprinkled or irrigated, and should extend to a depth of at least 4 inches below the base rock layer.

### 9.4 Asphalt Concrete, Aggregate Base and Subgrade

Asphalt concrete and aggregate base should conform to and be placed in accordance with the requirements of Caltrans Standard Specifications, latest edition, except that ASTM Test Designation D1557 should be used to determine the relative compaction of the aggregate base. Pavement subgrade should be prepared and compacted as described in the "Earthwork" section of this report.

### 9.5 Flatwork and Sidewalks

We recommend that exterior slabs-on-grade, such as flatwork and sidewalks be at least 4 inches thick and be underlain by at least 4 inches of Class 2 aggregate base compacted to a minimum of 90 percent relative compaction in accordance with ASTM Test Method D1557, latest edition. If sidewalks are subject to wheel loads, they should be designed in accordance with the "Exterior Portland Cement Concrete Pavements" section of this report.

## 10.0 LIMITATIONS

This report has been prepared for the sole use of SC Riverfront, LLC, specifically for design of the proposed Riverfront Apartments project in Santa Cruz, California. The opinions, conclusions, and recommendations presented in this report have been formulated in accordance with accepted geotechnical engineering practices that exist in the San Francisco Bay Area at the time this report was written. No other warranty, expressed or implied, is made or should be inferred.

The opinions, conclusions and recommendations contained in this report are based upon the information obtained from our investigation, which includes data from widely separated discrete locations, visual observations from our site reconnaissance, and review of other geotechnical data provided to us, along with local experience and engineering judgment. The recommendations presented in this report are based on the assumption that soil and geologic conditions at or between the borings and CPTs do not deviate substantially from those encountered or extrapolated from the information collected during our investigation. We are not responsible for the data presented by others.

We should be retained to review the geotechnical aspects of the final plans and specifications for conformance with our recommendations. The recommendations provided in this report are based on the assumption that we will be retained to provide observation and testing services during construction to confirm that conditions are similar to that assumed for design and to form an opinion as to whether the work has been performed in accordance with the project plans and specifications. If we are not retained for these services, TRC cannot assume any responsibility for any potential claims that may arise during or after construction as a result of misuse or misinterpretation of TRC's report by others. Furthermore, TRC will cease to be the Geotechnical-Engineer-of-Record if we are not retained for these services and/or at the time another consultant is retained for follow up service to this report.

The opinions presented in this report are valid as of the present date for the property evaluated. Changes in the condition of the property will likely occur with the passage of time due to natural processes and/or the works of man. In addition, changes in applicable standards of practice can occur as a result of legislation and/or the broadening of knowledge. Furthermore, geotechnical issues may arise that were not apparent at the time of our investigation. Accordingly, the opinions presented in this report may be invalidated, wholly or partially, by changes outside of our control. Therefore, this report is subject to review and should not be relied upon after a period of three years, nor should it be used, or is it applicable, for any other properties.

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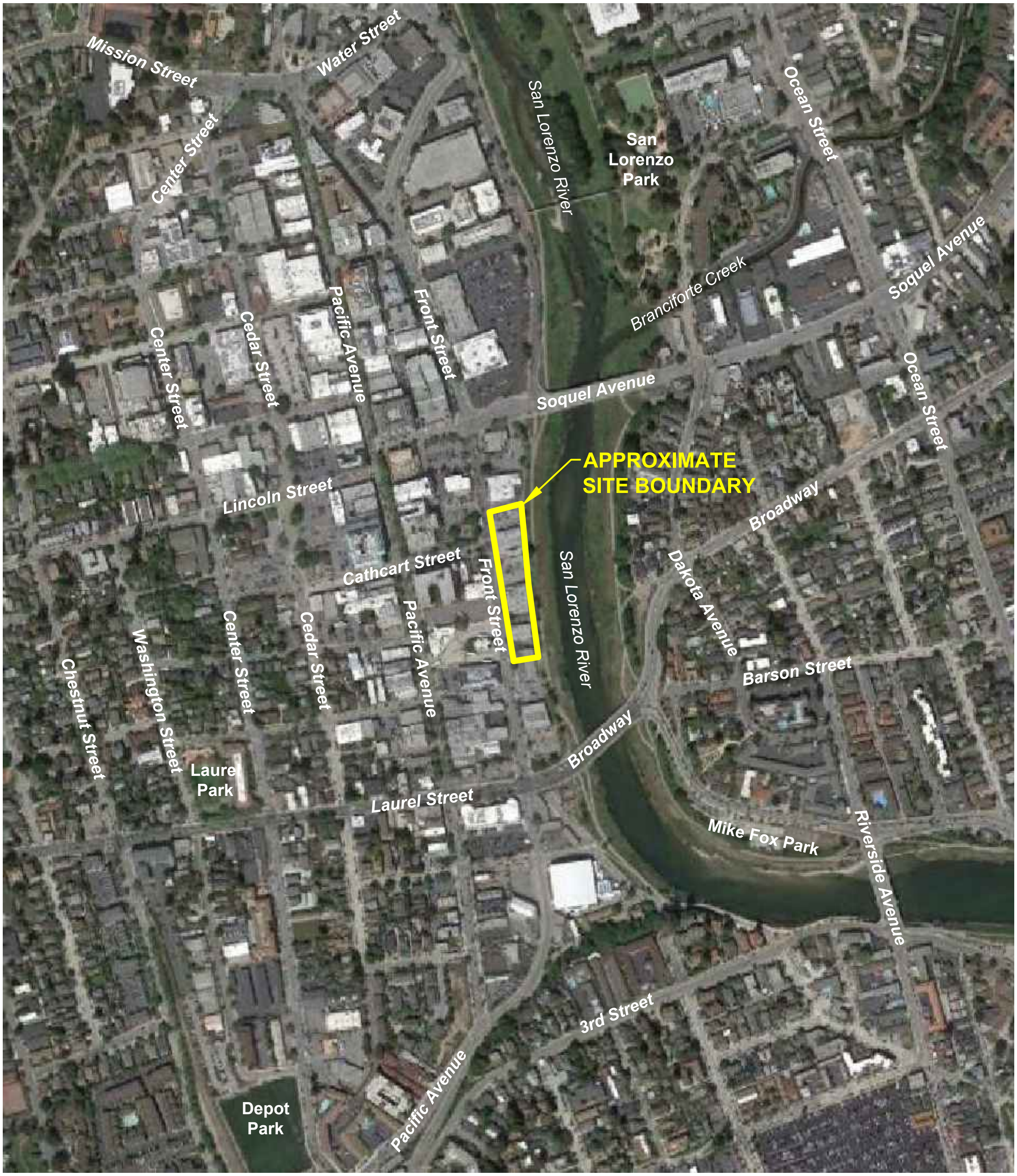
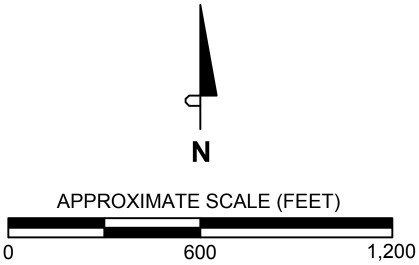


SOURCE AERIAL PHOTO: Google Earth, March 2015.



**VICINITY MAP**

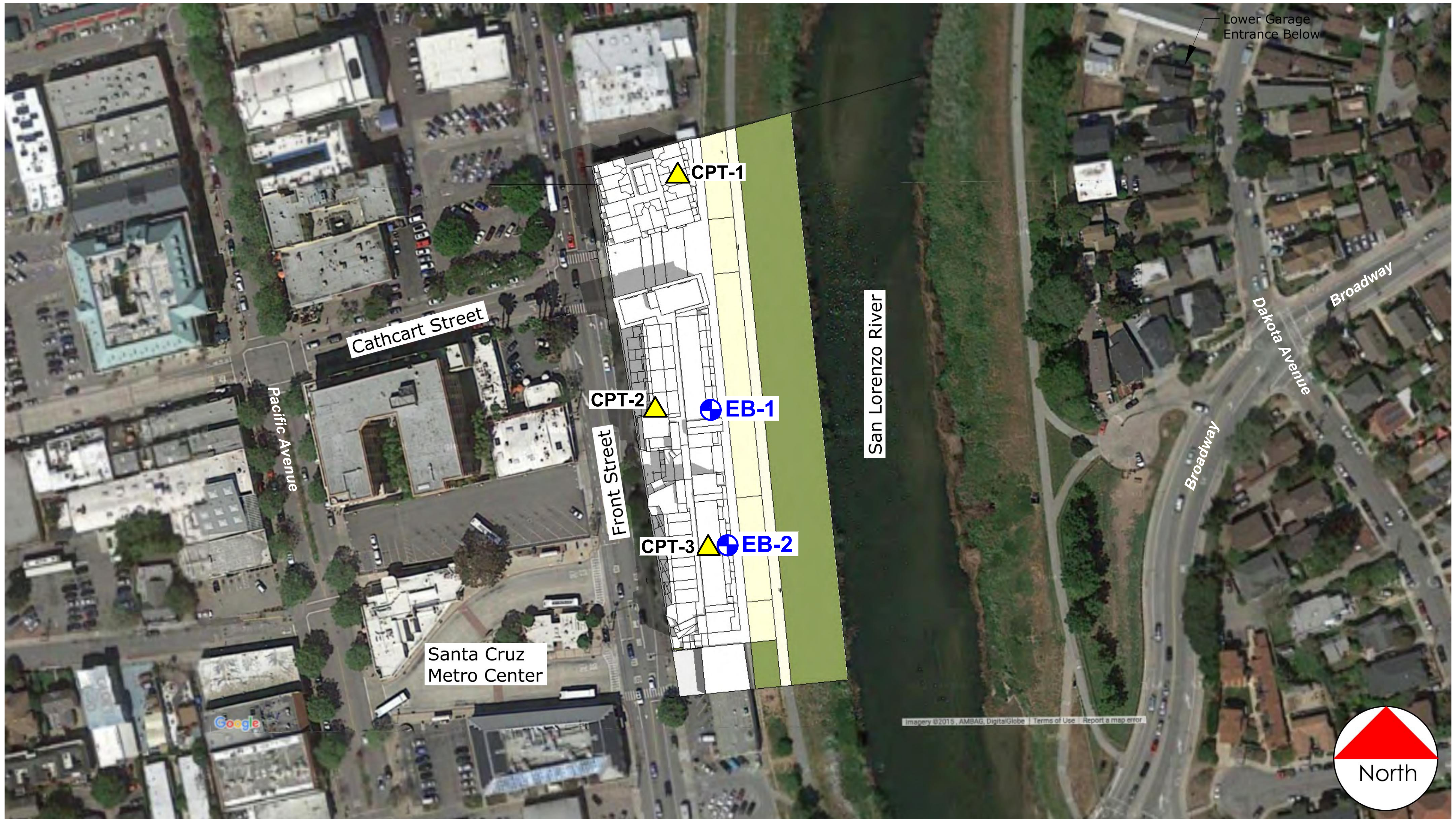
Riverfront Apartments
Front Street
Santa Cruz, California

TRC

252341

**FIGURE 1**

FILE NAME: Z:\CAD_DRAW\Current\Riverfront Apts_Santa Cruz\Fig1_Vicinity Map.dwg | Layout Tab: 8x11

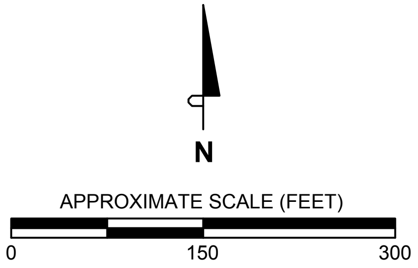


SOURCE: *Site plan by Jon Worden, January 2016.*

## LEGEND

*Approximate locations of:*

| | |
|---|---|
| △ | Cone penetration test (CPT) |
| ⊕ | Exploratory boring |

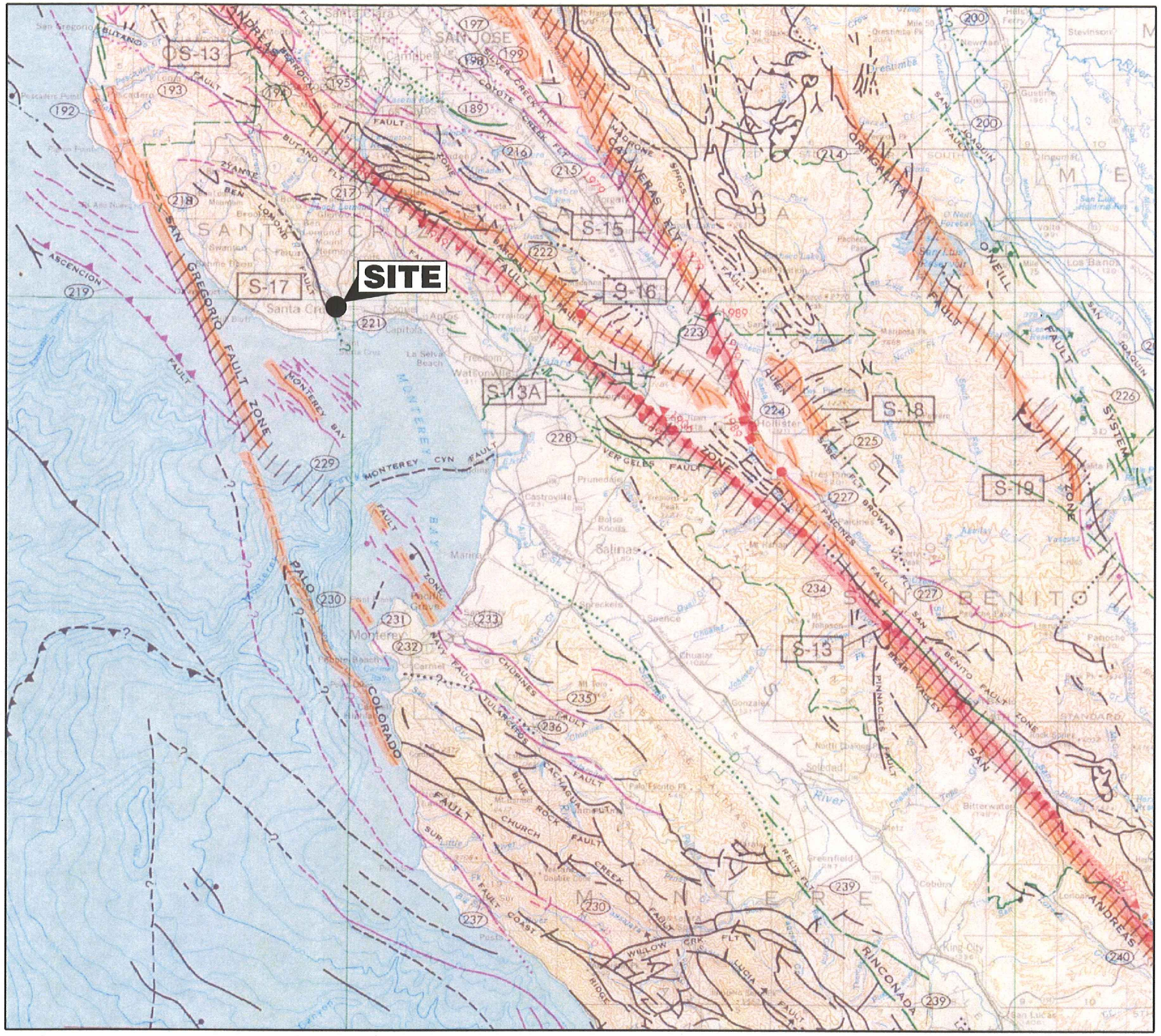


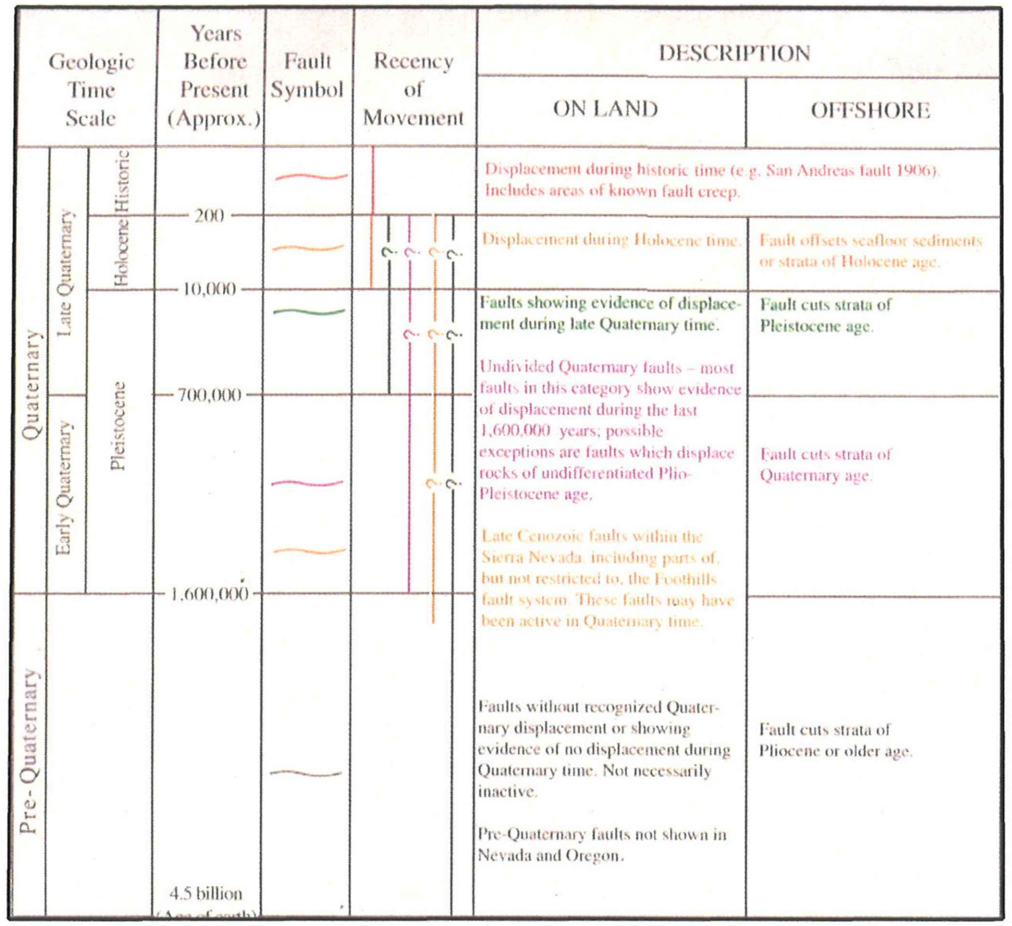
**SITE PLAN**

Riverfront Apartments
Front Street
Santa Cruz, California

TRC | 252341 | **FIGURE 2**

FILE NAME: Z:\CAD_DRAW\Current\Riverfront Apts_Santa Cruz\Fig2_Site Plan.dwg | Layout Tab: 11x17





| Geologic Time Scale | Years Before Present (Approx.) | Fault Symbol | Recency of Movement | DESCRIPTION<br>ON LAND | <br>OFFSHORE |
|---|---|---|---|---|---|
| Quaternary<br>Late Quaternary<br>Historic | 200 | ~ | | Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep. | |
| Holocene | 10,000 | ~ | ? ? ? ? | Displacement during Holocene time. | Fault offsets seafloor sediments or strata of Holocene age. |
| Pleistocene | 700,000 | ~ | ? ? ? | Faults showing evidence of displacement during late Quaternary time. | Fault cuts strata of Pleistocene age. |
| Early Quaternary | | ~ | ? ? | Undivided Quaternary faults – most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age. | Fault cuts strata of Quaternary age. |
| | 1,600,000 | ~ | | Late Cenozoic faults within the Sierra Nevada, including parts of, but not restricted to, the Foothills fault system. These faults may have been active in Quaternary time. | |
| Pre-Quaternary | | ~ | | Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive. | Fault cuts strata of Pliocene or older age. |
| | 4.5 billion (Age of earth) | | | Pre-Quaternary faults not shown in Nevada and Oregon. | |

Fault traces on land are indicated by solid lines where well located, by dashed lines where approximately located or inferred, and by dotted lines where concealed by younger rocks or by lakes or bays. Fault traces are queried where continuation or existence is uncertain. Concealed faults in the Great Valley are based on maps of selected subsurface horizons, so locations shown are approximately and may indicate structural trend only. All offshore faults based on seismic reflection profile records are shown as solid lines where well defined, dashed where inferred, queried where uncertain.

N

APPROXIMATE
SCALE (MILES)

0 10

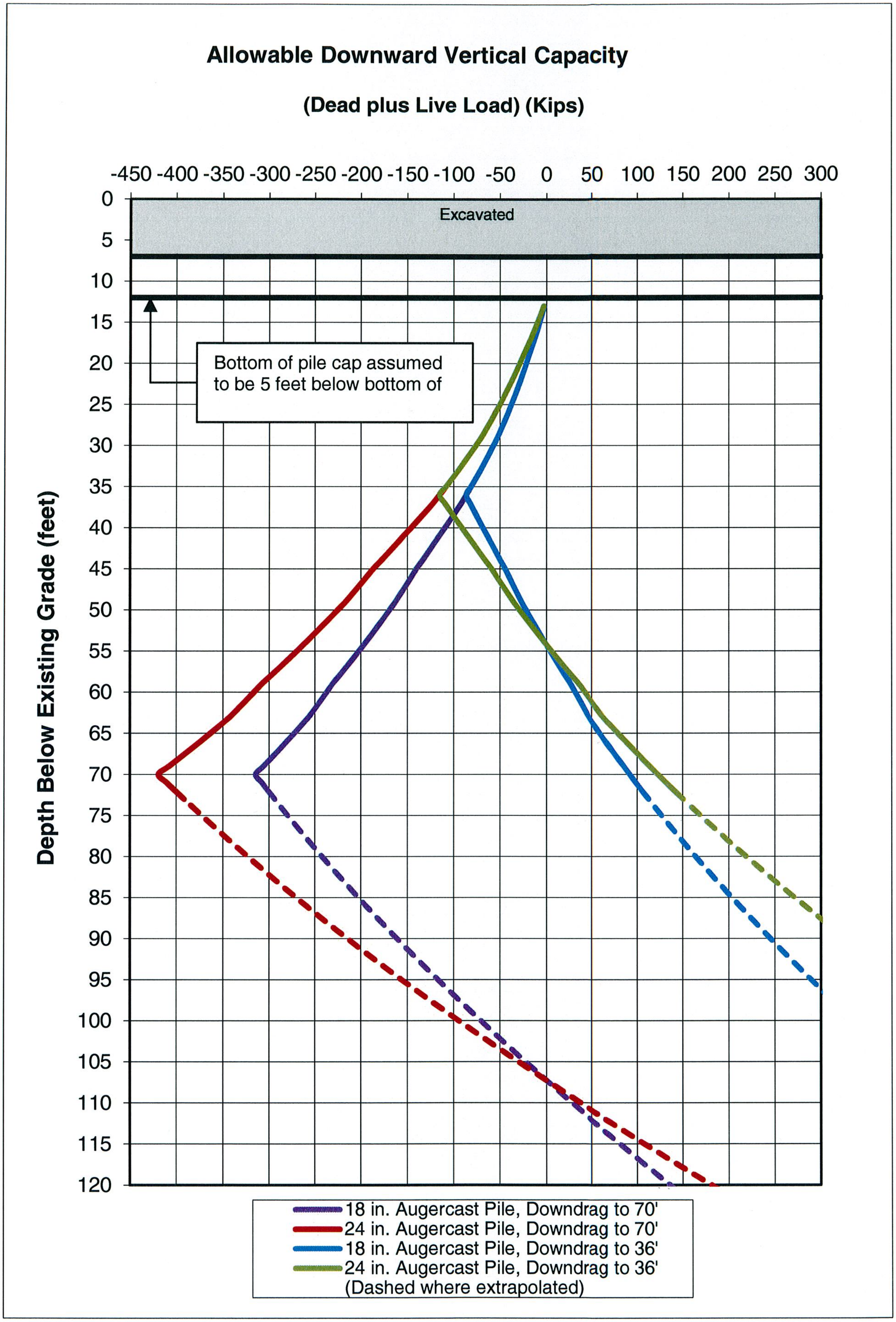
## REGIONAL FAULT MAP

Riverfront Apartments
Front Street
Santa Cruz, California

252341

**FIGURE 3**

FILE NAME: Z:\CAD_DRAW\Current\Riverfront Apts_Santa Cruz\Fig3_Regional Fault Map.dwg | Layout Tab: 11x17



**Allowable Downward Vertical Capacity**

**(Dead plus Live Load) (Kips)**

Bottom of pile cap assumed to be 5 feet below bottom of

9.37

Figure 4

# APPENDIX A
# FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using conventional, truck-mounted, hollow-stem auger drilling equipment and cone penetration test (CPT) equipment. Two 8-inch diameter exploratory borings were drilled on March 13, 2016 to a maximum depth of 41½ feet. Three CPTs were advanced on March 13, 2016 to a maximum depth of 73½ feet. The approximate locations of the exploratory borings and CPTs are shown on Figure 2. The soils encountered in the borings were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). The logs of the borings and CPTs, as well as a key to the classification of the soil and CPTs, are included as part of this appendix.

The locations of borings and CPTs were approximately determined by pacing from existing site boundaries. Elevations of the boring were not determined. The locations of the boring should be considered accurate only to the degree implied by the method used.

Representative soil samples were obtained from the borings at selected depths. All samples were returned to our laboratory for evaluation and appropriate testing. Penetration resistance blow counts were obtained by dropping a 140-pound hammer 30 inches. Modified California 3.0-inch outside diameter (O.D.) samples and Standard Penetration Test (SPT) 2-inch O.D. samples were obtained by driving the samplers 18 inches and recording the number of hammer blows for each 6 inches of penetration. Unless otherwise indicated, the blows per foot recorded on the boring logs represent the accumulated number of blows required to drive the samplers the last two 6-inch increments. When using the SPT sampler, the sum of the last two 6-inch increments is the uncorrected SPT measured blow count. The various samplers are denoted at the appropriate depth on the boring logs and symbolized as shown on Figure A-1.

The attached boring and CPT logs and related information depict subsurface conditions at the locations indicated and on the date designated on the logs. Subsurface conditions at other locations may differ from conditions occurring at these boring and CPT locations. The passage of time may result in altered subsurface conditions due to environmental changes. In addition, any stratification lines on the logs represent the approximate boundary between soil types and the transition may be gradual.

* * * * * * * * * * * *

| PRIMARY DIVISIONS | | | SOIL TYPE | | SECONDARY DIVISIONS |
|---|---|---|---|---|---|
| COARSE GRAINED SOILS<br>MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE | GRAVELS<br>MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE | CLEAN GRAVELS (Less than 5% Fines) | GW | ● | Well graded gravels, gravel–sand mixtures, little or no fines |
| | | | GP | ○ | Poorly graded gravels or gravel–sand mixtures, little or no fines |
| | | GRAVEL WITH FINES | GM | ○ | Silty gravels, gravel–sand–silt mixtures, plastic fines |
| | | | GC | ▨ | Clayey gravels, gravel–sand–clay mixtures, plastic fines |
| | SANDS<br>MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE | CLEAN SANDS (Less than 5% Fines) | SW | ⁘ | Well graded sands, gravelly sands, little or no fines |
| | | | SP | ⁘ | Poorly graded sands or gravelly sands, little or no fines |
| | | SANDS WITH FINES | SM | ▥ | Silty sands, sand–silt–mixtures, non–plastic fines |
| | | | SC | ▨ | Clayey sands, sand–clay mixtures, plastic fines |
| FINE GRAINED SOILS<br>MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE | SILTS AND CLAYS<br>LIQUID LIMIT IS LESS THAN 50 % | | ML | ▥ | Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity |
| | | | CL | ▨ | Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays |
| | | | OL | ☰ | Organic silts and organic silty clays of low plasticity |
| | SILTS AND CLAYS<br>LIQUID LIMIT IS GREATER THAN 50 % | | MH | ▥ | Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts |
| | | | 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 |

## DEFINITION OF TERMS

U.S. STANDARD SIEVE SIZE — CLEAR SQUARE SIEVE OPENINGS

200 — 40 — 10 — 4 — 3/4" — 3" — 12"

| SILTS AND CLAY | SAND | | | GRAVEL | | COBBLES | BOULDERS |
|---|---|---|---|---|---|---|---|
| | FINE | MEDIUM | COARSE | FINE | COARSE | | |

0.08 — 0.4 — 2 — 5 — 19 — 76mm

## GRAIN SIZES

☒ TERZAGHI SPLIT SPOON STANDARD PENETRATION

⧗ MODIFIED CALIFORNIA

▯ ROCK CORE

◪ PITCHER TUBE

○ NO RECOVERY

## SAMPLERS

| SAND AND GRAVEL | BLOWS/FOOT* |
|---|---|
| VERY LOOSE | 0–4 |
| LOOSE | 4–10 |
| MEDIUM DENSE | 10–30 |
| DENSE | 30–50 |
| VERY DENSE | OVER 50 |

## RELATIVE DENSITY

| SILTS AND CLAYS | STRENGTH+ | BLOWS/FOOT* |
|---|---|---|
| VERY SOFT | 0–1/4 | 0–2 |
| SOFT | 1/4–1/2 | 2–4 |
| MEDIUM STIFF | 1/2–1 | 4–8 |
| STIFF | 1–2 | 8–16 |
| VERY STIFF | 2–4 | 16–32 |
| HARD | OVER 4 | OVER 32 |

## CONSISTENCY

*Number of blows of 140 pound hammer falling 30 inches to drive a 2–inch O.D. (1–3/8 inch I.D.) split spoon (ASTM D–1586).
+Unconfined compressive strength in tons/sq.ft. as determined by laboratory testing or approximated by the standard penetration test (ASTM D–1586), pocket penetrometer, torvane, or visual observation.

# KEY TO EXPLORATORY BORING LOGS

### Unified Soil Classification System (ASTM D–2487)

TRC

**FIGURE A–1**

# EXPLORATORY BORING: EB-1

Sheet 1 of 2

DRILL RIG: TRUCK MOBILE CME-75
BORING TYPE: 8-INCH HOLLOW STEM AUGER
LOGGED BY: AC
START DATE: 3-3-16 FINISH DATE: 3-3-16

PROJECT NO: 252341
PROJECT: RIVERFRONT APARTMENTS
LOCATION: SANTA CRUZ, CA
COMPLETION DEPTH: 41.5 FT.

This log is a part of a report by TRC, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

Undrained Shear Strength (ksf)
○ Pocket Penetrometer
△ Torvane
● Unconfined Compression
▲ U-U Triaxial Compression
1.0 2.0 3.0 4.0

| ELEVATION (FT) | DEPTH (FT) | SOIL LEGEND | MATERIAL DESCRIPTION AND REMARKS<br>SURFACE ELEVATION: | SOIL TYPE | PENETRATION RESISTANCE (BLOWS/FT.) | SAMPLER | MOISTURE CONTENT (%) | DRY DENSITY (PCF) | PERCENT PASSING NO. 200 SIEVE |
|---|---|---|---|---|---|---|---|---|---|
| | 0 | | **4.5" of AC over 1.5" of AB** | AC/AB | | | | | |
| | | | **SILTY SAND (SM)**<br>loose, moist, brown, fine sand | SM | 10 | ⧓ | 16 | | 19 |
| | | | | | 8 | ⧓ | 6 | | |
| | 5 | | **POORLY GRADED SAND (SP)**<br>loose, moist, brown, fine sand | SP | | | | | |
| | | | **SILTY SAND (SM)**<br>medium dense, moist, dark brown, fine to coarse sand, trace fine gravel (sub-angular/rounded) | SM | 12 | ⋈ | 16 | | 16 |
| ▽ | | | **POORLY GRADED SAND (SP)**<br>medium dense, wet, brown, fine to coarse sand, fine gravel (sub-angular/rounded) | SP | | | | | |
| | 10 | | | | 11 | ⋈ | | | |
| | 15 | | loose | | 8 | ⋈ | | | |
| | 20 | | dense | | 31 | ⋈ | | | |
| | | | **POORLY GRADED SAND WITH SILT (SP-SM)**<br>very dense, wet, dark brown, fine to coarse sand, fine gravel (sub-angular/rounded) | SP-SM | | | | | |
| | 25 | | | | 51 | ⋈ | | | |
| | | | **SANDY SILT (ML)**<br>hard, moist, dark gray, low plasticity, fine sand | ML | | | | | |
| | 30 | | | | | | | | |

*Continued Next Page*

GROUND WATER OBSERVATIONS:
▽ : FREE GROUND WATER MEASURED DURING DRILLING AT 9.5 FEET

LA_CORP.GDT 3/17/16 MV*

# EXPLORATORY BORING: EB-1 Cont'd

Sheet 2 of 2

DRILL RIG: TRUCK MOBILE CME-75
BORING TYPE: 8-INCH HOLLOW STEM AUGER
LOGGED BY: AC
START DATE: 3-3-16 FINISH DATE: 3-3-16

PROJECT NO: 252341
PROJECT: RIVERFRONT APARTMENTS
LOCATION: SANTA CRUZ, CA
COMPLETION DEPTH: 41.5 FT.

This log is a part of a report by TRC, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

Undrained Shear Strength (ksf)
○ Pocket Penetrometer
△ Torvane
● Unconfined Compression
▲ U-U Triaxial Compression
1.0 2.0 3.0 4.0

| ELEVATION (FT) | DEPTH (FT) | SOIL LEGEND | MATERIAL DESCRIPTION AND REMARKS | SOIL TYPE | PENETRATION RESISTANCE (BLOWS/FT.) | SAMPLER | MOISTURE CONTENT (%) | DRY DENSITY (PCF) | PERCENT PASSING NO. 200 SIEVE | Undrained Shear Strength (ksf) |
|---|---|---|---|---|---|---|---|---|---|---|
| | 30 | | **SANDY SILT (ML)**<br>hard, moist, dark gray, low plasticity, fine sand | | 92 | ☒ | 48 | | | ○ |
| | 35 | | dark greenish gray | ML | 67 | ☒ | 38 | | | ○ |
| | 40 | | | | 51 | ☒ | 47 | | | ○ |
| | | | Bottom refusal at 41.5 feet | | | | | | | |
| | 45 | | | | | | | | | |
| | 50 | | | | | | | | | |
| | 55 | | | | | | | | | |
| | 60 | | | | | | | | | |

GROUND WATER OBSERVATIONS:
▽ : FREE GROUND WATER MEASURED DURING DRILLING AT 9.5 FEET

# EXPLORATORY BORING: EB-2

Sheet 1 of 1

| | |
|---|---|
| DRILL RIG: TRUCK MOBILE CME-75 | PROJECT NO: 252341 |
| BORING TYPE: 8-INCH HOLLOW STEM AUGER | PROJECT: RIVERFRONT APARTMENTS |
| LOGGED BY: AC | LOCATION: SANTA CRUZ, CA |
| START DATE: 3-3-16 FINISH DATE: 3-3-16 | COMPLETION DEPTH: 27.0 FT. |

This log is a part of a report by TRC, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

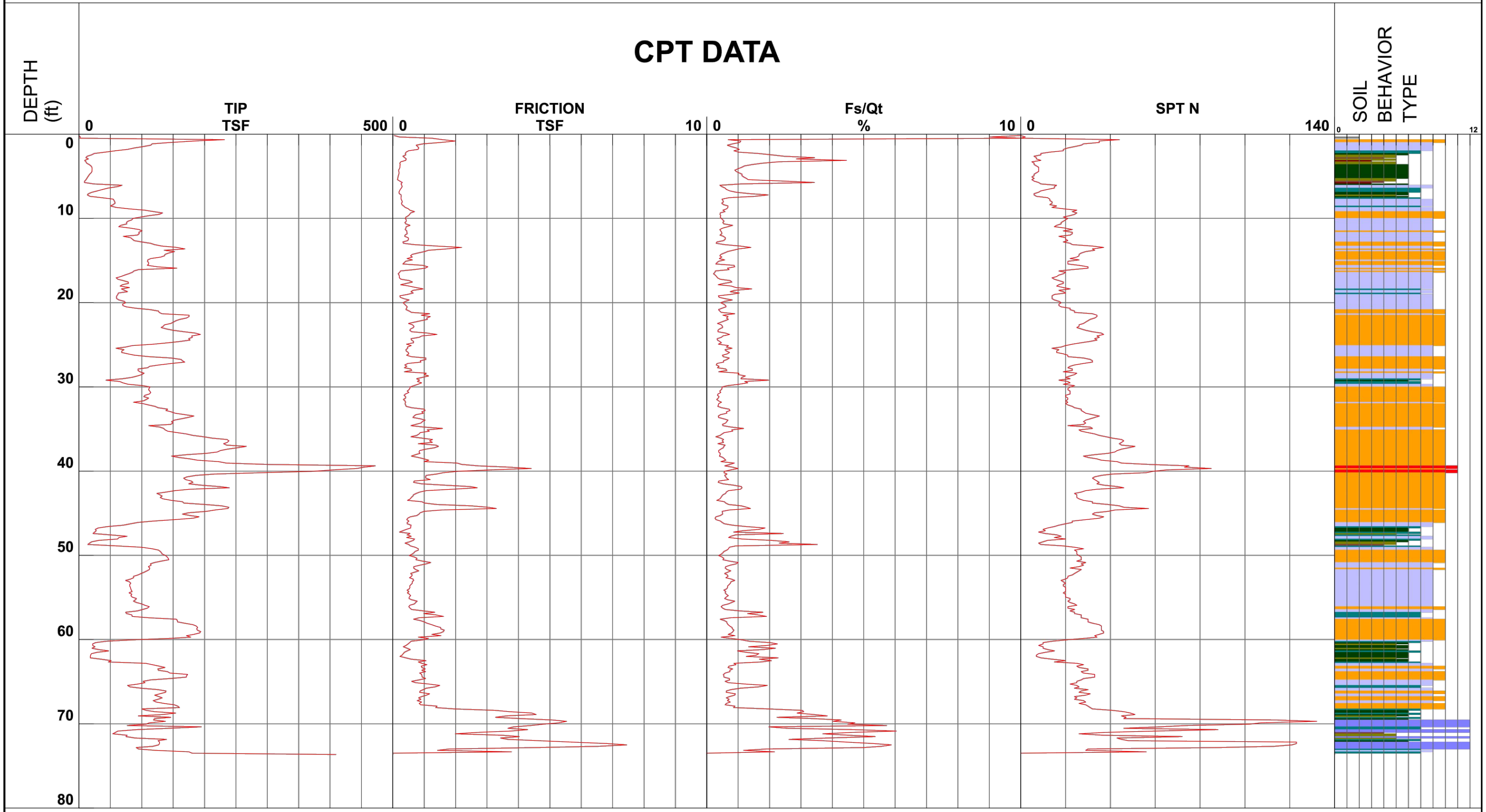
Undrained Shear Strength (ksf)
○ Pocket Penetrometer
△ Torvane
● Unconfined Compression
▲ U-U Triaxial Compression

| ELEVATION (FT) | DEPTH (FT) | SOIL LEGEND | MATERIAL DESCRIPTION AND REMARKS<br>SURFACE ELEVATION: | SOIL TYPE | PENETRATION RESISTANCE (BLOWS/FT.) | SAMPLER | MOISTURE CONTENT (%) | DRY DENSITY (PCF) | PERCENT PASSING NO. 200 SIEVE | 1.0 2.0 3.0 4.0 |
|---|---|---|---|---|---|---|---|---|---|---|
| | 0 | | **1" of AC**<br>straight drilling to 24.5 feet (no sampling) | AC/AB | | | | | | |
| | 5 | | | | | | | | | |
| ▽ | | | | | | | | | | |
| | 10 | | | | | | | | | |
| | 15 | | | | | | | | | |
| | 20 | | | | | | | | | |
| | 25 | | **SANDY SILT (ML)**<br>hard, moist, dark greenish gray, low plasticity, fine sand | ML | 40<br>50/3 | ⊠ | 54 | | | |
| | | | Boring refusal at 27 feet | | | | | | | |
| | 30 | | | | | | | | | |

GROUND WATER OBSERVATIONS:
▽ : FREE GROUND WATER MEASURED DURING DRILLING AT 8.5 FEET

LA_CORP.GDT 3/17/16 MV*

Net Area Ratio .8



■ 1 - sensitive fine grained

■ 2 - organic material

■ 3 - clay

■ 4 - silty clay to clay

■ 5 - clayey silt to silty clay

■ 6 - sandy silt to clayey silt

■ 7 - silty sand to sandy silt

■ 8 - sand to silty sand

■ 9 - sand

■ 10 - gravelly sand to sand

■ 11 - very stiff fine grained (*)

■ 12 - sand to clayey sand (*)

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

## CONE PENETRATION TEST
## CPT-1

Riverfront Apartments
Front Street
Santa Cruz, California

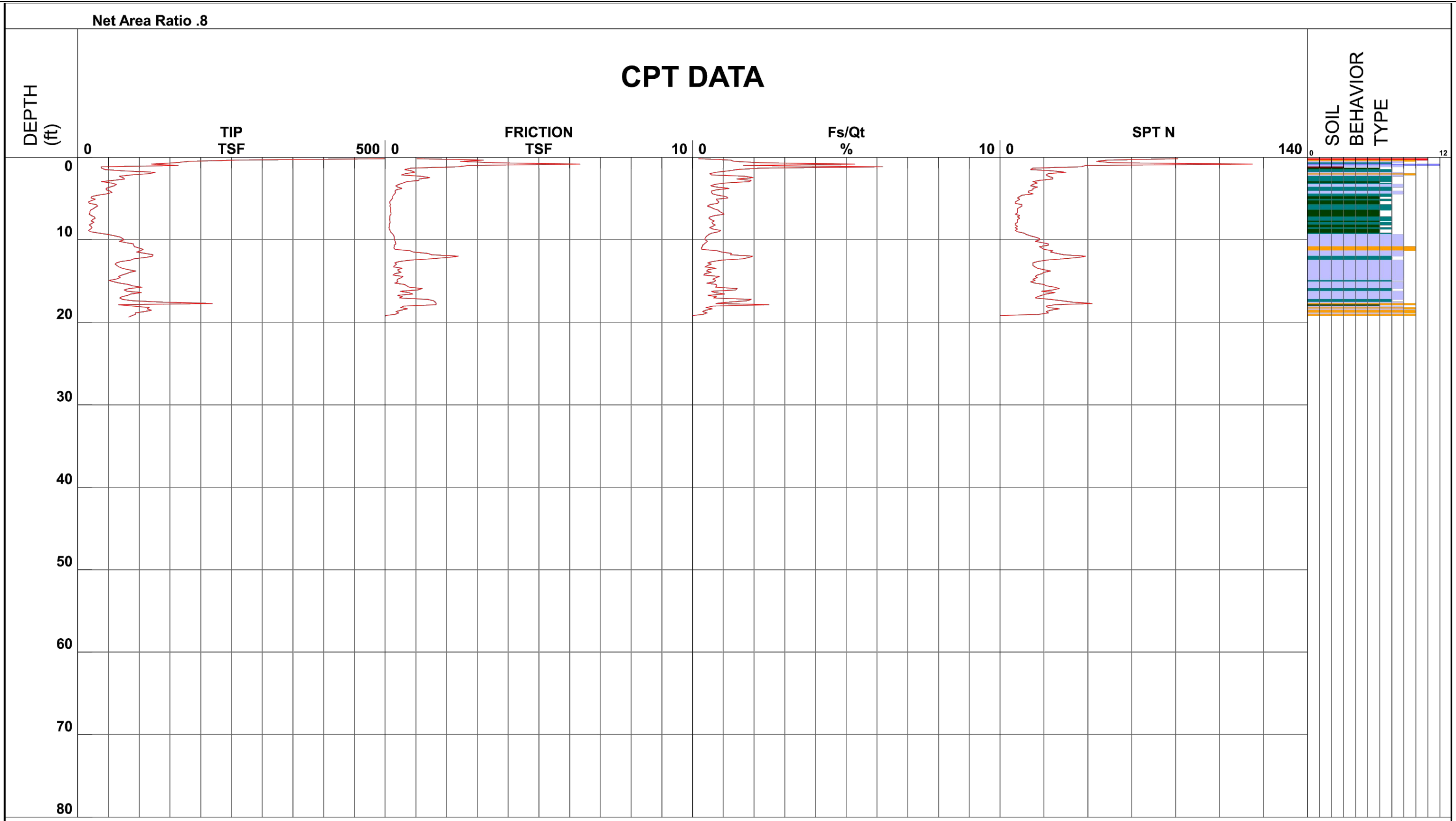
TRC

252341

**CPT-1**

FILE NAME: Z:\CAD_DRAW\Current\Riverfront Apts_Santa Cruz\CPT-1.dwg | Layout Tab: 8x11

Net Area Ratio .8



■ 1 - sensitive fine grained
■ 2 - organic material
■ 3 - clay

■ 4 - silty clay to clay
■ 5 - clayey silt to silty clay
■ 6 - sandy silt to clayey silt

■ 7 - silty sand to sandy silt
■ 8 - sand to silty sand
■ 9 - sand

■ 10 - gravelly sand to sand
■ 11 - very stiff fine grained (*)
■ 12 - sand to clayey sand (*)

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

## CONE PENETRATION TEST
## CPT-2

Riverfront Apartments
Front Street
Santa Cruz, California

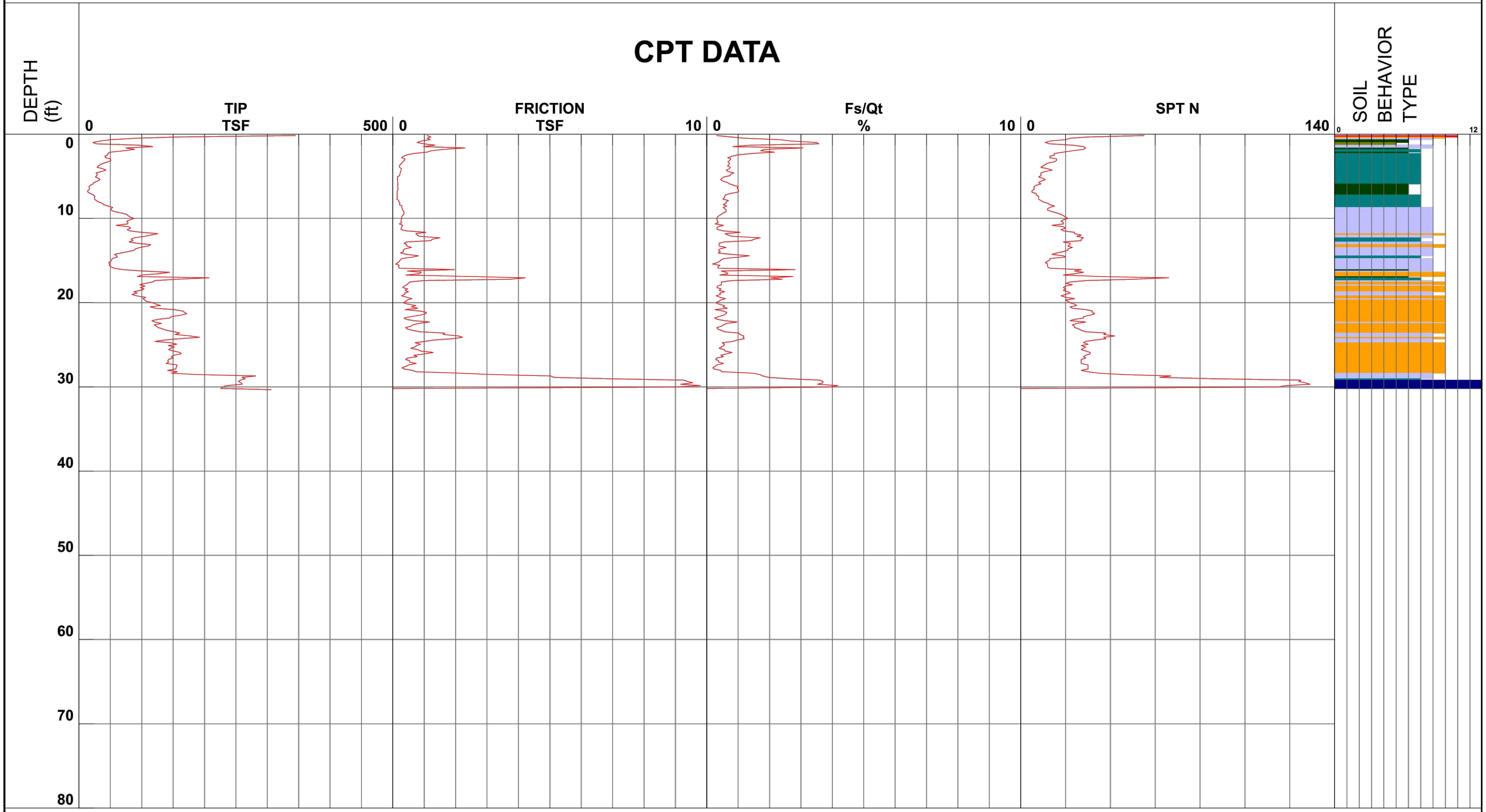
TRC

252341

**CPT-2**

FILE NAME: Z:\CAD_DRAW\Current\Riverfront Apts_Santa Cruz\CPT-2.dwg | Layout Tab: 8x11

Net Area Ratio .8



⬜ 1 - sensitive fine grained
⬛ 2 - organic material
🟥 3 - clay
🟫 4 - silty clay to clay
🟨 5 - clayey silt to silty clay
🟩 6 - sandy silt to clayey silt
🟦 7 - silty sand to sandy silt
🟪 8 - sand to silty sand
🟧 9 - sand
🟥 10 - gravelly sand to sand
🟪 11 - very stiff fine grained (*)
🟦 12 - sand to clayey sand (*)

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

# CONE PENETRATION TEST
# CPT-2A

Riverfront Apartments
Front Street
Santa Cruz, California

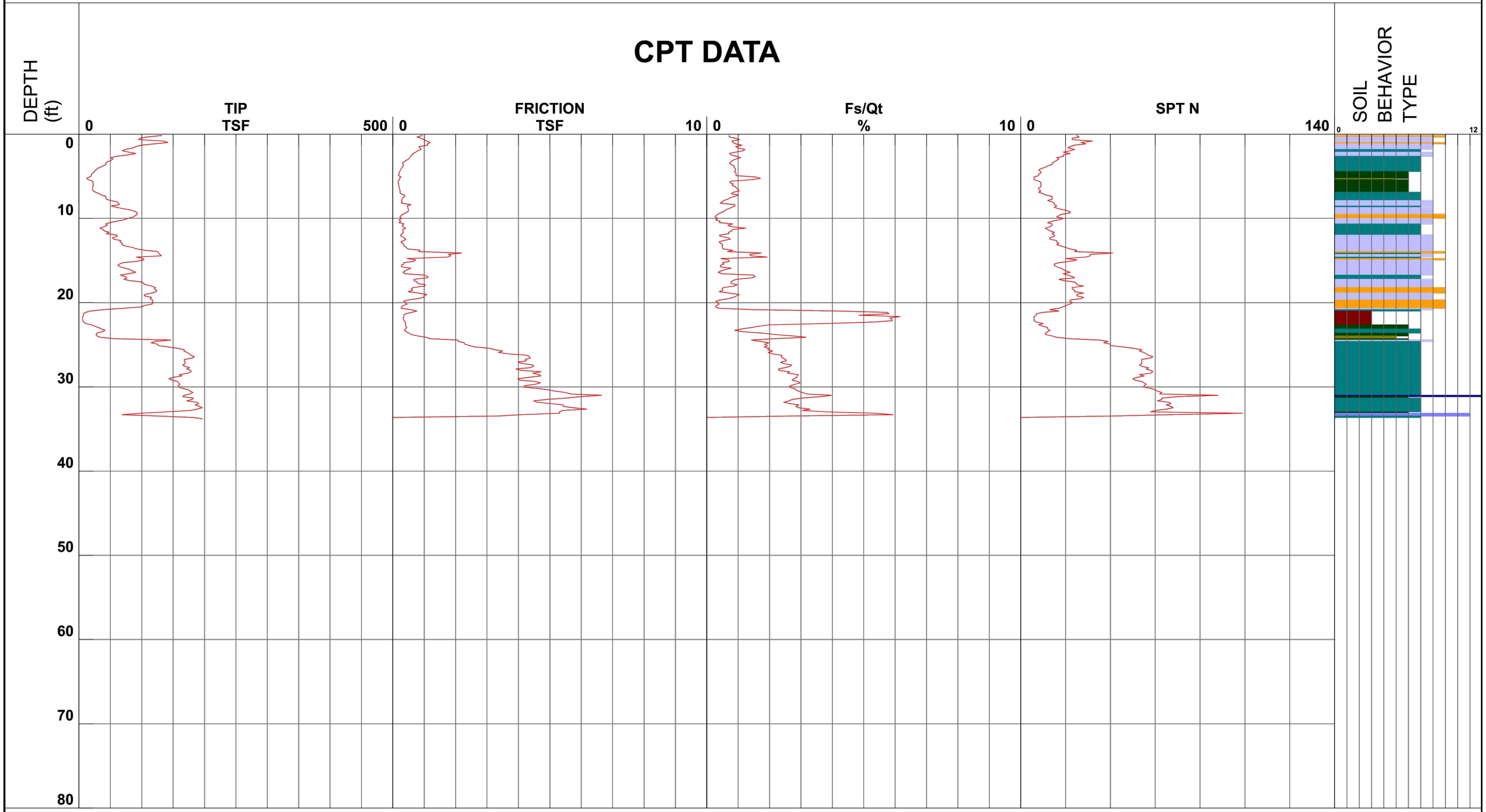
TRC

252341

**CPT-2A**

FILE NAME: Z:\CAD_DRAW\Current\Riverfront Apts_Santa Cruz\CPT-2A.dwg | Layout Tab: 8x11

Net Area Ratio .8



■ 1 - sensitive fine grained
■ 2 - organic material
■ 3 - clay
■ 4 - silty clay to clay
■ 5 - clayey silt to silty clay
■ 6 - sandy silt to clayey silt
■ 7 - silty sand to sandy silt
■ 8 - sand to silty sand
■ 9 - sand
■ 10 - gravelly sand to sand
■ 11 - very stiff fine grained (*)
■ 12 - sand to clayey sand (*)

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

## CONE PENETRATION TEST
## CPT-3

Riverfront Apartments
Front Street
Santa Cruz, California

252341

**CPT-3**

# APPENDIX B

# LABORATORY PROGRAM

The laboratory testing program was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the soils underlying the site and to aid in verifying soil classification.

**Moisture Content:** The natural water content was measured (ASTM D2216) on 7 samples of the materials recovered from the boring. These water contents are recorded on the boring log at the appropriate sample depths.

**Washed Sieve Analyses:** The percent soil fraction passing the No. 200 sieve (ASTM D1140) was performed on two samples of the subsurface soils to aid in the classification of these soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

**R-Value:** An R-value resistance test (California Test Method No. 301) was performed on a representative sample of the surface soils at the site to provide data for the pavement design. The test indicated an R-value of 72 at an exudation pressure of 300 pounds per square inch. The results of the test are presented in this appendix.

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# R-value Test Report (Caltrans 301)

**Job No.:** 028-2509 **Date:** 03/10/16

**Client:** TRC **Tested** MD

**Project:** Riverfront Apts - 252341 **Reduced** RU

**Sample** EB-1,Bulk @ 0-5' **Checked** DC

**Soil Type:** Olive Silty SAND w/ Gravel

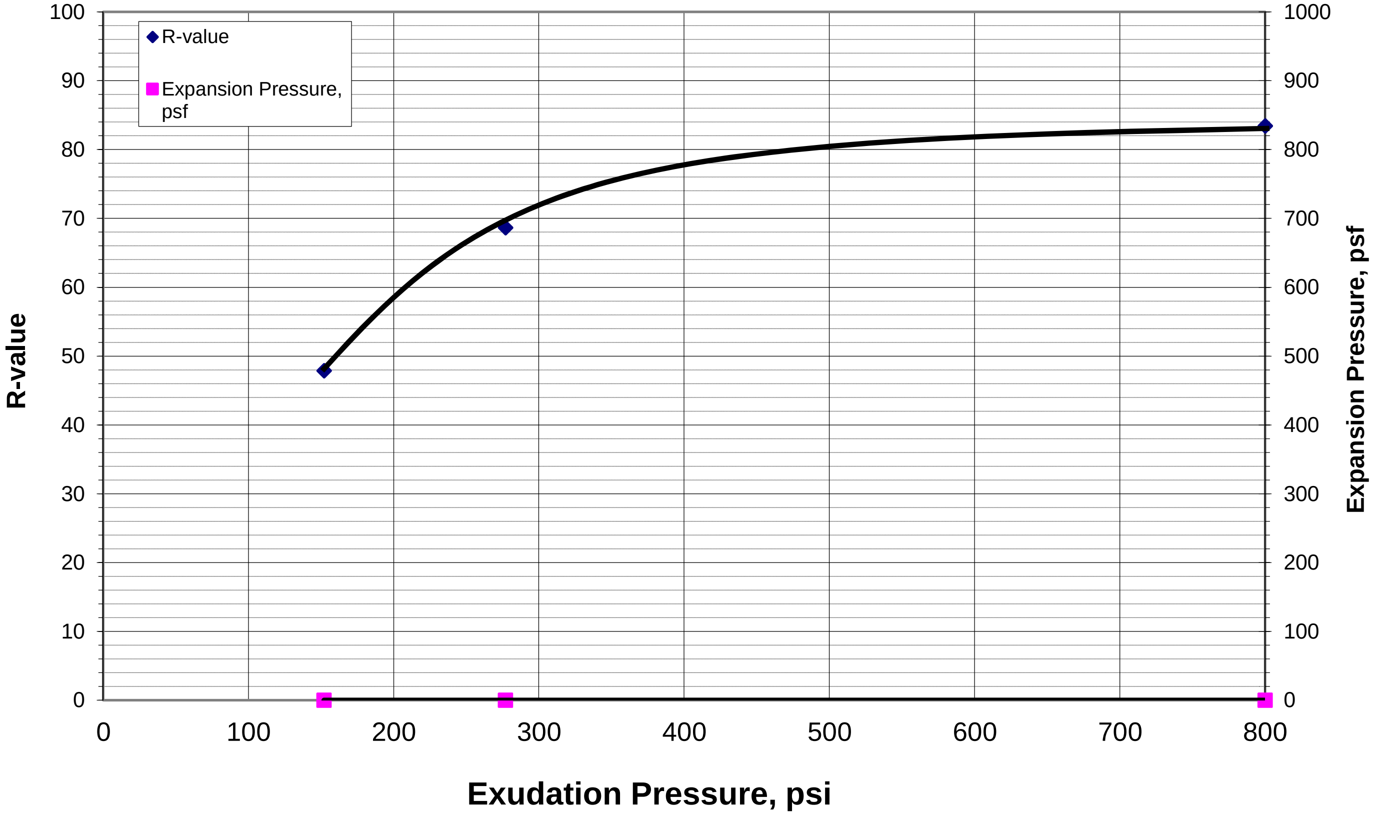
**Initial Moisture,** 5.8%

**R-value by Stabilometer** 72

**Expansion Pressure** 0 psf

| Specimen Number | A | B | C | D |
|---|---|---|---|---|
| **Exudation Pressure, psi** | 152 | 800 | 277 | |
| **Prepared Weight, grams** | 1200 | 1200 | 1200 | |
| **Final Water Added, grams/cc** | 47 | 26 | 35 | |
| **Weight of Soil & Mold, grams** | 3146 | 3160 | 3172 | |
| **Weight of Mold, grams** | 2099 | 2098 | 2102 | |
| **Height After Compaction, in.** | 2.36 | 2.38 | 2.39 | |
| **Moisture Content, %** | 10.0 | 8.1 | 8.9 | |
| **Dry Density, pcf** | 122.2 | 125.0 | 124.5 | |
| **Expansion Pressure, psf** | 0.0 | 0.0 | 0.0 | |
| **Stabilometer @ 1000** | | | | |
| **Stabilometer @ 2000** | 64 | 20 | 38 | |
| **Turns Displacement** | 3.5 | 3.2 | 3.31 | |
| **R-value** | 48 | 83 | 69 | |

**Remarks:**



# Corrosivity Tests Summary

**CTL #** 028-2509 **Date:** 3/9/2016 **Tested By:** PJ **Checked:** PJ
**Client:** TRC **Project:** Riverfront Apts **Proj. No:** 252341
**Remarks:**

| **Sample Location or ID** | | | **Resistivity @ 15.5 °C (Ohm-cm)** | | | **Chloride** | **Sulfate** | | **pH** | **ORP** | | **Sulfide** | **Moisture** | **Soil Visual Description** |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | **As Rec.** | **Min** | **Sat.** | **mg/kg** | **mg/kg** | **%** | | (Redox) | | Qualitative | At Test | |
| | | | | | | Dry Wt. | Dry Wt. | Dry Wt. | | $E_H$ (mv) | At Test | by Lead | **%** | |
| **Boring** | **Sample, No.** | **Depth, ft.** | ASTM G57 | Cal 643 | ASTM G57 | ASTM D4327 | ASTM D4327 | ASTM D4327 | ASTM G51 | ASTM G200 | Temp °C | Acetate Paper | ASTM D2216 | |
| EB-1 | 1A | 2.5 | - | - | 9,374 | <2 | 77 | 0.0077 | 7.7 | - | - | - | 20.7 | Olive Sandy SILT |

# Corrosivity Tests Summary

**CTL #** 028-2508 **Date:** 3/9/2016 **Tested By:** PJ **Checked:** PJ

**Client:** TRC **Project:** Riverfront Apts **Proj. No:** 252341

**Remarks:**

| Sample Location or ID | | | Resistivity @ 15.5 °C (Ohm-cm) | | | Chloride | Sulfate | | pH | ORP | | Sulfide | Moisture | Soil Visual Description |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | As Rec. | Min | Sat. | mg/kg | mg/kg | % | | (Redox) | | Qualitative | At Test | |
| | | | | | | Dry Wt. | Dry Wt. | Dry Wt. | | $E_H$ (mv) | At Test | by Lead | % | |
| Boring | Sample, No. | Depth, ft. | ASTM G57 | Cal 643 | ASTM G57 | ASTM D4327 | ASTM D4327 | ASTM D4327 | ASTM G51 | ASTM G200 | Temp °C | Acetate Paper | ASTM D2216 | |
| EB-1 | 2A | 4.5 | - | - | 12,478 | <2 | 68 | 0.0068 | 7.9 | - | - | - | 8.1 | Olive Silty SAND |
| | | | | | | | | | | | | | | |
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