GRAY GEOTECH

GEOTECHNICAL REPORT

Proposed Building Additions and Relocation
4110 Alhambra Way, Martinez, CA
Project No. 20-096-01

November 2, 2020

Submitted to:

American Housing Inc.

c/o Dinesh Sawhney

Table of Contents

1 Introduction and Project Description

At your request, we are providing this geotechnical report to assist with your planning and design for improvements to an assisted living facility located at 4110 Alhambra Way in Martinez, California. We have based this report on our review of publicly available regional geologic maps of the area, our visual reconnaissance to observe the ground surface at the proposed building areas, and a subsurface exploration consisting of two borings and analysis of the slope stability of a creek bank along the western boundary of the site.

This report is limited to the evaluation of the soil conditions near the proposed improvements. Evaluation of other areas of the property or for different future improvements is not within the scope of this report.

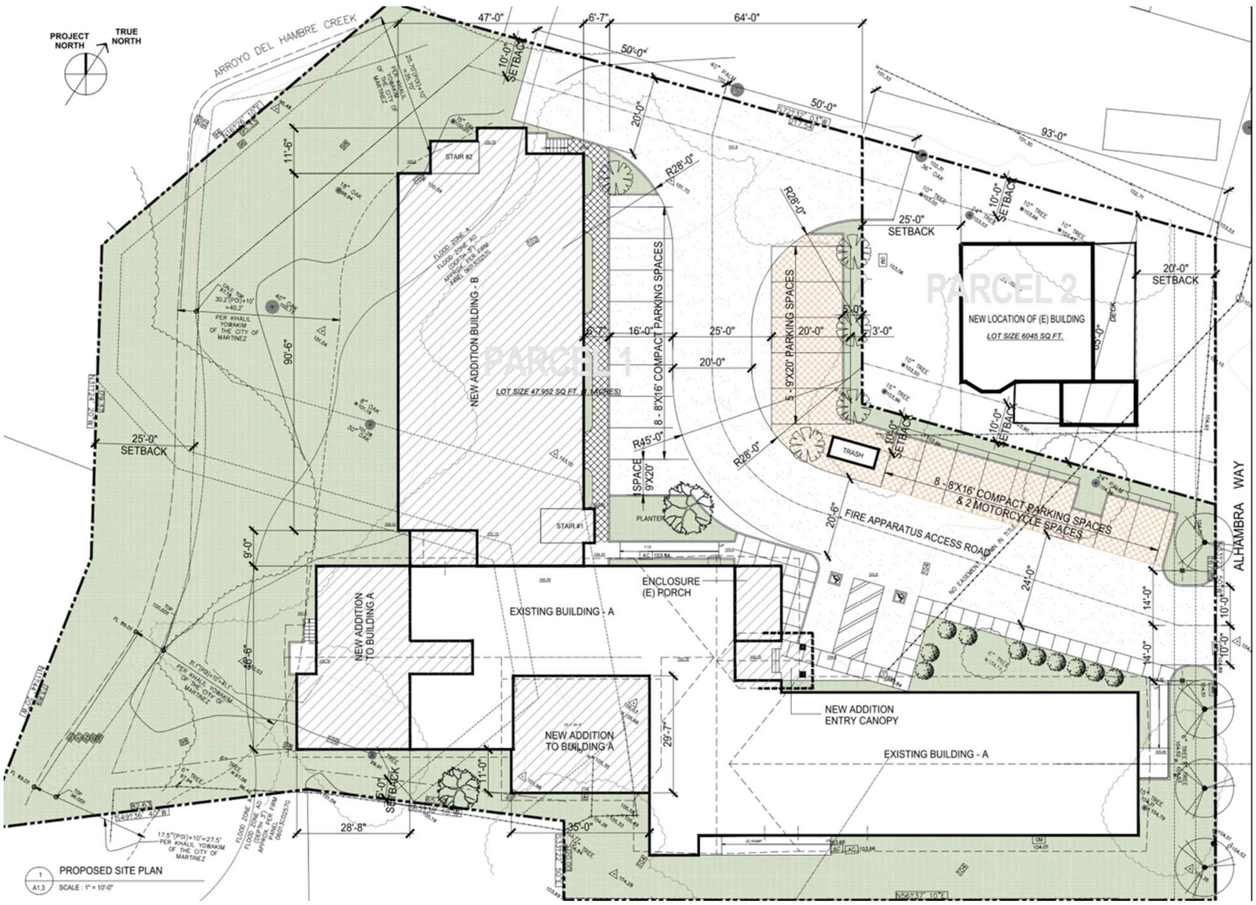
1.1 Site Description

The project area is property of about 1.13 acres located in the City of Martinez. The parcel has a legal identification of APN 370-291-013, and currently zoned as a residential retirement home. There are three existing buildings located on the site; a two-story timber framed dwelling, a single-story housing building (which appears to have been the main facility building for the past retirement home), and a small detached garage. County records show that original construction was circa 1958, which we assume applies to the single-story housing building. Based on our review of historic aerial photographs, it appears that the two-story dwelling was built sometime prior to 1946 and the property's past land use was as an orchard.

Figure 1: Site Vicinity Map

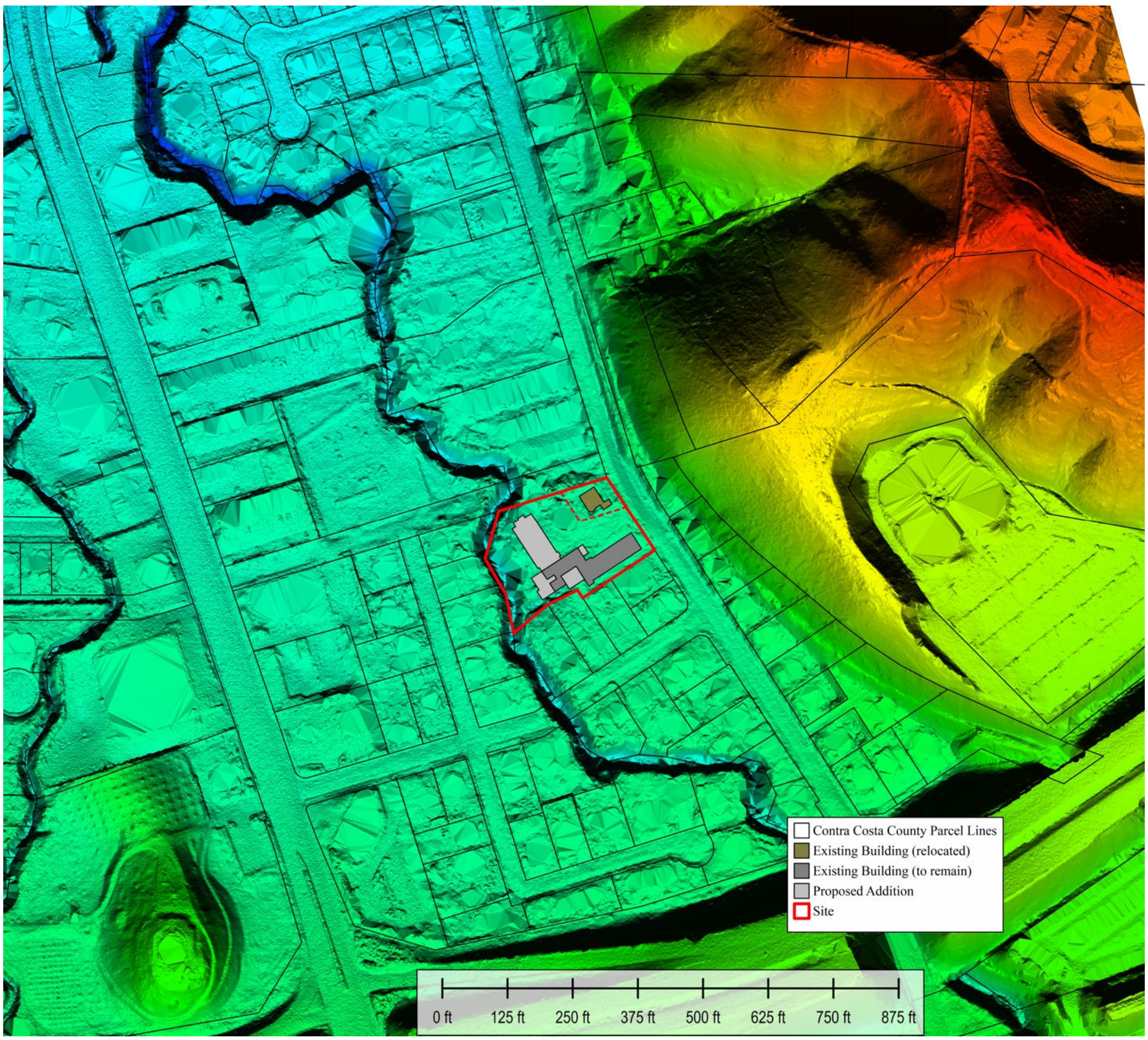
We understand that current plans to improve the property consist of demolishing the garage building, constructing additions to the single-story building to expand the footprint in three locations, and relocating the two-story building to the east. The project will involve a subdivision to form a new parcel for this building. An excerpt from Kodama Diseno Architects and Planners is shown below.

Figure 2: Proposed Site Plan



We obtained data from a 2018 LiDAR survey of the area, which was performed under direction of the USGS. This LiDAR survey consists of aerial laser measurements generating point elevations of the structures, vegetation and the ground surface. With this, we were able to generate a high-resolution terrain map for use in our analysis. As shown below, the site is located on a relatively flat parcel, there is a small north-south trending ridgeline within the properties to the east of Alhambra Way, and a flat valley floor extending to the west, which drains to the north by several meandering creeks. The Arroyo de Hambre Creek is located along the western boundary of the site. Based on the LiDAR data, the creek has a width of between 30 to 55 feet (between tops of the bank) and a channel depth of between 8 to 13 feet. We understand that portions of the site are within a flood zone, however hydrologic analysis and assessing the risk of flooding is not within the scope of this report.

Figure 3: Site Topography

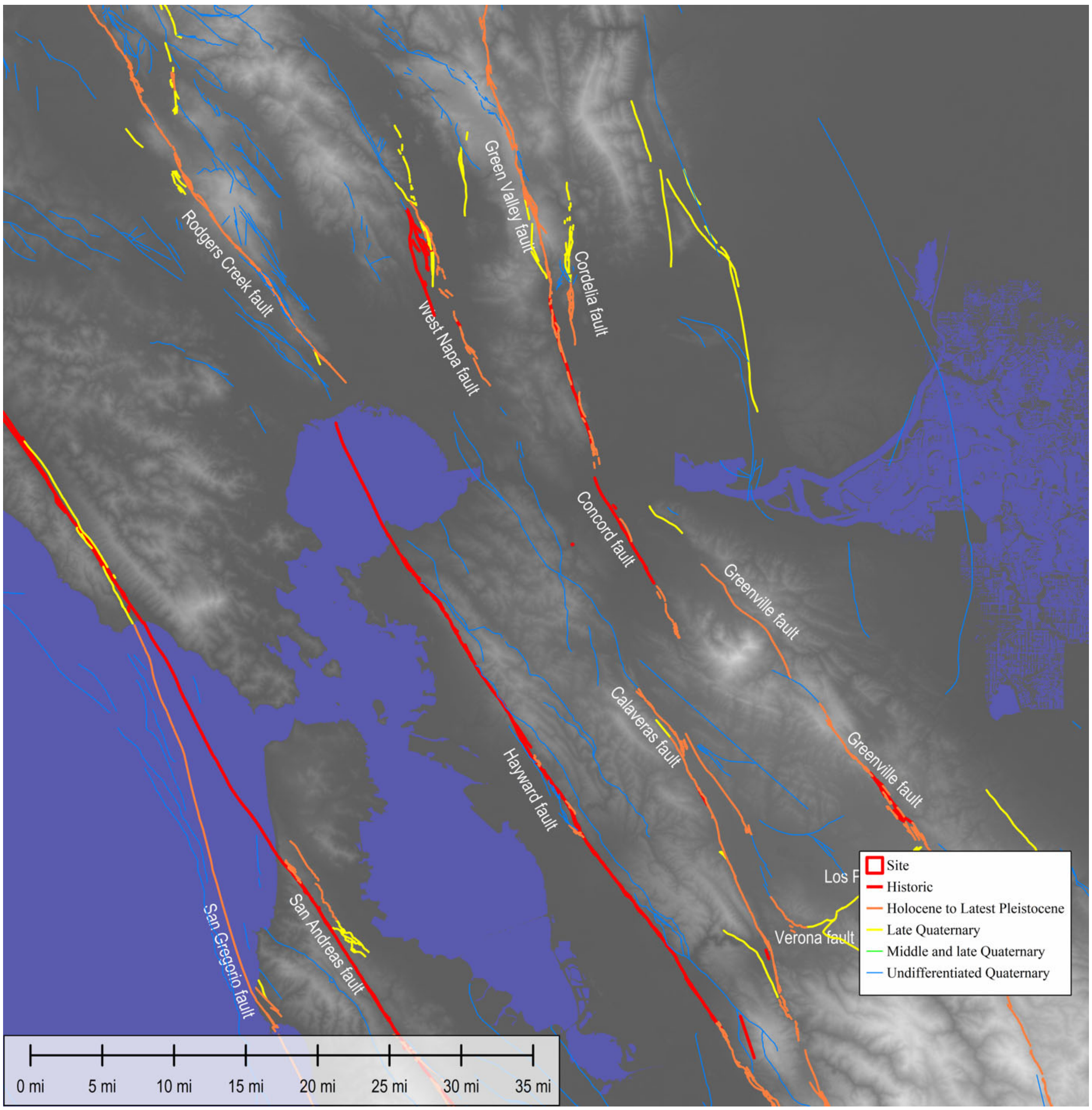


2 Findings

2.1 Faults and Seismicity

The San Francisco Bay Area is a seismically active area; at least 20 strong earthquakes measuring M6 or greater have occurred in the Bay Area in the last 200 years (Ellsworth, 1990), many of these would have likely resulted in moderate to severe ground shaking at the site. The map below shows the location of faults that have been historic and ancient earthquake sources in the San Francisco Bay Area, classed based on the age of last known movement.

Figure 4: USGS Quaternary Fault Map



It is likely the site will experience one or more episodes of strong ground shaking during the design life of the proposed improvements. The United States Geologic Survey and the State of California have developed an earthquake rupture forecast; an estimate on “when and where” a future earthquake might occur amongst the State’s many faults. This model (referred to as UCERF3) provides a 63 percent probability that M6.7 or greater earthquake will occur in the Bay Area by 2044, and classes the Hayward fault and the Calaveras fault as two of the area’s “particularly ready faults”.

The table below summarizes significant active faults located within 50 km of the project site, including estimated slip rates and Maximum Moment Magnitude. The maximum moment magnitude earthquake (Mmax) is defined as the largest earthquake that a given fault is considered capable of generating.

Table 1: Distance to known active faults

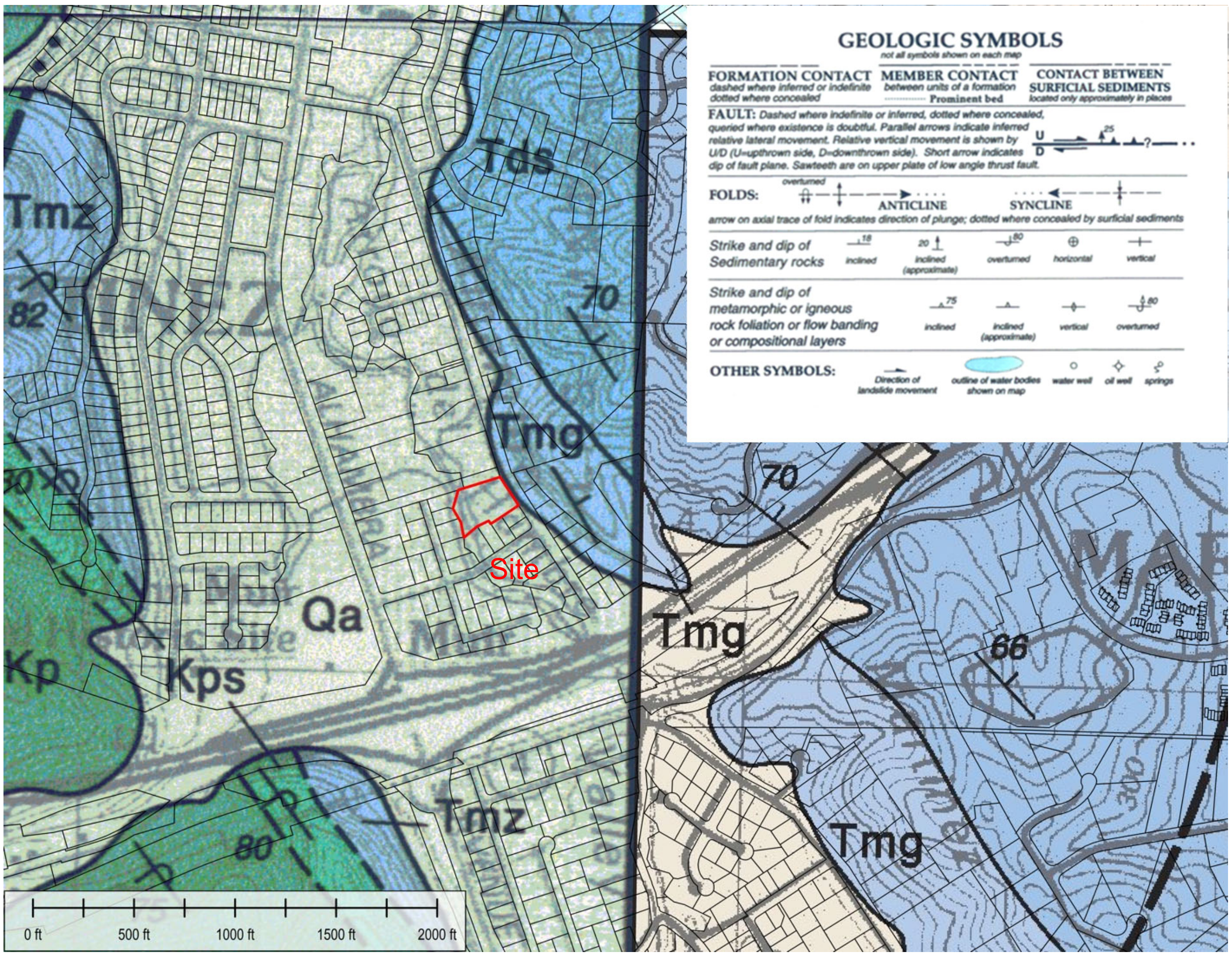
Fault Name	Distance to Site	Maximum Moment Magnitude	Slip Rate (mm/year)
Calaveras Fault	18 km	7.0	6 ±2
Concord - Green Valley Fault	5.7 km	6.8	4 ±2
Greenville Fault	14 km	6.2	2 ±1
Hayward Fault	16 km	7.0	9 ±2
Los Positas Fault	50 km	6.4	0.5 mm/yr
Pleasanton Fault	24 km	6.6	--
Rodgers Creek Fault	34 km	7.1	9 ±2
San Andreas Fault	46 km	7.9	24 ±3
Verona Fault	45 km	6.3	--
West Napa Fault	20 km	6.7	1 ±1

In addition to ground shaking, other seismic hazards include fault rupture or displacement. The State of California has prepared a series of maps known as Seismic Hazard Zone Maps (SHZM), which delineate regulatory hazard zones in accordance with the Alquist-Priolo Act. The initial hazard zone shown on the SHZM is along active earthquake faults. In addition, the SHZM have been periodically updated since 1972 to include other risks such as earthquake induced landslides and liquefaction. Projects within these zones require special studies (fault investigation reports) to attempt to identify the location of fault trace(s) and to confirm the age of the last fault activity or to determine the risk of property damage from liquefaction or landslide movement. The site is not located within an AP fault rupture zone, liquefaction zone or earthquake trigged landslide hazard zone.

2.2 Regional Geology

The site is located in the Coastal Range geomorphic province of California. The Coastal Range province is characterized by a series of nearly parallel northwest-trending mountain ranges and alluviated valleys that were formed from tectonic activity between the Pacific and North American Plates. Considerable faulting, deformation, and erosion have resulted in irregular topography and contacts between the various geologic units. A regional geologic map (below) shows the site is underlain by young (Holocene aged) alluvial soil, typically consisting of clay, silt and sand.

Figure 5: Regional Geology (Dibblee 2005)



A map prepared by the USGS (Nilsen 1975) identifies areas of hillside deposits that may be at risk to ground movement (based on interpretation of aerial photographs) to redflag sites that may require further site-specific investigation prior to development. Based on this map (shown below), there are no known landslides within the site or on adjacent properties which might affect the subject site.

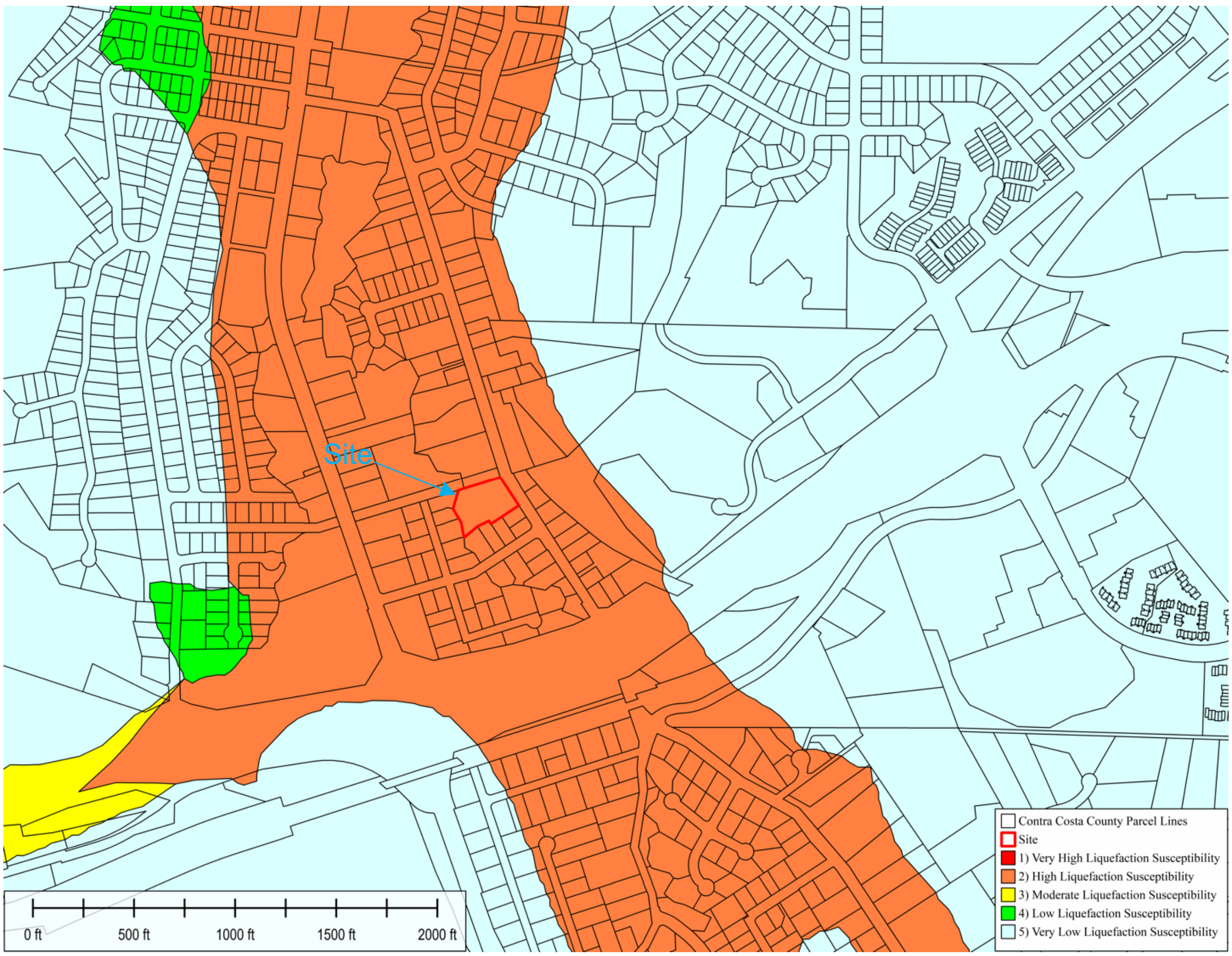
Figure 6: Distribution of Landslides and Earthflows (USGS 1975)

2.3 Liquefaction Susceptibility

Liquefaction is a phenomenon in which saturated loose granular and low plasticity soils have a temporary loss of strength due to cyclic stresses and increased pore pressure as a result of strong ground shaking caused by earthquakes. The potential consequences of liquefaction include settlement of the land and structures, loss of bearing capacity for foundation elements located within or near the liquefied soil layers, deformation and buoyancy effects on utilities and below groundwater structures, lateral spreading or displacement of the ground, and ground surface rupture and sand boils with large volumes of silt, sand and water ejected to the surface.

A USGS map shows region wide liquefaction susceptibility of the site to be high, where there are five susceptibility categories; very low, low, moderate, high and very high. As such, we have carried out a site-specific liquefaction hazard analysis, as described in Section 2.5 below.

Figure 7: Liquefaction Susceptibility Map (USGS OFR06-037)



2.4 Subsurface Investigations

We carried out a subsurface exploration consisting of two Cone Penetration Tests (CPTs) on October 13, 2020. The exploration locations are presented in Figure 8 below, and were advanced to a depth of 47 feet (CPT1) and 50 feet (CPT2) below the ground surface.

Figure 8: Exploration Site Plan

The CPT probe gathers raw data including cone tip resistance, friction sleeve resistance, and pore water pressure at 2.5 cm intervals during the test. This information is used to infer the soil type, soil density, consistency, and other engineering parameters. The CPT data indicates the top 6 to 10-feet consists of semi-granular silty clay and sandy silt, which is underlain by a soft clay that is at least 20 to 40 feet thick. CPT1 has an interbedded lens of sandy silt between 28 to 34 feet, and encountered bedrock at a depth of 46 feet. Bedrock was not encountered in CPT2, and this exploration was terminated in soft clay soil at a depth 50 feet. A pore pressure dissipation test was carried out at both CPT locations, which indicated that the depth to the water table was 16 feet at the time of our exploration.

2.5 Liquefaction and Creek Bank Stability Analysis

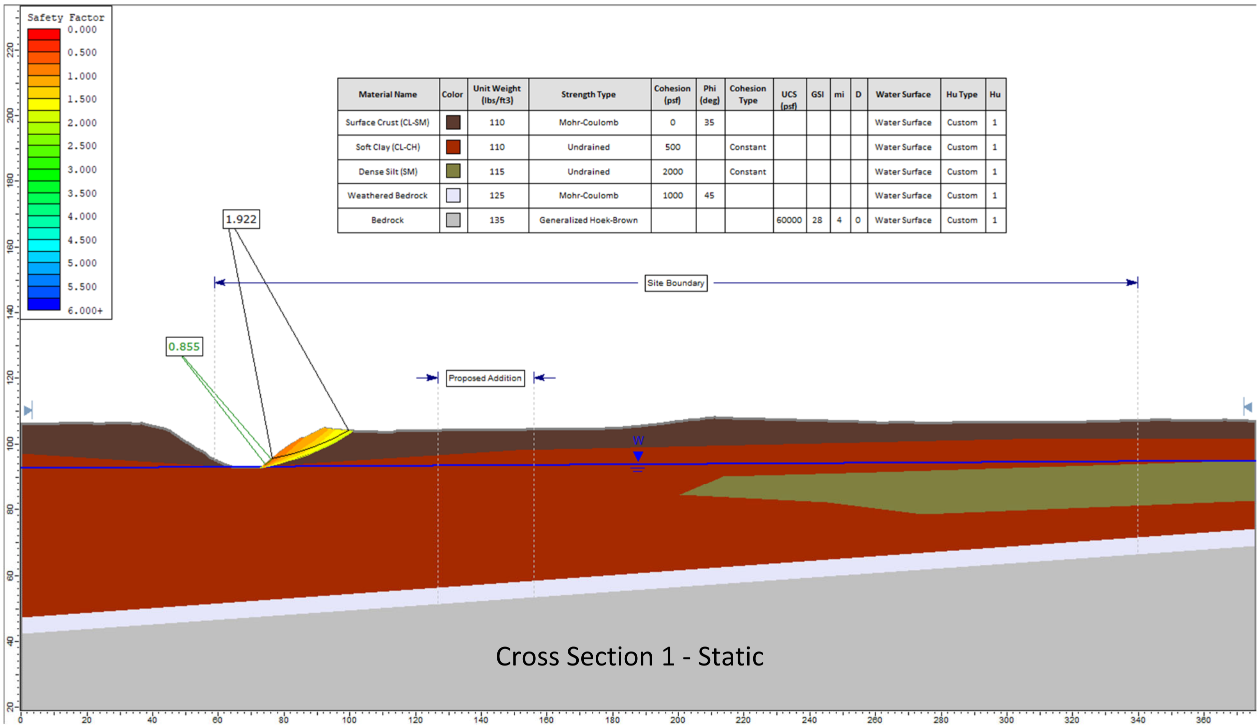
We have evaluated the risk of soil liquefaction based on the CPT exploration data, using ground motions of PGA of 0.67g and earthquake magnitude of Mw=6.7. We used a groundwater depth of 10 feet in our analysis, which was conservatively selected as a shallower depth than the groundwater level measured at the time of our field exploration. Based on the time of year and potential for variation due to rainfall or tidal effects at this site, it is our opinion that this is a reasonable depth for the purpose of liquefaction analysis.

Based on our analysis, we estimate that up to 1.7 inches of vertical settlement may occur in the case of a strong future earthquake, with between 9 to 12 inches of lateral displacement towards the creek. Based on the high fines content in the soil, there will not be a significant strength loss in the surface soils or significantly ejecta of sand and groundwater (sand boils) at the surface.

We performed a limit equilibrium slope stability analysis to check the risk of failures occurring within the near creek bank in both the static a seismic case. We based the strength of the soil considering the results of our field investigation, and show the generalized parameters for the soil and rock are shown in figures 9, 10 and 11. We performed a seismic analysis using Newmark sliding block model and ground motions approximately equivalent to an earthquake risk with a 5% probability in 50 years (modelled as a Magnitude 6.9Mw and PGA=0.6).

Based on this, it is our opinion that the site-specific liquefaction risk is low, does not require mitigation to meet the minimum performance levels required by Code. A shallow foundation system is allowable for this level of seismic risk, but it will require special design considerations (which are discussed later in this report). It is also our opinion that the static factor of safety in the area of the proposed additions is within an acceptable range (currently greater than 2.0). The static factor could be impacted (reduced) following a very strong storm, partculaly if any large trees fall into the creek or if the creek bank is scoured by high velocity flow. We recommend that the bank is re-evaluted in the case of a rare/damaging level storm occurs in the future.

Figure 9: Slope Stability – Cross Section 1



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	UCS (psf)	GSI	mi	D	Water Surface	Hu Type	Hu
Surface Crust (CL-SM)		110	Mohr-Coulomb	0	35						Water Surface	Custom	1
Soft Clay (CL-CH)		110	Undrained	500		Constant					Water Surface	Custom	1
Dense Silt (SM)		115	Undrained	2000		Constant					Water Surface	Custom	1
Weathered Bedrock		125	Mohr-Coulomb	1000	45						Water Surface	Custom	1
Bedrock		135	Generalized Hoek-Brown				60000	28	4	0	Water Surface	Custom	1

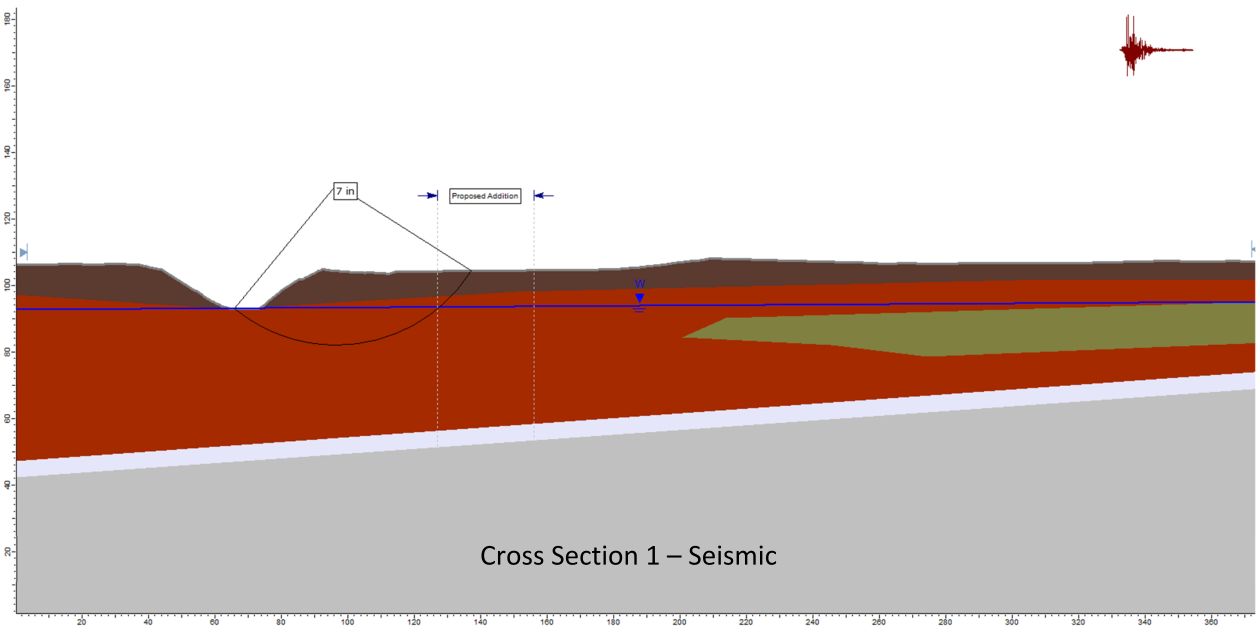
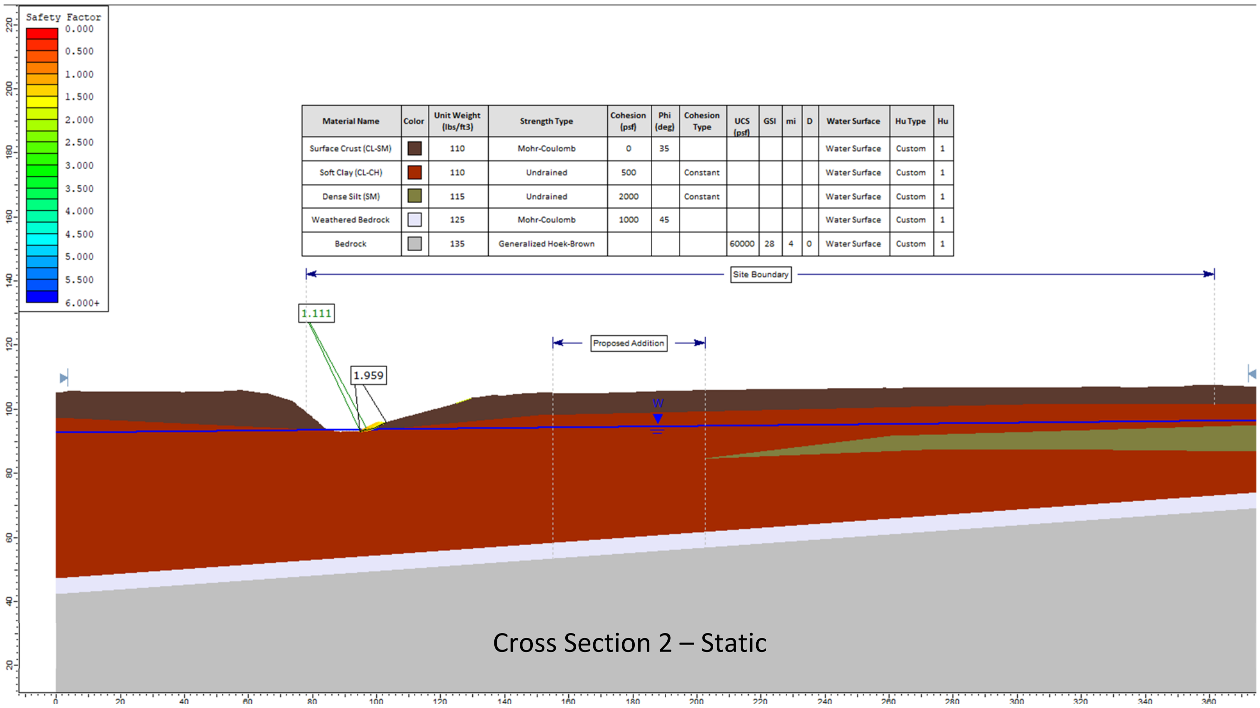


Figure 10: Slope Stability – Cross Section 2



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	UCS (psf)	GSI	mi	D	Water Surface	Hu Type	Hu
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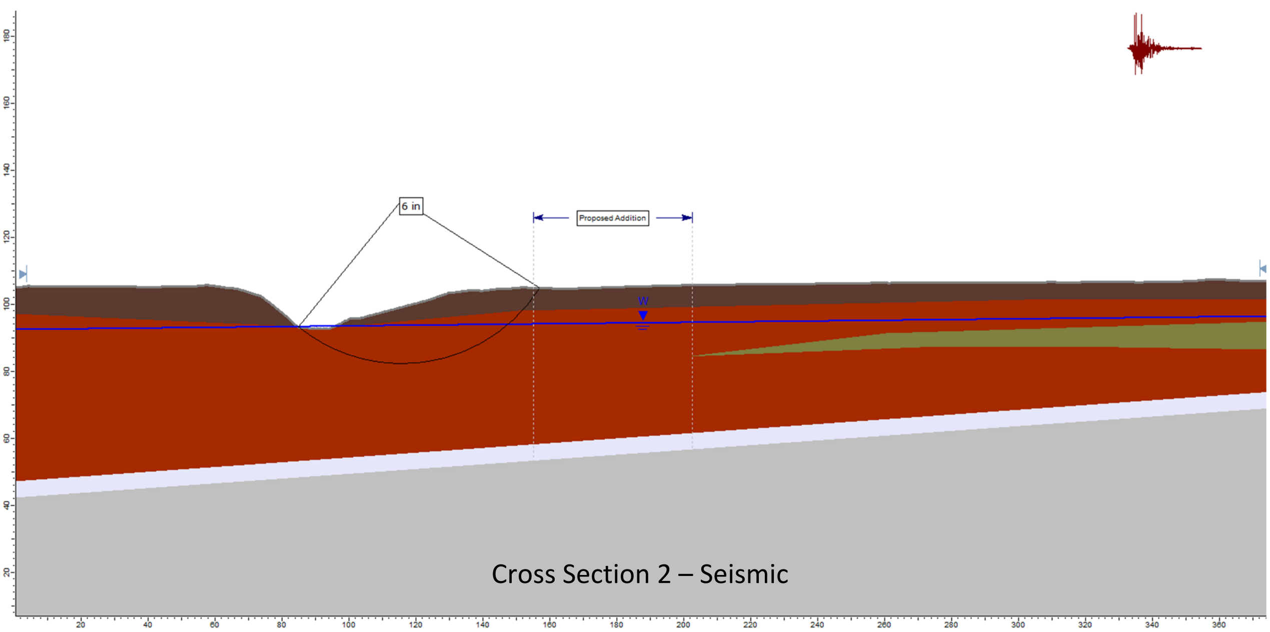
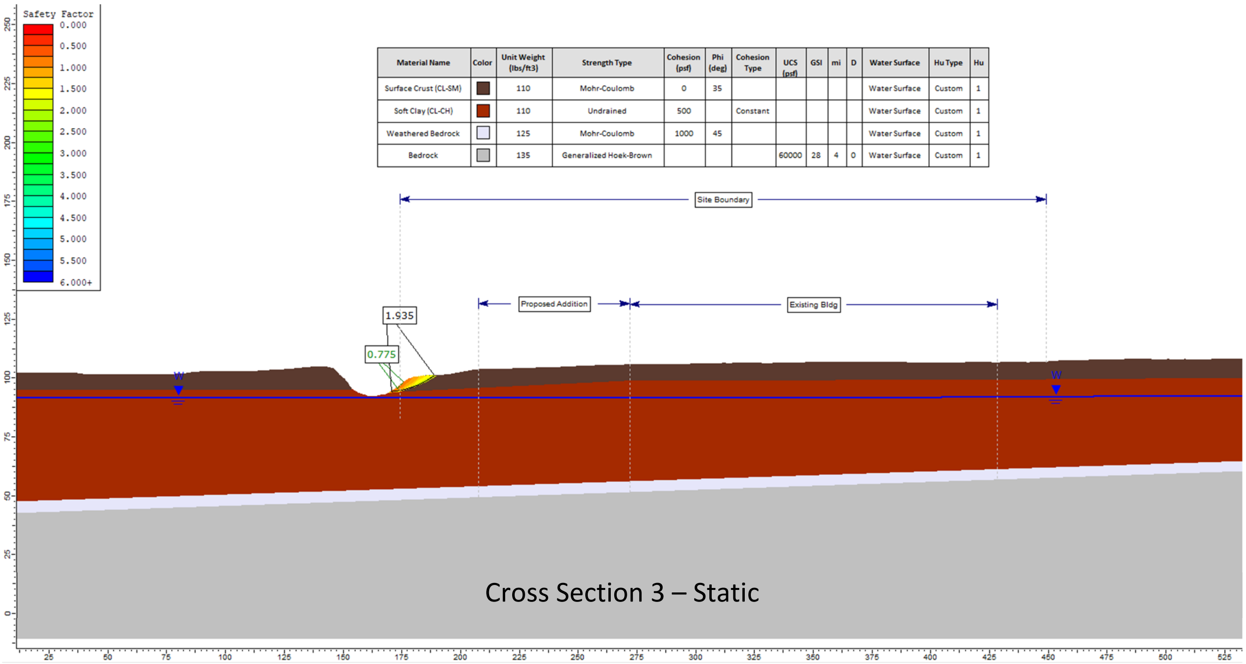
Weathered Bedrock		125	Mohr-Coulomb	1000	45						Water Surface	Custom	1
Bedrock		135	Generalized Hoek-Brown				60000	28	4	0	Water Surface	Custom	1

Cross Section 2 – Static



Cross Section 2 – Seismic

Figure 11: Slope Stability – Cross Section 3



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	UCS (psf)	GSI	mi	D	Water Surface	Hu Type	Hu
Surface Crust (CL-SM)		110	Mohr-Coulomb	0	35						Water Surface	Custom	1
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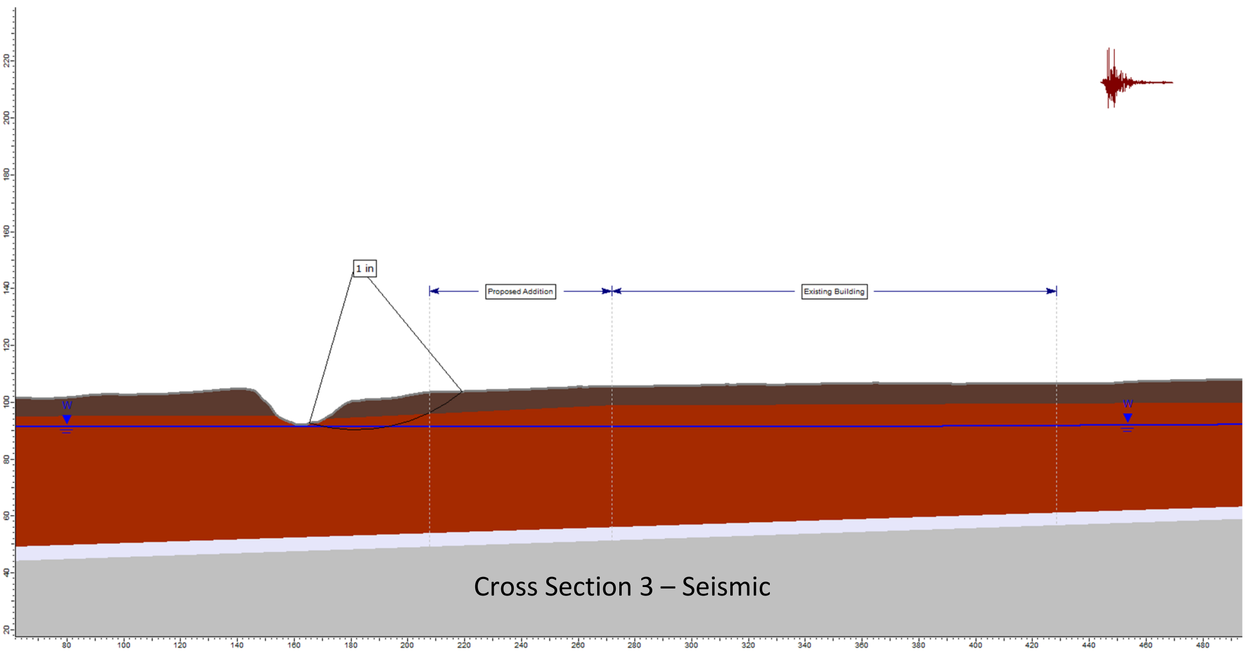


3 Assessment of Findings

3.1 General

It is our opinion that the site is suitable for the proposed project from a geotechnical point of view, provided the recommendations presented in this report and standard development practices are incorporated in the design and construction of the project.

The primary geotechnical considerations for the project are the potential of unsuitable fill, buried objects or tree roots from past land use, differential movement between portions of the existing structure and new additions, ground motions in a future seismic event, and managing flood risk and storm water.

3.2 Earthwork

3.2.1 Site Preparation

The building site and areas to receive fill should be stripped of all topsoil, soil with heavy roots or an organics content greater than 6 percent, desiccated material and debris. In areas of tree or shrub removal, where planned structures or roadways are located, the full root ball and/or main mass of roots at the base of the plant should be removed.

Existing underground utilities, tanks or structures, if affected by construction activities, should be removed or relocated prior to site development. Based on the age of the two-story dwelling, we anticipate that a septic tank and leachfield may have treated wastewater onsite, however we are not aware of the location of the system or if it has been already properly removed.

Debris generated from the demolition of underground facilities, including abandoned pipes, should be removed from the site as construction proceeds. If pipes are abandoned in place, they should be capped or filled to mitigate the potential of water seepage, loss of soil into the pipe, or risk of pipe collapse. In general, this may be accomplished with filling pipes greater than 4-inches in diameter with a plug of lean cement that has a length at least 4 times the pipe diameter or to the extent of the property boundary, and smaller pipes may be sealed with an end cap.

3.2.2 Excavation Stability

The contractor is the sole party responsible for excavation stability and compliance with OSHA work site safety regulations. Trenches or narrow excavations greater than 4 feet deep may require shoring for worker safety. Deep excavations and temporary cut slopes should be benched at a gradient no steeper than 1:1 (H:V) or retained. As a preliminary value, we recommend that an undrained shear strength of 500 psf, a soil unit weight of 120 pcf and an active pressure coefficient of k_a=0.35 be used to determine the required shoring type.

Temporary shoring or underpinning may be required if excavations to construct new foundations or underground utilities extend below a 2:1 (H:V) plane projected downward from the bottom of existing foundations.

3.2.3 Placement of Fill

Following site preparation (as outlined in Section 3.2.1) the subgrade in fill areas should be scarified and moisture conditioned. Depending on the time of year, fill material should be blended and allowed to temper following addition of water and prior to compaction.

Fill should be benched and keyed if the existing ground surface is steeper than 4:1 (H:V). We recommend that the benching remove topsoil or other poor quality soil with sidewalls cut 4 feet into competent material. Keyways should have a minimum embedment of 2 feet into intact rock or approved native soil, have a minimum width of 10 feet and the bottom of the keyway should be graded at 2% into the slope to a 4-inch perforated drain pipe. We recommend that fill slopes have a maximum height of 10 feet and a gradient no steeper that 3:1 (H:V). We recommend taller or steeper fill slopes, if planned, are designed in as part of a remedial grading plan with additional geotechnical investigation.

Fill should be placed in lifts not exceeding approximately 8-inches in loose thickness and compacted using mechanical compaction equipment. Unless otherwise specified or approved, material to be used as structural fill and backfill should be non-expansive with the following properties:

- predominantly granular material should be well-graded with crushed or angular particles (typically with 75% having at least two fracture faces), and;
- particles should be less than 4 inches in any dimension, and;
- isolated cobbles up to 12 inches in diameter may remain in the fill, provided the oversized material is not nesting or stacked together to form voids or prevent compaction of the smaller soil particles, and;
- free of organic and inorganic debris, and;
- contain less than 30 percent of mostly non-plastic fines passing the No. 200 sieve, and;
- have a liquid limit less than 35 and plasticity index less than 12.

Non-expansive aggregate fill should be compacted to at least 95 percent of the maximum dry density and at or above the optimum moisture point as determined per ASTM D1557. Onsite soil should be evaluated by the geotechnical engineer prior to use as engineered fill and approved low or moderate plasticity soils should be compacted to between 88 to 92 percent of the maximum dry density and at least 4 points over the optimum moisture content. Test results of imported fill and backfill materials should be submitted to the geotechnical engineer prior to delivery to site (or stockpiling of onsite material for reuse), to confirm that they meet the above criteria.

3.3 Expansive Soils

Regional soil survey databases indicate that the plasticity index (PI) of the soil at the site ranges between 22 to 29 with a liquid limit (LL) of 45 to 53, which indicates that site soils are highly expansive. Plasticity Index testing is commonly used as a screening test, and there are general relationships between Plasticity Index and the swell potential of expansive soils. The California Building Code describes expansive soils as having a Plasticity Index greater than 15. The plasticity index is also generally related to categories swell potential in the following table:

Table 2: Soil expansivity prediction by liquid limits and plasticity index (Chen 1975)

Swelling Potential	Liquid Limit	Plasticity Index
Low	<30	0 to 15
Medium	30-40	10 to 35
High	40-60	20 to 55
Very High	>60	35 and above

As a screening test, plasticity index testing allows for relatively inexpensive laboratory testing to identify problem soil types or layers. However, the behavior of expansive soils is effected by many factors not captured by the plasticity index test such as the soil mineral type, confining pressure from overlying structures or fill loads, and the thickness of the soil layer. The shrink/swell behavior is also controlled by the variation of moisture level such as removal of moisture due to vegetation, saturation from irrigation or plumbing leaks, or climate changes. As expansive soils swell, they are capable of lifting some foundation types or causing undulations to pavement and the ground surface. If restrained by a fixed or rigid foundation, structures may be subject to uplift pressures to the underside of foundations, pile shafts or backside of retaining walls.

Measures to mitigate the risk of expansive soils typically include;

- Design of the structure with sufficient rigidity to distribute differential movement over a longer span or minimize curving of the slab (hogging or dishing) of the slab or foundations. This is often used in combination with design of the superstructure, plumbing and vertical elements to allow differential movement, such as with the use of control joints in slabs or hardscape, impervious flexible joints between floors and footings/walls, cladding with articulated joints or panels, and modular construction so walls, floors or portions of the building can move as a unit.
- Since shrink/swell behavior typically occurs as a result of seasonal moisture variation; certain construction and maintenance practices may be used to promote constant moisture in the foundation soils, such as surface drainage to eliminate ponded water, protecting excavations from drying, and construction of the foundation should be in the period following the wet season or use of soakage hoses to saturate the subgrade. Avoid curbs or depressed flower beds that allow for ponding of water near the structure, avoid or remove trees and heavy vegetation within 10 to 15 feet of the foundation or 1 to 1.5 times the tree height, and maintain gutters, spouts and drains to convey runoff away from the structure. Plumbing or utility trenches may contribute to soil moisture beneath the foundation. Use a plug of non-permeable material (such as controlled density fill or certain clays) at the point where trenches enter the building footprint to prevent infiltration of groundwater through the pipe bedding or backfill.
- Full or partial removal of the expansive material and replacement with non-expansive material or in-situ lime/cement mixing of limited depth. This typically requires excavation to below the active zone or to a non-expansive layer to create a more uniform condition for shallow foundations and slabs with different embedment depths and confining loads. A partial excavation may reduce (but not fully eliminate) the potential shrink/swell behavior.

Selection of one or more of the above measures to mitigate expansive soil will be a function of the final foundation type, cladding/superstructure type, architectural detailing, and planned depth of cuts and fills. This is discussed further in the following sections.

3.4 Seismic Design and Ground Motion Parameters

Based on the regional geology, subsurface conditions encountered mapped seismic ground motions determined using ASCE 7-16 procedures and 2019 California Building Code, we present site coefficients for seismic design on Table 3, below.

Table 3: Seismic Design Criteria

	Factor	Value
Site Class		D
Mapped Short-Period MCE_R, g	S_{s*}	1.5
Mapped MCE_R at 1 second, g	S_{1*}	0.6
Short Period Site Coefficient	F_a	1.0
Long Period Site Coefficient	F_v	null
Site Short-Period MCE_R, g	S_{MS}	1.5
Site MCE_R at 1 second, g	S_{M1}	null
Short Period Design Spectral Response Acceleration Parameter, g	S_{DS}	1.0
1 second Design Spectral Response Acceleration Parameter, g	S_{D1}	null
Peak Ground Acceleration	PGA	0.55

Seismic design provisions of current building codes generally prescribe minimum lateral forces, applied statically to the structure, combined with the gravity forces of dead-and-live loads. The prescribed lateral forces are generally considered to be substantially smaller than the actual peak forces that would be associated with a major earthquake. Consequently, structures should be able to: (1) resist minor earthquakes without damage, (2) resist moderate earthquakes without structural damage but with some nonstructural damage, and (3) resist major earthquakes without collapse but with some structural as well as nonstructural damage. Conformance to the current building code recommendations does not constitute any kind of guarantee that significant structural damage would not occur in the event of a maximum magnitude earthquake; however, it is reasonable to expect that a well-designed and well-constructed structure will not collapse or cause loss of life in a major earthquake (SEAOC, 1996).

3.5 Shallow Foundations

Based minimum code requirements proposed building additions and new foundations for the relocated dwelling may be supported on a shallow foundation that is properly detailed to account for both differential settlement and the amount of lateral spread that will occur in a future strong earthquake. We recommend that the shallow foundation consists of a system of grade beams with isolated spread footings (used to carry point loads) tied into the grade beams. The grade beams and foundation ties should be detailed to carry a tension load equal to or greater than the point load times 0.10_{DS} or 0.25 times the dead load of ½ of the building area, whichever is greatest.

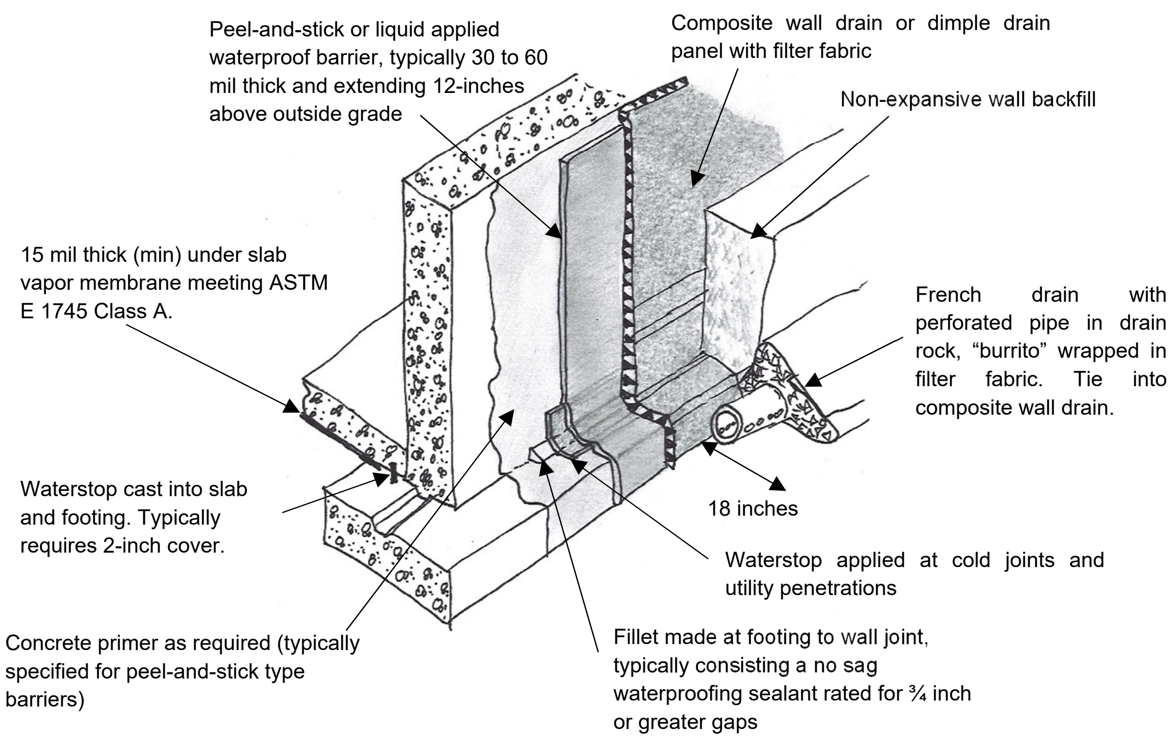
The grade beams should be designed to have a minimum embedment depth of 30 inches, and designed with an allowable bearing pressure of 2,500 pounds per square foot (psf) for dead plus sustained live load, which may be increased by 30 percent to include seismic or wind loads. Lateral loads may be resisted by passive pressure of 250 pcf on the embedded portion of the footing (neglecting the capacity from the upper 12-inches). Lateral loads from unbalanced fill should be applied as an “at-rest” pressure using equivalent fluid pressure of 50 pcf.

We estimate that the differential settlement of new foundations will generally be less than 1 to 2 inches over a span of 20 to 30 feet. It is also our experience that some amount of differential movement occurs between existing structures and areas of structural alterations. This is often as a result of slight differences in rigidity and structural design or seasoning of fresh timber. We recommend that a control joint is detailed within the exterior cladding, internal walls, ceiling and the floors joining the existing foundation with new foundation areas, to accommodate differential settlement between the two.

3.6 Dampproofing and Waterproofing

Slab-on-grade floors and basement walls or partially below grade walls that are part of the dwelling (or other areas treated as conditioned space with moisture sensitive finishes) shall be coated with an approved waterproofing material on the exterior or pressure side. Slab-on-grade floor slabs should be underlain by a vapor barrier meeting ASTM 1745 Class A specifications. Ensure that this barrier is strong enough to resist puncture during slab construction. Joints and penetrations should be sealed with a waterproof material. There are several types of waterproofing products available; the application of each system varies depending on the specific system. We recommend referring to the manufacture's specifications for detailed design requirements and determining other products that may be required (such as primers, adhesives, sealants, etc.), and installation should be carried out by a contractor familiar with the selected system. We provide the following sketch for general concepts and best-practices for basement waterproofing systems.

Figure 12: Typical Basement Waterproofing Concepts



3.1 Crawl Space Treatment

Typical crawl space construction involves ventilation of the underfloor space through openings in the foundation walls or exterior walls, and a bare earth surface inside of the perimeter footing or grade beam. It is our experience, for the majority of cases, the soil under the central portions of the building will achieve very dry condition over time as moisture vapor is pulled out through the ventilation or diffuses through the floor system (due to the pressure between areas of high humidity and low humidity). It is also our experience that some amount of water may infiltrate from outside the building perimeter, and cause the soil along the interior margin to be moist or stand in ponded water, particularly if there is excessive irrigation or following long duration and heavy storms. Refer to Section 3.3 for measures to reduce the amount of water that may infiltrate into the building area.

The differential of soil moisture from the building perimeter to the center can lead to differential shrink/swell movement or expansive forces being applied to the foundation system. In addition, moisture vapor, dust and allergens can be pulled from crawl space and spread into the conditioned space within the house, affecting air quality and the efficiency of the HVAC system.

One method to provide a cleaner crawlspace and reduce moisture vapor transmission into the living area is to seal the bare earth with a plastic membrane, or install a thin concrete “rat slab”. A crawl space with a sealed earth surface may allow for a reduction in the area of ventilation openings. In some designs this may extend to complete encapsulation of the crawlspace area (where the exterior foundation walls are insulated and have no ventilation openings to the outside) and mechanical ventilation provides conditioned air to the crawl space. This may allow for a higher level of energy efficiency for the home. We recommend consulting with your architect or mechanical designer for details related to the building ventilation and HVAC design, which may have more stringent requirements. In order to provide improved foundation performance, we recommend the crawlspace construction considers the following:

Earth Seal:

- Exposed earth should be fully covered with a 15-mil thick (min) vapor membrane meeting ASTM E 1745 Class A. Joints of the vapor retarder shall overlap by 6 inches (152 mm) and shall be sealed or taped. The edges of the vapor retarder shall extend not less than 6 inches (152 mm) up the stem wall and shall be attached and sealed to the stem wall or insulation. All penetrations passing through the vapor barrier should be taped or sealed.
- It is our experience that plastic vapor barrier may become damaged with time (either punctured from people accessing the area, the taped seals may separate as they age, and the plastic can be chewed/damaged by pests). In addition, a plastic barrier will follow the undulations in the earth surface, which can lead to areas of ponded water at low points or where there are folds/creases in the plastic. To provide a more durable seal, the vapor barrier may be covered with a thin concrete “rat slab”
- Rat slabs are typically 2.5-inch to 4-inch thick unreinforced concrete, and are not typically finished to a high level. We recommend that the rat slab is at least raked and bull floated. We recommend using a 2,500 psi strength concrete with fibremesh, batched in accordance with the manufacturer’s specifications (typically a minimum of 1 to 3 pounds per cubic yard depending on fiber type). We recommend that the concrete slump is 4-inch or less and placed on a grade sloped at 10:1 (H:V) or flatter, unless the contractor is able to place the concrete with a greater slope or slump.
- Utility pipes that pass through the foundation wall should be sleeved, and the annular spaced sealed with closed-cell foam (so the pipe can be removed/replaced through the sleeve if repairs are required.

Ventilation:

- Mix crawlspace and living space air only if the crawlspace is accessible and able to be frequently visited/inspected (for example, if it is used for storage or adjoins a basement). If it is inaccessible and unvisited, it may become wet without detection, and air entering the living space may contain fungal odors or even harmful microorganisms generated in the crawlspace.
- Do not locate vent fan exhausts, dryer exhausts or condensate drains in the crawlspace.
- Integrate mechanical ventilation as required to meet Code and to maintain a healthy relative humidity level in the crawlspace. This will typically involve air conditioning using a dehumidifier, ventilation using a small amount of air supply from HVAC system, or installing small exhaust fans.

Drainage:

- The finished ground level of the under-floor space shall uniformly be sloped to drain to at least one side where the interior level is equal to or higher than the outside finished ground. Where this is not practical, an interior foundation drain system should be installed to pipe water by gravity flow to a low point outside the building pad, or a sump and pump system should be installed to remove any water that accumulates inside the crawl space.
- Special design measures, beyond the recommendations in this report, may be required for buildings located in flood hazard areas.
- The under-floor grade shall be cleaned of all vegetation and organic material. All wood forms used for placing concrete and any construction materials shall be removed before a building is occupied or used for any purpose.

3.2 Secondary Slabs, Pavement and Exterior Flatwork (on moderate to high EI)

We recommend that slabs and other exterior hardscape areas, including concrete patios are supported directly on a layer of compacted non-expansive fill at least 12 inches thick and structurally independent from the perimeter foundations and “free-floating”. Alternatively, the slab may be structurally suspended by piers or specially design grade beam with a void under the slab portion. Slab-on-grade should be expected to crack. Control joints should be at a maximum spacing of 10 feet in both directions. The slabs should be designed as a rigid slab capable of resisting shrink/swell movement of expansive soil without significant deformation or cracking. Secondary slabs-on-grade should be designed specifically for their intended use and loading requirements. As a minimum requirement and in reference to the WRI design manual for slab on grade foundations with expansive soils, we recommend that the slabs are designed to support a cantilever span of 9 feet and reinforced with a minimum Asfy of 5,200 lbs.

3.3 Surface Drainage and Storm Water Management

Ponding of storm water should not be permitted near or under the building or footings during prolonged periods of inclement weather.

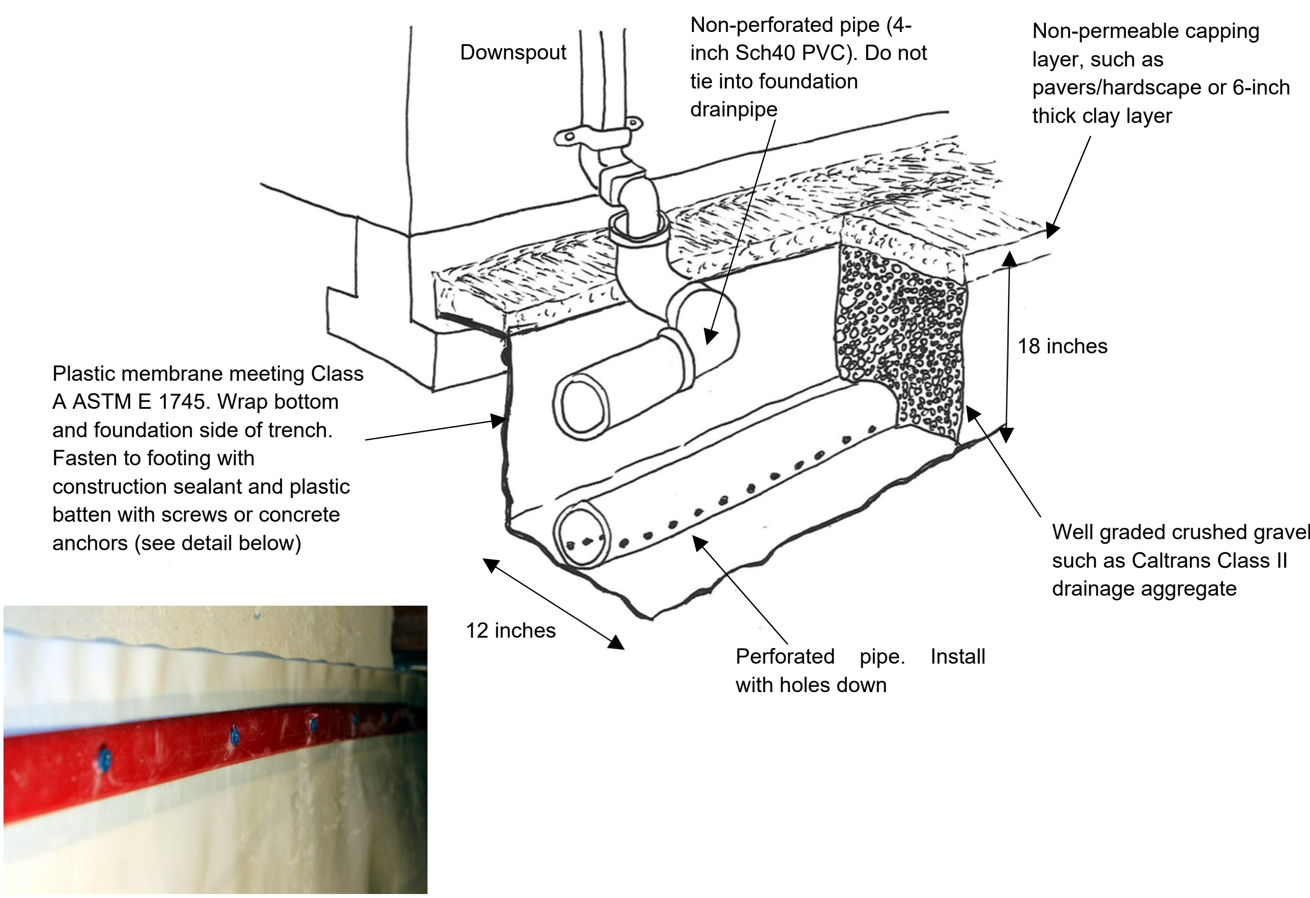
We recommend that the building pad is positively graded at all times to provide for rapid removal of surface water runoff from around the foundation elements and to prevent ponding of water or seepage toward the foundation systems at any time during or after construction. As a minimum requirement, finished grades in landscaped areas should have downslopes of at least 5 percent within 10 feet from the exterior walls to allow surface water to drain positively away from the structures. For hardscape areas, the slope gradient can be reduced to 2 percent. Storm water from roof downspouts should be directed to a solid pipe. We recommend that drains are sloped at a minimum of 5 percent towards an engineered drain system. The native soil has a high silt and clay content, so infiltration devices and storm water management practices that rely on percolation into the native soil will not be effective.

In addition to the above, we recommend the following measures to minimize soil moisture problems with the foundation:

- Maintenance of drains and downspouts to prevent water from saturating the soil around the building.
- Eliminating or reducing irrigation of garden areas around the building.
- As practical, do not install landscaping where the root zone of nearby plants results in wicking/drying of the soils under the building. This is typically the case where a tree's dripline is adjacent or over the building footprint, or where the roots extend under the foundation.
- Construction of a foundation subdrain along the outside building perimeter to stabilize the soil moisture variation under the building. The exact location of these drains may vary, but should be placed relative to the pattern of surface water flow and upstream of the dwelling.

We recommend that the foundation drain pipes are separate from the pipes that carry water from downspouts and surface drain inlets. The downspouts from roof gutters should be connected to the drain system in a way that prevents overtopping or spilling of the captured runoff or backflow into the subdrain pipes. The figure below shows typical drain details.

Figure 13: Typical Foundation Subdrain Detail



Subdrains should be provided with adequately spaced cleanouts so that their effectiveness can be monitored in the future. Use 4 inch diameter or larger, rigid-walled, PVC drainage pipe (Class SDR35 or stronger) with glued joints. Exposed PVC drain inlets and downspout adapters may be protected with an opaque covering or painted with acrylic-based latex paint to increase UV protection.

Excavation of perimeter drains parallel to existing footings should remain outside the foundation influence zone (above a 1:1 plane extending down from the bottom of the footing), make use of 'hit-and-miss' excavation staging, or use underpinning to prevent undermining the existing footing.

4 Corrosion

The American Concrete Institute guideline ACI318 outlines exposure categories regarding the attack on concrete (due to water-soluble sulfate) or reinforcing steel (due to water-soluble chloride ion). The durability design depends on the exposure class, typically specifying a maximum water-cement ratio, a minimum compressive strength or use a specific cement type. We did not perform site-specific soil corrosivity tests, and recommend consulting with a corrosion engineer or your structural engineer regarding measures that may be appropriate for the protection of buried steel or concrete in contact with soil and bedrock.

5 Construction Considerations

The geotechnical engineer should review project plans and specifications prior to construction to check that the geotechnical aspects of the project are consistent with the intent of the recommendations presented herein. This is to confirm that geotechnical conditions have been interpreted with the intent of the recommendations of this report. In addition, the local Building Official may require we issue a letter documenting our review of the final plans, to be included with the permit submittal. We recommend that the designers discuss their designs with us as their work progresses to avoid surprises at submittal time, which could delay the job and commencement of construction.

Although the information in this report is primarily intended for the design engineers, it may also be useful to the contractors. However, it is the responsibility of the bidders and contractors to evaluate soil and groundwater conditions independently and to develop their own conclusions and designs regarding excavation, grading, foundation construction, and other construction or safety aspects.

We recommend that the following items are visually inspected, tested or documented during the construction (by appropriately qualified personnel which are engaged by the owner and independent of the contractor).

- Observe site preparation in areas to receive fill, assess ground conditions, and consultation regarding the need for over-excavation to remove unsuitable material.
- Evaluate and approve material to be used as engineered fill.
- Observation of fill placement and record measurements of relative compaction and moisture content of engineered fills.
- Observation and documentation of the excavation/drilling depth of footings or other foundation elements.

6 Closing

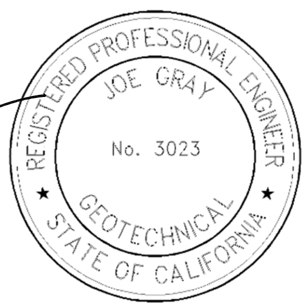
This report has been prepared in accordance with generally accepted professional geotechnical engineering practice for the exclusive use of American Housing Inc. in relation to the specified project brief described in this report. In the event that any changes are made in the character, design, or layout of the proposed project, the conclusions and recommendations contained in this report should be reviewed by Gray Geotech Inc. to determine whether modifications to the report are necessary.

No other warranty, express or implied, is made. No liability is accepted for the use of any part of the report for any other purpose or by any other person or entity. The analyses and recommendations submitted in this report are based on the ground conditions indicated from published sources and investigations described in this report based on accepted normal methods of site investigations. Only a limited amount of information has been collected to meet the specific financial and technical requirements of the Client's brief in accordance with our work agreement dated September 30, 2020, and this report does not purport to completely describe all the site characteristics and properties. The nature and extent of variations within the project site may not become evident until construction. In the event variations occur, it will be necessary to reevaluate the recommendations of this report.

Subsurface conditions relevant to construction works should be assessed by contractors who can make their own interpretation of the factual data provided. They should perform any additional tests as necessary for their own purposes.

We hope this provides the information that you require at this time. If you have any questions, please contact us.

Sincerely,



Joe Gray, GE

GRAY
GEOTECH

APPENDIX 1

CPT logs

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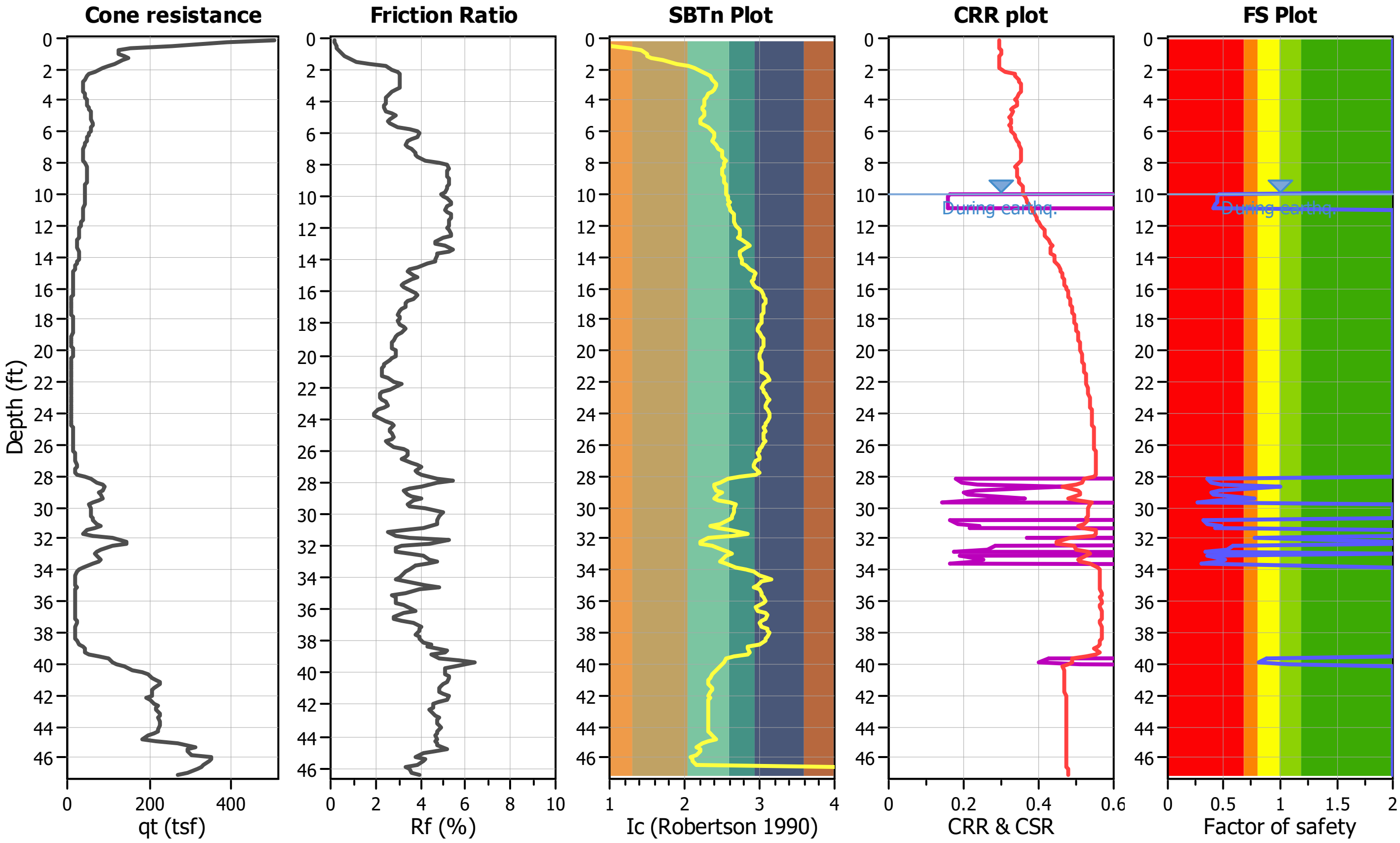
LIQUEFACTION ANALYSIS REPORT

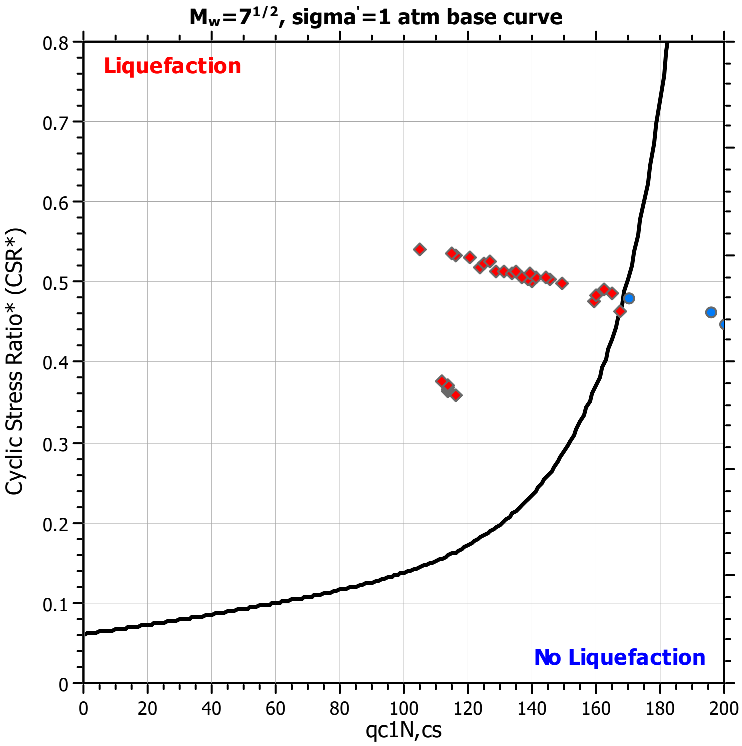
Project title : **Location : 4110 Alhambra Way, Martinez, CA**

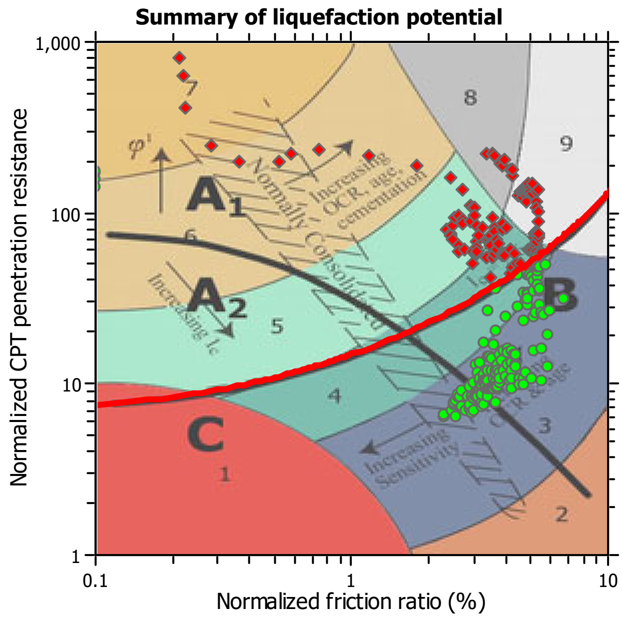
CPT file : CPT1

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	16.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.70	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method
Peak ground acceleration:	0.67	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

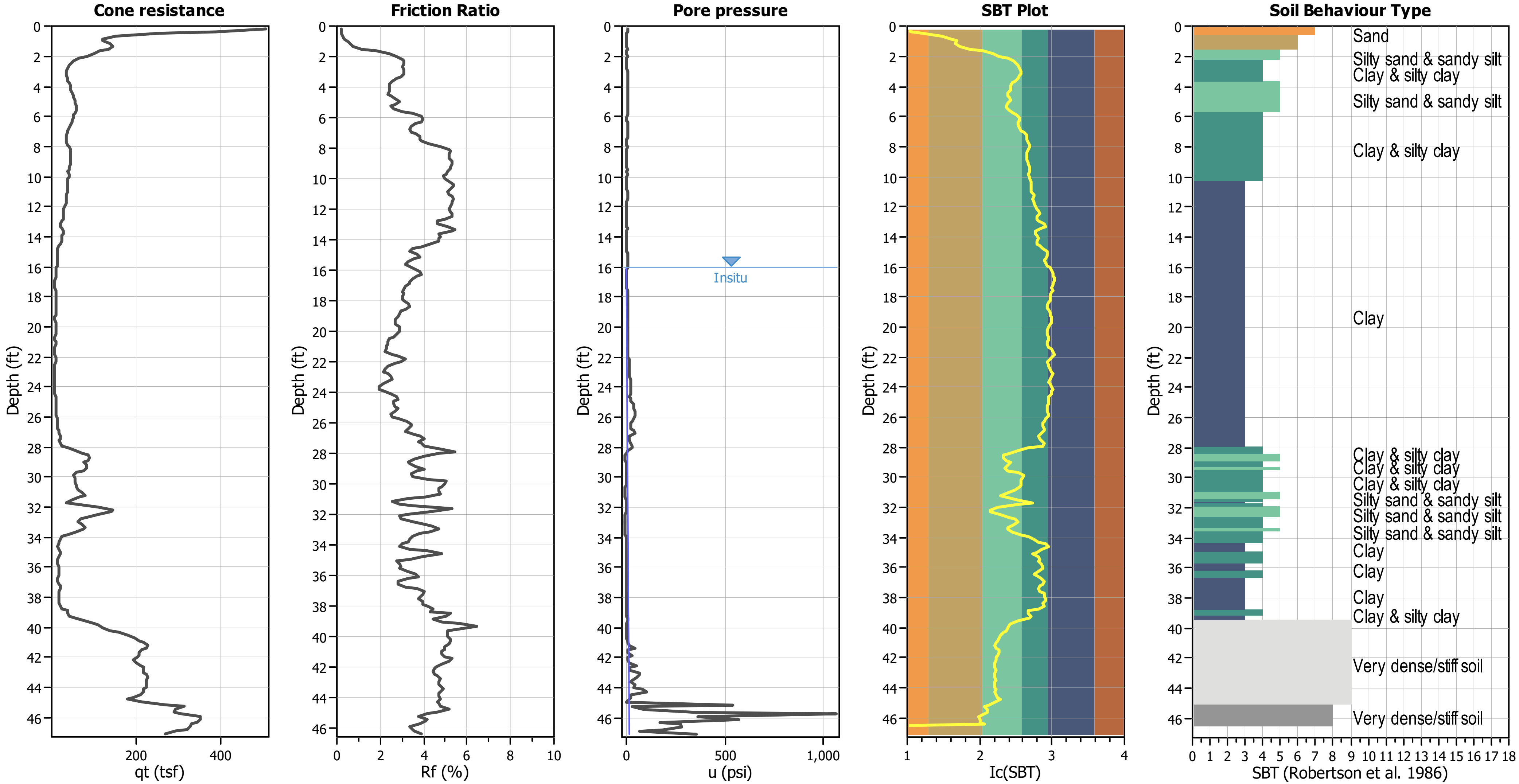






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots



Input parameters and analysis data

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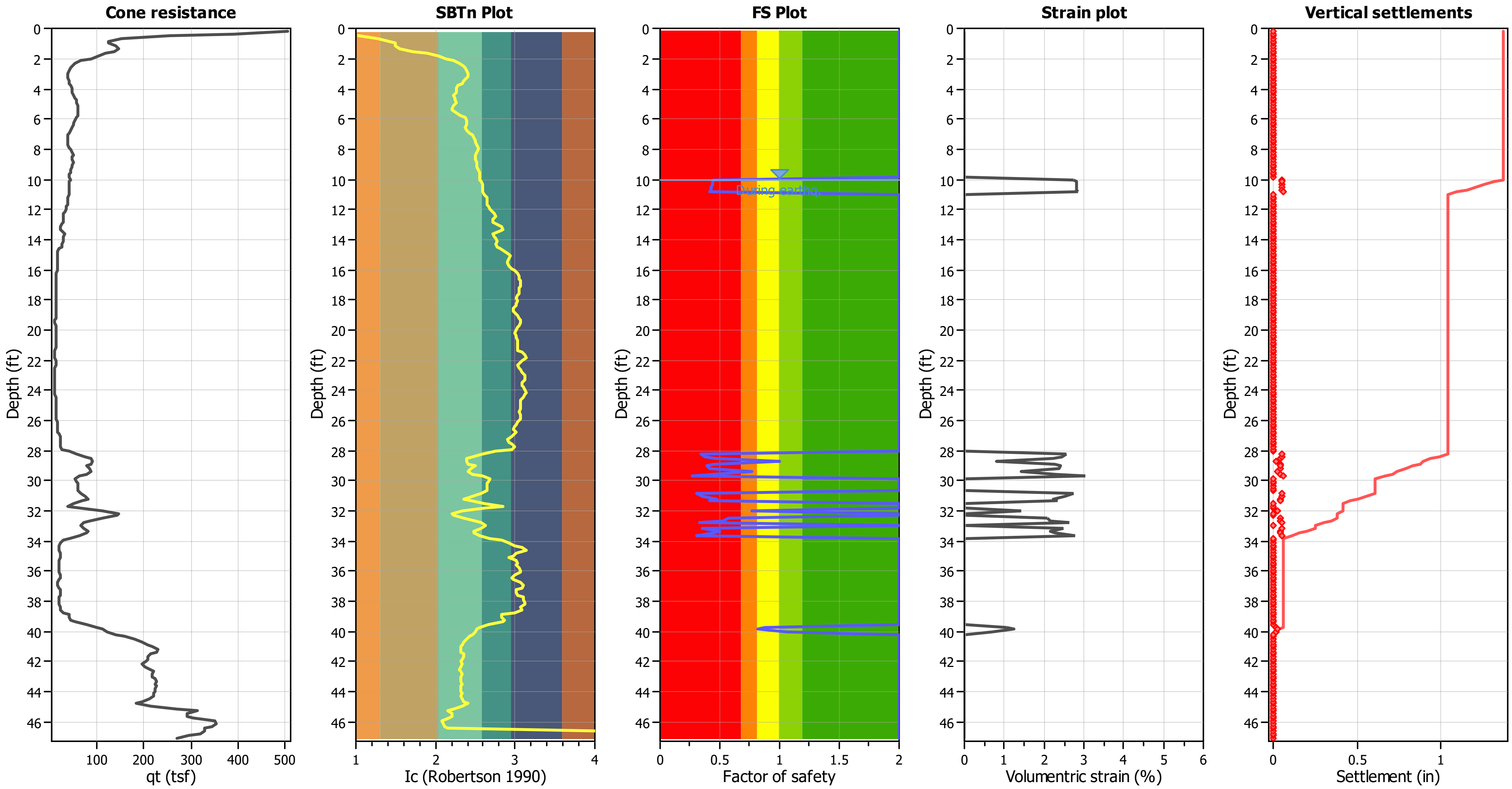
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.70	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.67	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	16.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

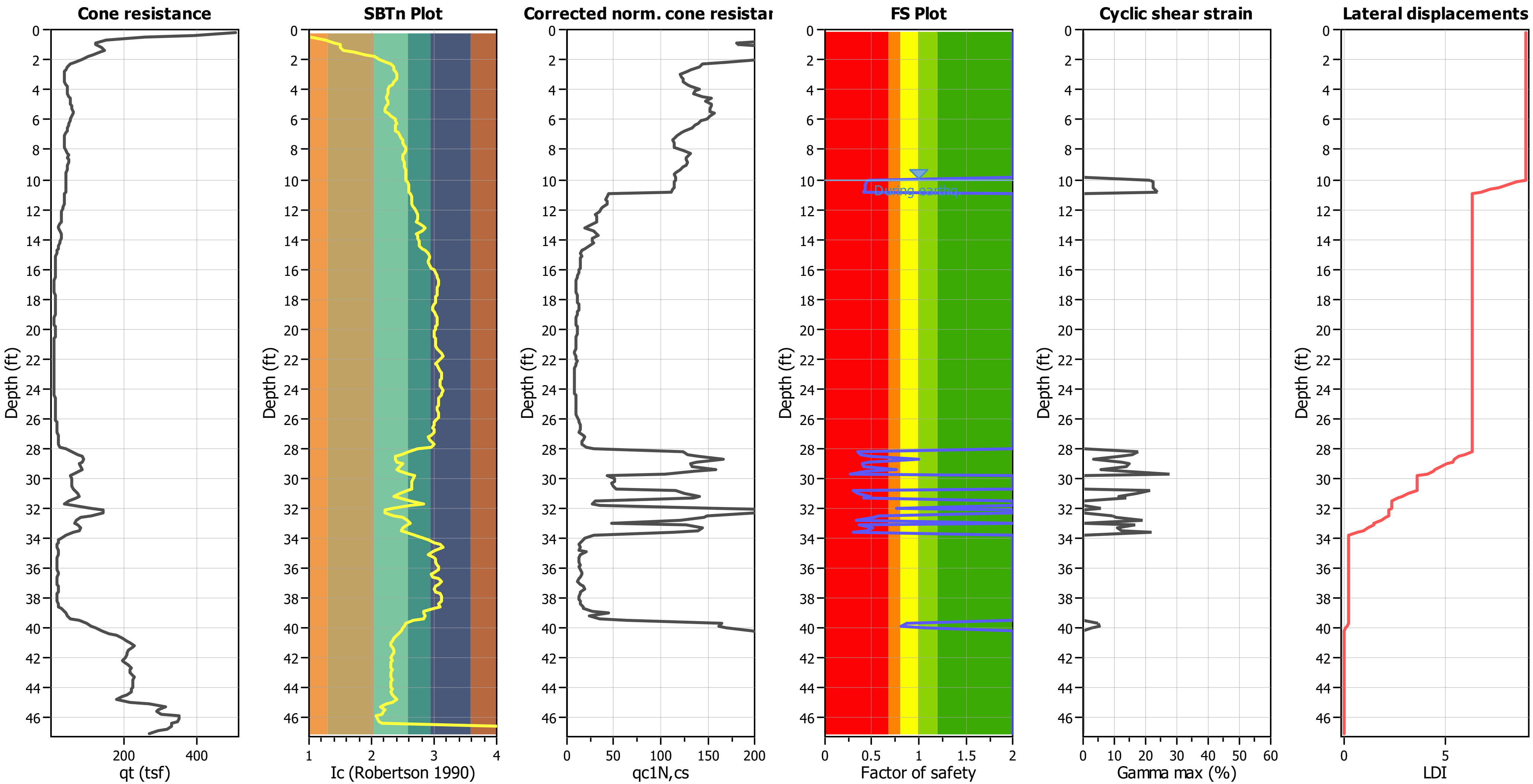
Estimation of post-earthquake settlements



Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
FS: Calculated Factor of Safety against liquefaction
Volumentric strain: Post-liquefaction volumentric strain

Estimation of post-earthquake lateral Displacements



Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$q_{c1N,cs}$: Equivalent clean sand normalized CPT total cone resistance

F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

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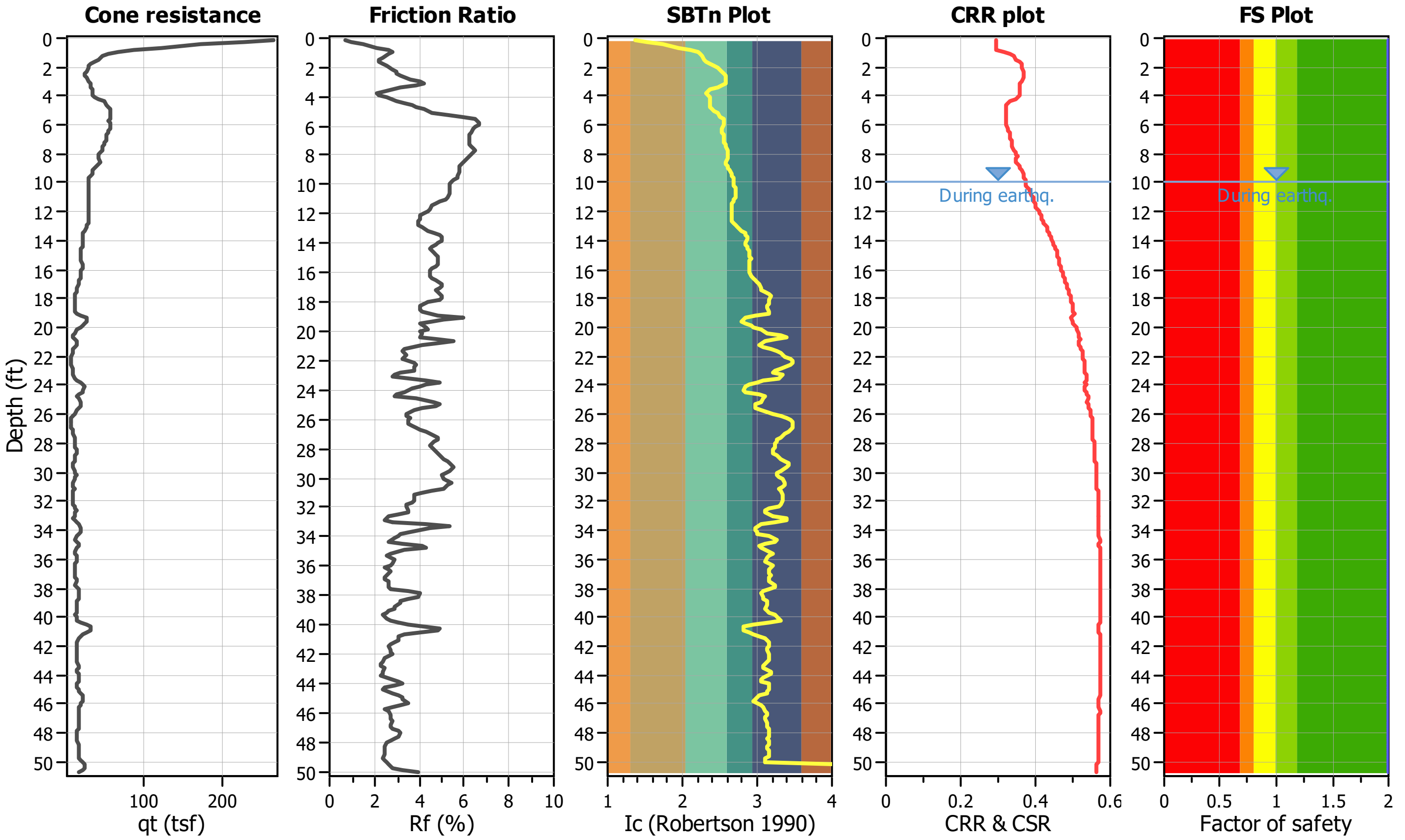
LIQUEFACTION ANALYSIS REPORT

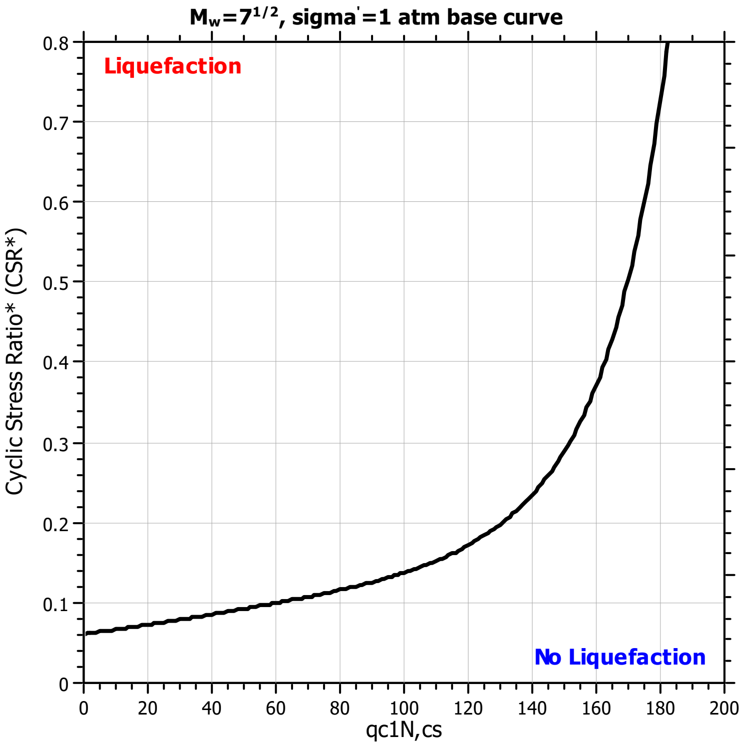
Project title : **Location : 4110 Alhambra Way, Martinez, CA**

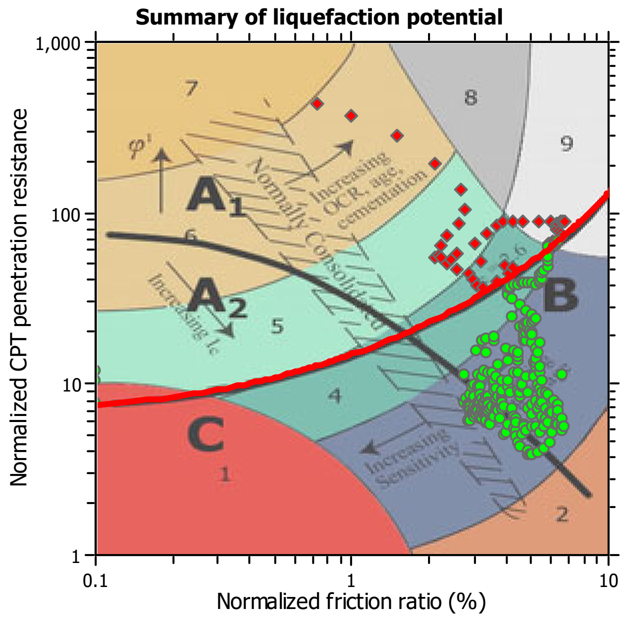
CPT file : CPT2

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	16.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.70	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method
Peak ground acceleration:	0.67	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

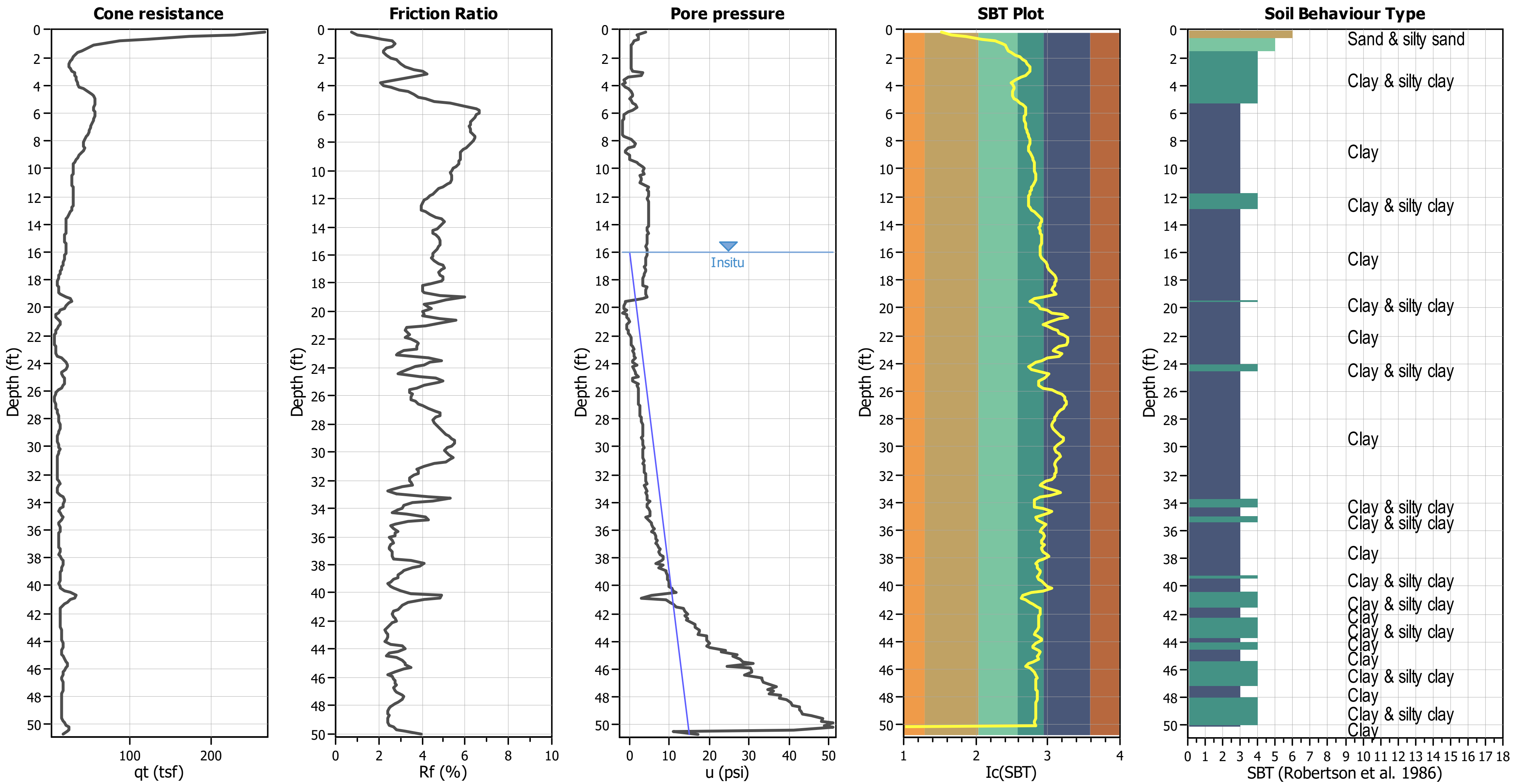






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots



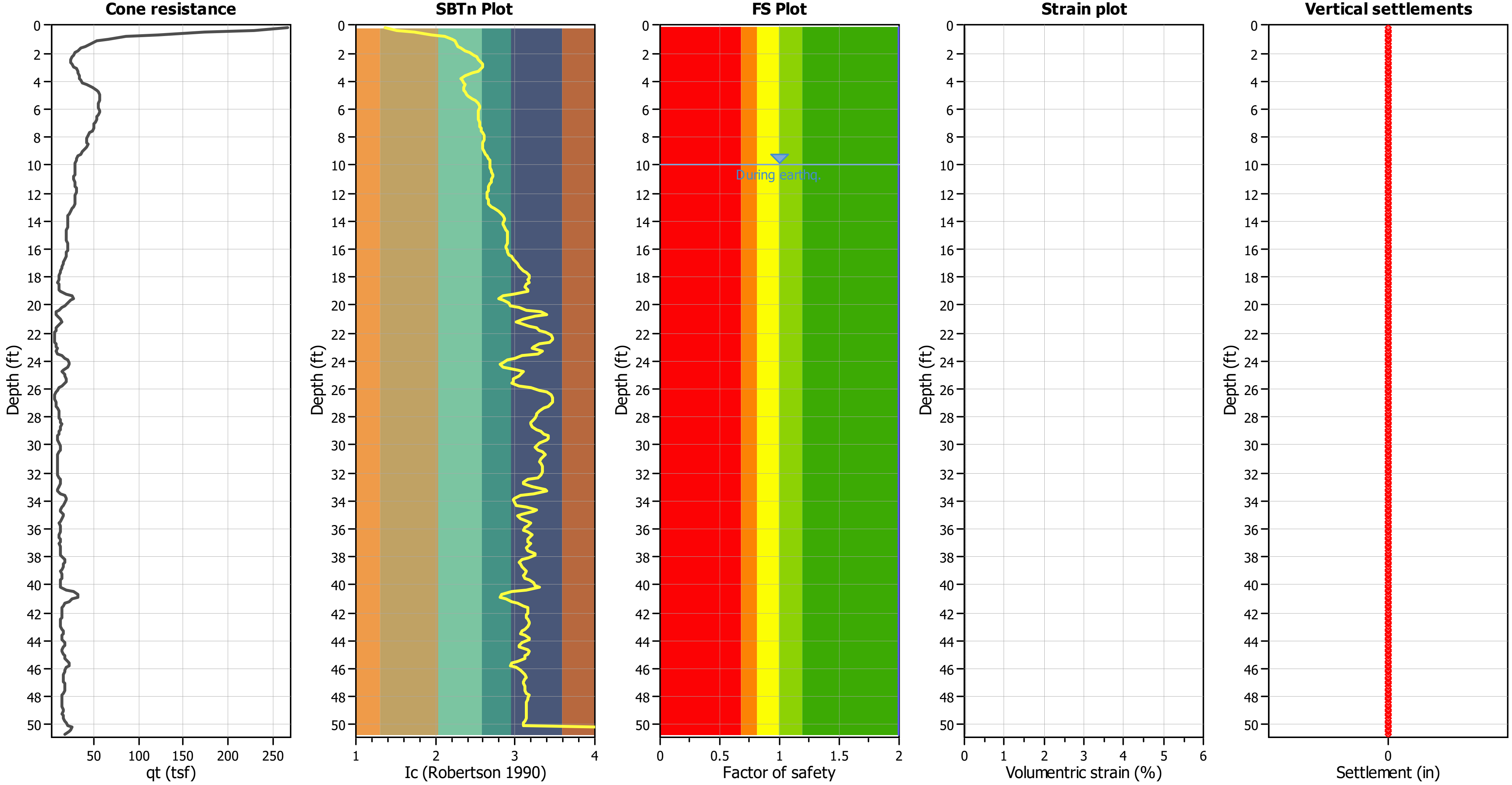
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.70	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.67	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	16.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

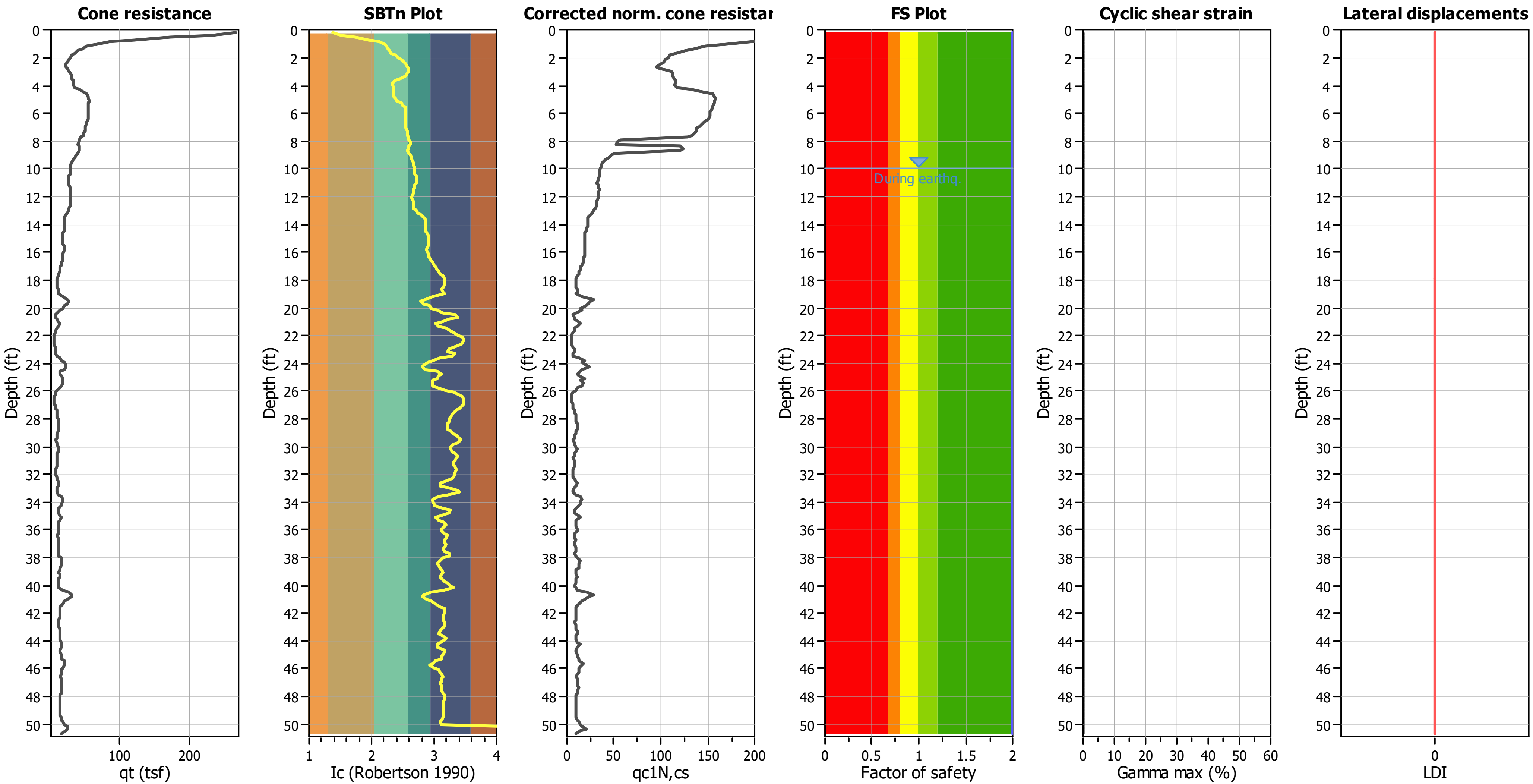
Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake lateral Displacements



Abbreviations

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$q_{c1N,cs}$: Equivalent clean sand normalized CPT total cone resistance

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