CORNERSTONE EARTH GROUP

Type of Services	**Geotechnical Investigation**
Project Name	**Broadway Terrace Residence**
Location	**10040 Broadway Terrace Oakland, California**
Client	[redacted]
Client Address	**600 Grand Avenue, Suite 302 Oakland, California**
Project Number	**1300-1-1**
Date	**August 11, 2021**

Prepared by **Stephen C. Ohlsen, P.E.**
Project Engineer
Geotechnical Project Manager

John R. Dye, P.E., G.E.
Principal Engineer
Quality Assurance Reviewer

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SECTION 1: INTRODUCTION

This geotechnical report was prepared for the sole use of Mr. Dong Zhang for the Broadway Terrace Residence in Oakland, California. The location of the site is shown on the Vicinity Map, Figure 1. For our use, we were provided with the following documents:

- Architectural plans titled Project2, sheets A2 and A5 prepared by HPA, undated.

1.1 PROJECT DESCRIPTION

Based on our review of Sheets A2 and A5, the project will consist of a new single-family residence. The residence will be three levels, with the garage on the ground floor (street level), the kitchen and storage on the second and bedrooms and bathroom on the third floor. The second level extends into the hillside beyond the footprint of the garage. The ground-, second-, and third-floors will be at Elevations 102, 112, and 122 feet, respectively, based on an assumed datum of 100 feet.

Grading is anticipated to include cuts and fills of up to 10 feet. We assume the garage and second level walls will be designed and built as retaining walls to retain the cut into the existing hillside and any adjacent fills. The backyard will also be terraced and include retaining walls and stairways, which will be accessed from the third floor. Appurtenant utilities, landscaping, driveways, and other improvements necessary for lot development is also planned.

Structural loads are not available at this time, however, loads for the structure are anticipated to be typical of similar type structures.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated June 21, 2021 and consisted of field and laboratory programs to evaluate physical and engineering properties of the subsurface soils, engineering analysis to prepare recommendations for site work and grading, building foundations, flatwork, retaining walls, and pavements, and preparation of this report. Brief descriptions of our exploration and laboratory programs are presented below.

1.3 EXPLORATION PROGRAM

Field exploration consisted of two borings drilled on July 16, 2021 with limited-access, Minuteman drilling equipment. The borings were drilled to depths ranging from 6½ to 8½ feet. The borings were backfilled with cement grout in accordance with local requirements; exploration permits were obtained as required by local jurisdictions.

The approximate locations of our exploratory borings are shown on the Site Plan, Figure 2. Details regarding our field program are included in Appendix A.

1.4 LABORATORY TESTING PROGRAM

In addition to visual classification of samples, the laboratory program focused on obtaining data for foundation design and seismic ground deformation estimates. Testing included moisture contents, dry densities, and Plasticity Index tests. Details regarding our laboratory program are included in Appendix B.

1.5 ENVIRONMENTAL SERVICES

Environmental services were not requested for this project. If environmental concerns are determined to be present during future evaluation, the project environmental consultant should review our geotechnical recommendations for compatibility with the environmental concerns.

SECTION 2: REGIONAL SETTING

2.1 GEOLOGICAL SETTING

The site is located within the East Bay Hills just southwest of the Mt. Diablo range. The San Andreas Fault system, including the Monte Vista-Shannon Fault, exists within the Santa Cruz Mountains and the Hayward and Calaveras Fault systems exist within the Diablo Range. Based on our review of a local geologic map (Graymer, 2000), the site is mapped as being underlain by an unnamed glauconitic mudstone unit (Tsm). A regional geologic map is presented in Figure 3.

2.2 REGIONAL SEISMICITY

While seismologists cannot predict earthquake events, geologists from the U.S. Geological Survey have recently updated (in 2015) earlier estimates from their 2014 Uniform California Earthquake Rupture Forecast (Version 3; UCERF3) publication. The estimated probability of one or more magnitude 6.7 earthquakes (the size of the destructive 1994 Northridge earthquake) expected to occur somewhere in the San Francisco Bay Area has been revised (increased) to 72 percent for the period 2014 to 2043 (Aagaard et al., 2016). The faults in the region with the highest estimated probability of generating damaging earthquakes between

2014 and 2043 are the Hayward (33%), Calaveras (26%), and San Andreas Faults (22%). In this 30-year period, the probability of an earthquake of magnitude 6.7 or larger occurring is 22 percent along the San Andreas Fault and 33 percent for the Hayward Fault.

The faults considered capable of generating significant earthquakes are generally associated with the well-defined areas of crustal movement, which trend northwesterly. The table below presents the State-considered active faults within 25 kilometers of the site.

Table 1: Approximate Fault Distances

Fault Name	Distance	
	(miles)	(kilometers)
Hayward (Total Length)	0.4	0.6
Calaveras	9.4	15.1
Hayward (Southeast Extension)	9.6	15.5

A regional fault map is presented as Figure 4, illustrating the relative distances of the site to significant fault zones.

SECTION 3: SITE CONDITIONS

3.1 SURFACE DESCRIPTION

The undeveloped hillside property is located in a residential area of the East Bay Hills. The site is bounded by Broadway Terrace to the west and developed residential properties on all other sides. The site is currently covered with various shrubs and multiple young to mature trees. Based on the proposed plans, the uphill side of the garage (street level) and second level will be cut into the hillside and daylight along the downhill side.

Detailed topographic information for the site was not made available at the time of this report. However, based on our visual observations and field measurements, the slope appears to range from approximately 2:1 to 3:1 (horizontal:vertical). We also observed a storm drain manhole on the upslope southeast corner of the property that appears to have a concrete apron and approximately 12-inch-diameter pipe that discharges onto the slope. The location of the manhole is noted on our site plan, Figure 2. We observed shallow erosion and associated rills and gullies below the storm drain outfall and undermining of the existing concrete apron. The erosion channel appears to extend downslope across the site to Broadway Terrace.

3.2 SUBSURFACE CONDITIONS

Our explorations generally encountered residual soils overlying sedimentary bedrock. Boring EB-1 encountered approximately 3½ feet of hard sandy lean clay overlying low to moderately hard sandy mudstone to the terminal depth of approximately 6⅓ feet. Boring EB-2 encountered very stiff to hard sandy lean clay to a depth of approximately 6½ feet. The lean clay was underlain by sandy mudstone to the maximum depth explored of approximately 8½ feet.

3.2.1 Plasticity/Expansion Potential

We performed one Plasticity Index (PI) test on a representative sample. Test results were used to evaluate the expansion potential of surficial soils. The results of the surficial PI test indicated a PI of 14, indicating low plasticity and expansion potential to wetting and drying cycles.

3.2.2 In-Situ Moisture Contents

Laboratory testing indicated that the in-situ moisture contents within the upper 6 to 8 feet range from 2 to 6 percent over the estimated laboratory optimum moisture.

3.3 GROUNDWATER

Groundwater was not encountered in either of our borings during drilling; however, the borings were not left open but were immediately backfilled when the boring was completed. We noted no evidence of springing activity on site and groundwater was not encountered in our borings. The California Geological Survey (CGS) indicates the neighborhood is not in an area known to have a laterally continuous groundwater table. Based on our previous experience in the area and depth to groundwater maps by CGS, (Oakland East 7.5-minute quadrangle, 2003), we anticipate groundwater to be greater than 40 feet below current grades; however, as with all hillside environments, there is a potential for temporary perched water conditions in winter months. It is not uncommon for water to be perched at the top of bedrock.

Fluctuations in groundwater levels occur due to many factors including seasonal fluctuation, underground drainage patterns, regional fluctuations, and other factors.

SECTION 4: GEOLOGIC HAZARDS

4.1 FAULT SURFACE RUPTURE

As discussed above several significant faults are located within 25 kilometers of the site. The site is not located within a State-designated Alquist-Priolo Earthquake Fault Zone. As shown in Figure 3, no known surface expression of fault traces is thought to cross the site; therefore, fault surface rupture hazard is not a significant geologic hazard at the site.

4.2 ESTIMATED GROUND SHAKING

Moderate to severe (design-level) earthquakes can cause strong ground shaking, which is the case for most sites within the Bay Area. A peak ground acceleration (PGA) was estimated for analysis using a value equal to F_{PGA}*PGA, as allowed in the 2019 edition of the California Building Code. For our analysis we used a PGA_M of 1.185g.

4.3 LIQUEFACTION POTENTIAL

The site is not located within a State-designated Liquefaction Hazard Zone (CGS, Oakland East Quadrangle, 2003). However, we screened the site for liquefaction during our site exploration by retrieving samples from the site, performing visual classification on sampled materials, and performing various tests to further classify the soil properties.

During strong seismic shaking, cyclically induced stresses can cause increased pore pressures within the soil matrix that can result in liquefaction triggering, soil softening due to shear stress loss, potentially significant ground deformation due to settlement within sandy liquefiable layers as pore pressures dissipate, and/or flow failures in sloping ground or where open faces are present (lateral spreading) (NCEER 1998). Limited field and laboratory data is available regarding ground deformation due to settlement; however, in clean sand layers settlement on the order of 2 to 4 percent of the liquefied layer thickness can occur. Soils most susceptible to liquefaction are loose, non-cohesive soils that are saturated and are bedded with poor drainage, such as sand and silt layers bedded with a cohesive cap.

As discussed in the "Subsurface" section above, we primarily encountered stiff cohesive soils underlain by bedrock. In addition, the design ground water level is anticipated to be below any granular soils. Based on the above, our screening of the site for liquefaction indicates a low potential for liquefaction.

4.4 LATERAL SPREADING

Lateral spreading is horizontal/lateral ground movement of relatively flat-lying soil deposits towards a free face such as an excavation, channel, or open body of water; typically, lateral spreading is associated with liquefaction of one or more subsurface layers near the bottom of the exposed slope. As failure tends to propagate as block failures, it is difficult to analyze and estimate where the first tension crack will form.

Due to the low potential for liquefaction, there are no open faces within a distance considered susceptible to lateral spreading; therefore, in our opinion, the potential for lateral spreading to affect the site is low.

4.5 SEISMIC SETTLEMENT/UNSATURATED SAND SHAKING

Loose unsaturated sandy soils can settle during strong seismic shaking. As the soils encountered at the site were predominantly stiff to very stiff clays and medium dense to dense sands, in our opinion, the potential for significant differential seismic settlement affecting the proposed improvements is low.

4.6 LANDSLIDING

Available published maps do not show active landslides at or immediately adjacent to the site, nor is the site located within a State-designated Landslide Hazard Zone (CGS, Oakland East Quadrangle, 2003). During our field investigation, we did not observe any landslides on, or adjacent to the property, however, we recommend that proposed foundations extend into bedrock to reduce the potential of movement from the surficial soils. We recommend that any graded slopes be covered by erosion control fabric or vegetation to reduce the potential for erosion and surficial soil movement.

SECTION 5: CONCLUSIONS

5.1 SUMMARY

From a geotechnical viewpoint, the project is feasible provided the concerns listed below are addressed in the project design. Descriptions of each concern with brief outlines of our recommendations follow the listed concerns.

- Potential for differential foundation settlement
- Excavation difficulties due to shallow bedrock
- Surficial erosion
- Potential for perched groundwater condition

5.1.1 Potential for Differential Foundation Settlement

As discussed above, our explorations encountered between 3½ and 6½ feet of clayey surficial soils overlying sandy mudstone bedrock. Based on our review of the provided architectural schematic plans, we anticipate that soil/bedrock transitions may occur below the garage and second floor levels. Due to the potentially varying subgrade support conditions, i.e. soil versus bedrock, foundations may experience differential settlement under static and seismic loadings. Differing subsurface conditions underneath the buildings could increase the potential differential movement under the buildings at transitions. Therefore, we recommend the building foundation and retaining wall, as well as exterior landscape walls be supported on drilled, cast-in-place pier foundations. Recommendations are presented in the “Foundations” section below.

5.1.2 Excavation Difficulties Due to Shallow Bedrock

Based on our experience with the geology in the site vicinity, our exploratory borings, and considering cuts for the proposed structure will be on the order of 5 to 10 feet, we anticipate that excavations for the garage and second floor retaining walls will encounter bedrock. Small excavators and backhoes may have difficulty excavating through the bedrock. If localized harder, cemented bedrock is encountered, it may require the use of larger equipment or a different excavation technique. Additionally, based on our experience and the bedrock materials encountered, slower drilling rates will occur in the bedrock and the use of rock augers should be anticipated by the contractor when drilling the pier foundations.

5.1.3 Surficial Erosion

As discussed above, we observed shallow soil erosion that appeared to stem from the upslope storm drain manhole discharge onto the slope. We recommend that the storm drain outfall be diverted/modified to prevent storm water discharge onto the slope, and that drainage of upslope areas from the proposed structure be directed away from foundation elements to discharge to a free draining outlet.

5.1.4 Potential for Perched Groundwater Conditions

Groundwater was not encountered during our field investigation; however, seasonally perched groundwater conditions are anticipated following periods of heavy rainfall. Perched groundwater may be encountered within the cuts into bedrock that will need to be addressed both as a temporary construction consideration and to mitigate long-term seepage. To reduce the potential water seepage beneath foundations, we recommend that adequate subsurface drainage be installed behind all retaining walls and around building foundations. Walls within habitable areas/areas with moisture sensitive floor coverings should be waterproofed. Recommendations addressing this concern are presented in the following sections.

5.2 PLANS AND SPECIFICATIONS REVIEW

We recommend that we be retained to review the geotechnical aspects of the project structural, civil, and landscape plans and specifications, allowing sufficient time to provide the design team with any comments prior to issuing the plans for construction.

5.3 CONSTRUCTION OBSERVATION AND TESTING

As site conditions may vary significantly between the small-diameter borings performed during this investigation, we also recommend that a Cornerstone representative be present to provide geotechnical observation and testing during earthwork and foundation construction. This will allow us to form an opinion and prepare a letter at the end of construction regarding contractor compliance with project plans and specifications, and with the recommendations in our report. We will also be allowed to evaluate any conditions differing from those encountered during our investigation and provide supplemental recommendations as necessary. For these reasons, the recommendations in this report are contingent of Cornerstone providing observation and testing during construction. Contractors should provide at least a 48-hour notice when scheduling our field personnel.

SECTION 6: EARTHWORK

6.1 SITE DEMOLITION

All existing improvements not to be reused for the current development, including any foundations, flatwork, pavements, utilities, and other improvements should be demolished and removed from the site. Recommendations in this section apply to the removal of these improvements, which may be present on the site, prior to the start of mass grading or the construction of new improvements for the project.

Cornerstone should be notified prior to the start of demolition and should be present on at least a part-time basis during all backfill and mass grading as a result of demolition.

6.1.2 Abandonment of Existing Utilities

Any existing utilities should be completely removed from within planned building areas. For any utility line to be considered acceptable to remain within building areas, the utility line must be completely backfilled with grout or sand-cement slurry (sand slurry is not acceptable), the ends outside the building area capped with concrete, and the trench fills either removed and replaced as engineered fill with the trench side slopes flattened to at least 1:1, or the trench fills are determined not to be a risk to the structure. The assessment of the level of risk posed by the particular utility line will determine whether the utility may be abandoned in place or needs to be completely removed. The contractor should assume that all utilities will be removed from within building areas unless provided written confirmation from both the owner and the geotechnical engineer.

Utilities extending beyond the building area may be abandoned in place provided the ends are plugged with concrete, they do not conflict with planned improvements, and that the trench fills do not pose significant risk to the planned surface improvements.

The risk for owners associated with abandoning utilities in place include the potential for future differential settlement of existing trench fills, and/or partial collapse and potential ground loss into utility lines that are not completely filled with grout.

6.2 SITE CLEARING AND PREPARATION

6.2.1 Site Stripping

The site should be stripped of all surface vegetation, and surface and subsurface improvements to be removed within the proposed development area. Surface vegetation and topsoil should be stripped to a sufficient depth to remove all material greater than 3 percent organic content by weight. Based on our site observations, surficial stripping should extend about 3 to 6 inches below existing grade in vegetated areas.

6.2.2 Tree and Shrub Removal

Trees and shrubs designated for removal should have the root balls and any roots greater than ½-inch diameter removed completely. Mature trees are estimated to have root balls extending to depths of 2 to 4 feet, depending on the tree size. Significant root zones are anticipated to extend to the diameter of the tree canopy. Grade depressions resulting from root ball removal should be cleaned of loose material and backfilled in accordance with the recommendations in the "Compaction" section of this report.

6.3 TEMPORARY CUT AND FILL SLOPES

The contractor is responsible for maintaining all temporary slopes and providing temporary shoring where required. Temporary shoring, bracing, and cuts/fills should be performed in accordance with the strictest government safety standards. On a preliminary basis, the upper 5 feet of surficial clayey soil may be classified as Site C material, transitioning to Site B material where bedrock is encountered. A Cornerstone representative should be retained to confirm the preliminary site classification. If temporary shoring is considered for the planned retaining wall cuts, we should be contacted to review and provide supplemental recommendations, as needed.

Excavations for the planned first and second level retaining walls should be sloped in accordance with OSHA soil classification requirements unless temporary shoring is planned.

6.4 SUBGRADE PREPARATION

After site clearing and demolition is complete, and prior to backfilling any excavations resulting from planned cuts or demolition, the excavation subgrade and subgrade within areas to receive additional site fills, slabs-on-grade and/or pavements should be scarified to a depth of 6 inches, moisture conditioned, and compacted in accordance with the "Compaction" section below.

6.4.1 Cut/Fill Transition

As discussed above, we anticipate that the garage level of the proposed residence will be supported on a slab-on-grade and that a cut/fill transition may occur beneath the proposed slab-on-grade. Building pads with cut/fill transitions should be over-excavated to provide a relatively uniform fill thickness beneath the building footprint. The depth of over-excavation below pad grade should be equal to the maximum fill thickness in the pad but not exceed 3 feet. We should review the final grading and foundation plans to confirm if over-excavation is required for garage level slab-on-grade.

6.5 MATERIAL FOR FILL

6.5.1 Re-Use of On-site Soils

On-site soils with an organic content less than 3 percent by weight may be reused as general fill. General fill should not have lumps, clods or cobble pieces larger than 6 inches in diameter; 85 percent of the fill should be smaller than 2½ inches in diameter. Minor amounts of oversize material (smaller than 12 inches in diameter) may be allowed provided the oversized pieces are not allowed to nest together and the compaction method will allow for loosely placed lifts not exceeding 12 inches.

6.5.2 Potential Import Sources

Imported soil for use as general fill material should be inorganic with a Plasticity Index (PI) of 15 or less, and not contain recycled asphalt concrete where it will be used within the habitable building areas. To prevent significant caving during trenching or foundation construction, imported material should have sufficient fines. Samples of potential import sources should be delivered to our office at least 10 days prior to the desired import start date. Information regarding the import source should be provided, such as any site geotechnical reports. If the material will be derived from an excavation rather than a stockpile, potholes will likely be required to collect samples from throughout the depth of the planned cut that will be imported. At a minimum, laboratory testing will include PI tests. Material data sheets for select fill materials (Class 2 aggregate base, ¾-inch crushed rock, quarry fines, etc.) listing current laboratory testing data (not older than 6 months from the import date) may be provided for our review without providing a sample. If current data is not available, specification testing will need to be completed prior to approval.

Environmental and soil corrosion characterization should also be considered by the project team prior to acceptance. Suitable environmental laboratory data to the planned import quantity should be provided to the project environmental consultant; additional laboratory testing may be required based on the project environmental consultant's review. The potential import source should also not be more corrosive than the on-site soils, based on pH, saturated resistivity, and soluble sulfate and chloride testing.

6.6 COMPACTION REQUIREMENTS

All fills, and subgrade areas where fill, slabs-on-grade and pavements are planned, should be placed in loose lifts 8 inches thick or less and compacted in accordance with ASTM D1557 (latest version) requirements as shown in the table below. In general, clayey soils should be compacted with sheepsfoot equipment and sandy/gravelly soils with vibratory equipment; open-graded materials such as crushed rock should be placed in lifts no thicker than 18 inches consolidated in place with vibratory equipment. Each lift of fill and all subgrade should be firm and unyielding under construction equipment loading in addition to meeting the compaction requirements to be approved. The contractor (with input from a Cornerstone representative) should evaluate the in-situ moisture conditions, as the use of vibratory equipment on soils with high moistures can cause unstable conditions.

Table 2: Compaction Requirements

Description	Material Description	Minimum Relative[1] Compaction (percent)	Moisture[2] Content (percent)
General Fill	On-Site Soils	90	>1
Retaining Wall Backfill	Without Surface Improvements	90	>1
	With Surface Improvements	95[4]	>1
Trench Backfill	On-Site Soils	90	>1
Trench Backfill (upper 6 inches of pavement subgrade)	On-Site Soils	95	>1
Crushed Rock Fill	¾-inch Clean Crushed Rock	Consolidate In-Place	NA
Flatwork Subgrade	On-Site Soils	90	>1
Flatwork Aggregate Base	Class 2 Aggregate Base[3]	90	Optimum
Pavement Subgrade	On-Site Soils	95	>1
Pavement Aggregate Base	Class 2 Aggregate Base[3]	95	Optimum

1 – Relative compaction based on maximum density determined by ASTM D1557 (latest version)
2 – Moisture content based on optimum moisture content determined by ASTM D1557 (latest version)
3 – Class 2 aggregate base shall conform to Caltrans Standard Specifications, latest edition, except that the relative compaction should be determined by ASTM D1557 (latest version)
4 – Using light-weight compaction or walls should be braced

6.7 TRENCH BACKFILL

Utility lines constructed within public right-of-way should be trenched, bedded and shaded, and backfilled in accordance with the local or governing jurisdictional requirements. Utility lines in private improvement areas should be constructed in accordance with the following requirements unless superseded by other governing requirements.

All utility lines should be bedded and shaded to at least 6 inches over the top of the lines with crushed rock (⅜-inch-diameter or greater) or well-graded sand and gravel materials conforming to the pipe manufacturer's requirements. Open-graded shading materials should be consolidated in place with vibratory equipment and well-graded materials should be compacted to at least 90 percent relative compaction with vibratory equipment prior to placing subsequent backfill materials.

General backfill over shading materials may consist of on-site native materials provided they meet the requirements in the "Material for Fill" section, and are moisture conditioned and compacted in accordance with the requirements in the "Compaction" section.

Where utility lines will cross perpendicular to strip footings, the footing should be deepened to encase the utility line, providing sleeves or flexible cushions to protect the pipes from anticipated foundation settlement, or the utility lines should be backfilled to the bottom of footing with sand-cement slurry or lean concrete. Where utility lines will parallel footings and will extend below the "foundation plane of influence," an imaginary 1:1 plane projected down from the bottom edge of the footing, either the footing will need to be deepened so that the pipe is above the foundation plane of influence or the utility trench will need to be backfilled with sand-cement slurry or lean concrete within the influence zone. Sand-cement slurry used within foundation influence zones should have a minimum compressive strength of 75 psi.

6.8 PERMANENT CUT AND FILL SLOPES

All permanent fill slopes, as well as cut slopes in soil or bedrock, should have a maximum inclination of 2:1 (horizontal:vertical). Fill slopes should be overbuilt and trimmed back, exposing engineered fill when complete. Refer to the "Erosion Control" section of this report for a discussion regarding protection of slope surfaces.

6.9 SITE DRAINAGE

6.9.1 Surface Drainage

Surface runoff should not be allowed to flow over the top of or pond at the top or toe of engineered fill slopes or retaining walls. Ponding should also not be allowed on or adjacent to building foundations, slabs-on-grade, or pavements. Hardscape surfaces should slope at least 2 percent towards suitable discharge facilities; landscape areas should slope at least 3 percent towards suitable discharge facilities. Roof runoff should be directed away from building areas in closed conduits, to approved infiltration facilities, or on to hardscaped surfaces that drain to suitable facilities. Retention, detention or infiltration facilities should be spaced at least 10 feet from buildings, and preferably at least 5 feet from slabs-on-grade or pavements. These facilities are not recommended where stormwater infiltration may affect slopes at lower elevations on or adjacent to the site.

Lined or unlined v-ditches should be included at the toe of slopes or behind retaining walls. All v-ditches and drain inlets should be sized to accommodate the design storm events for the upslope tributary area. If considered, concrete-lined v-ditches should be reinforced as required and have adequate control and construction joints and should be constructed neat in excavations; backfill around formed ditches should not be allowed.

As discussed in the "Conclusions" section, upslope sources of water should be evaluated, such as the existing storm drain outfall or upslope properties. If upslope irrigation is present or planned, additional surface and subsurface drainage may be needed to protect site improvements. We should be consulted if this issue will affect the project.

6.9.2 Subsurface Drainage

For residential lots with sloping ground conditions, water accumulation in crawl spaces or below slabs-on-grade is possible even if adequate surface drainage is provided adjacent to the structure. Although water seepage below foundations does not generally affect foundation performance from a geotechnical viewpoint, it may have undesirable impacts to the floor system, such as rusting joist hangers or mildew/mold on subfloor insulation or ductwork.

To reduce water seepage into potential crawl space areas, a perimeter trench drain may be required depending on the location of retaining wall drains relative to the location of exterior improvements. If required, the trench drain should be located on the uphill and sides of the foundations and should discharge to a free draining outlet. To further reduce potential standing water or moist soil below foundations, exposed crawl space soil should be graded to drain to a common low point designed with a surface drain inlet. Adequate crawl space ventilation should also be provided to aid seasonal drying of locally moist soil and reduce humidity beneath the foundation. Supplemental recommendations can be provided, as needed, once grading and foundation plans have been prepared.

6.10 PERMANENT EROSION CONTROL MEASURES

Hillside grading will require periodic maintenance after construction to reduce the potential for erosion and sloughing. At a minimum all graded slopes should be vegetated by hydroseeding or other landscape ground cover. The establishment of vegetation will help reduce runoff velocities, allow some infiltration and transpiration, trap sediment within runoff, and protect the soil from raindrop impact. Depending on the exposed material type and the slope inclination, more aggressive erosion control measures may be needed to protect slopes for one or more winter seasons while vegetation is establishing. For slopes with inclinations of 2:1 (horizontal:vertical), erosion control may consist of jute netting, straw matting, or erosion control blankets used in combination with hydroseeding.

Both construction and post-construction Storm Water Pollution Prevention Plans (SWPPPs) should be prepared for the project-specific requirements. We recommend that final grading plans be provided for our review.

6.11 LANDSCAPE CONSIDERATIONS

To reduce water migration below proposed foundations and retaining walls, we recommend greatly reducing the amount of surface water infiltrating these soils near foundations and exterior slabs-on-grade. This can typically be achieved by:

- Using drip irrigation,
- Avoiding open planting within 3 feet of the building perimeter or near the top of existing slopes,
- Regulating the amount of water distributed to lawns or planter areas by using irrigation timers, and
- Selecting landscaping that requires little or no watering, especially near foundations.

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

SECTION 7: 2019 CBC SEISMIC DESIGN CRITERIA

7.1 SEISMIC DESIGN CRITERIA

We understand that the project structural design will be based on the 2019 California Building Code (CBC), which provides criteria for the seismic design of buildings in Chapter 16. The "Seismic Coefficients" used to design buildings are established based on a series of tables and figures addressing different site factors, including the soil profile in the upper 100 feet below grade and mapped spectral acceleration parameters based on distance to the controlling seismic source/fault system.

Our explorations generally encountered bedrock a depth of 3½ to 6½ feet. Based on our borings and review of local geology, the site is underlain by shallow rock with typical SPT "N" values above 50 blows per foot. Therefore, we have classified the site as Soil Classification C. The mapped spectral acceleration parameters S_s and S_1 were calculated using the web-based program ATC Hazards by Locations, located at https://hazards.atcouncil.org/, based on the site coordinates presented below and the site classification. Recommended values for design are presented in Table 3. The table below lists the various factors used to determine the seismic coefficients and other parameters.

Table 3: CBC Site Categorization and Site Coefficients

Classification/Coefficient	Design Value
Site Class	C
Site Latitude	37.84089°
Site Longitude	-122.215766°
0.2-second Period Mapped Spectral Acceleration[1], S_S	2.353g
1-second Period Mapped Spectral Acceleration[1], S_1	0.905g
Short-Period Site Coefficient – Fa	1.2
Long-Period Site Coefficient – Fv	1.4
0.2-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects - S_{MS}	2.824g
1-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects – S_{M1}	1.267g
0.2-second Period, Design Earthquake Spectral Response Acceleration – S_{DS}	1.882g
1-second Period, Design Earthquake Spectral Response Acceleration – S_{D1}	0.845g

SECTION 8: FOUNDATIONS

8.1 SUMMARY OF RECOMMENDATIONS

As discussed in the "Conclusions" section, due to the potential for differential settlement and the existing moderately steep sloping ground, we recommend the proposed residence and any exterior retaining walls be founded on drilled piers and designed with the parameters recommended below.

8.2 DEEP FOUNDATIONS

8.2.1 Drilled Pier

The proposed residential structure and site retaining walls may be supported on drilled, cast-in-place, straight-shaft friction piers. The piers should have a minimum diameter of 16 inches and extend to a depth of at least 10 feet below the bottom of the grade beams or at least 5 feet into competent bedrock, whichever is greater. Adjacent piers centers should be spaced at least three diameters apart, otherwise, a reduction for group effects may be required. Grade beams should span between piers and/or pier caps in accordance with structural requirements. Conventional slabs-on-grade may be used provided the subgrade soils are prepared in accordance with the "Earthwork" section.

8.2.2 Vertical and Lateral Capacity

The vertical and lateral capacity of the piers may be designed using the criteria summarized in the following table. Allowable vertical capacity is for combined dead plus live loads based on a factor of safety of 2.0; dead loads should not exceed two-thirds of the allowable capacities. The allowable skin friction may be increased by one-third for wind and seismic loads. Frictional resistance to uplift loads may be developed along the pier shafts based on an allowable frictional resistance of 80 percent of the downward capacities.

Table 4: Summary of Drilled Pier Design Criteria

Design Criteria	Design Value	Comments
Minimum Pier Diameter	16 inches	--
Minimum Pier Depth	10 feet or 5 feet into competent rock, which ever is deeper	Below bottom of grade beam or lowest adjacent grade
Minimum Pier Spacing	3 pier diameters	--
Allowable Skin Friction	600 psf for D+L loads	For piers on level cut areas
	400 psf for D+L loads	For piers on sloping ground
Allowable Passive Resistance	350 pcf EFP* to max. 3,000 psf at depth	For piers on level cut areas; applied over 2 pier diameters
	300 pcf EFP to max. 3,000 psf at depth	For piers on sloping ground; applied over 2 pier diameters
Depth to Neglect	12 inches below surface grades beams	Not required for grade beams embedded at least 12 inches deep.
Lateral Creep Force	60 pcf EFP	Applied to upper 2 feet of piers situated on sloping ground; not required for piers on flat tiered building pads

*EFP = equivalent fluid pressure

8.2.3 Drilled Pier Settlement

Total settlement of individual piers should not exceed ¼ to ½ inch to mobilize static capacities, and post-construction differential settlement over a horizontal distance of 20 feet should not exceed ¼ inch due to static loads.

8.2.4 Construction Considerations

The excavation of all drilled shafts should be observed by a Cornerstone representative to confirm the bedrock profile, verify that the piers extend the minimum depth into suitable materials and that the piers are constructed in accordance with our recommendations and project requirements. The drilled shafts should be straight, dry, and relatively free of loose material before reinforcing steel is installed and concrete is placed. If perched ground water is encountered and cannot be removed from the excavations prior to concrete placement, the concrete should be placed using a tremie pipe, keeping the tremie pipe below the surface of the concrete to avoid entrapment of water in the concrete.

If drilling refusal is encountered due to hard or resistant bedrock material, the pier capacity and acceptance of final pier depth should be evaluated on a case-by case basis by Cornerstone Earth Group in coordination with the structural engineer. Acceptable refusal criteria will depend on the actual pier depth, bedrock material encountered, as well as type of drill rig, drill bit and level of effort used. We recommend that local drilling contractors experienced in rock drilling methods be contacted to aid in determining the efficiency, time and costs associated with excavations in these types of bedrock.

SECTION 9: CONCRETE SLABS AND PEDESTRIAN PAVEMENTS

9.1 INTERIOR SLABS-ON-GRADE

As the Plasticity Index (PI) of the surficial soils is 15 or less, the proposed garage slabs-on-grade should be underlain by at least 4 inches of Class 2 aggregate base or crushed rock overlying subgrade soil prepared in accordance with the recommendations in the "Earthwork" section of this report. If moisture-sensitive floor coverings are planned, the recommendations in the "Interior Slabs Moisture Protection Considerations" section below may be incorporated in the project design if desired. If significant time elapses between initial subgrade preparation and slab-on-grade construction, the subgrade should be proof-rolled to confirm subgrade stability, and if the soil has been allowed to dry out, the subgrade should be re-moisture conditioned to near optimum moisture content.

The structural engineer should determine the appropriate slab reinforcement for the loading requirements. For unreinforced concrete slabs, ACI 302.1R recommends limiting control joint spacing to 24 to 36 times the slab thickness in each direction or a maximum of 18 feet. We recommend the garage slab-on-grade be isolated from the adjacent perimeter grade beams or retaining walls.

9.2 INTERIOR SLABS MOISTURE PROTECTION CONSIDERATIONS

The following general guidelines for concrete slab-on-grade construction where floor coverings are planned are presented for the consideration by the developer, design team, and contractor. These guidelines are based on information obtained from a variety of sources, including the American Concrete Institute (ACI) and are intended to reduce the potential for moisture-related problems causing floor covering failures, and may be supplemented as necessary based on project-specific requirements. The application of these guidelines or not will not affect the geotechnical aspects of the slab-on-grade performance.

- Place a minimum 15 mil vapor retarder conforming to ASTM E 1745, Class C requirements or better directly below the concrete slab; the vapor retarder should extend to the slab edges and be sealed at all seams and penetrations in accordance with manufacturer's recommendations and ASTM E 1643 requirements. A 4-inch-thick capillary break, consisting of crushed rock should be placed below the vapor retarder and consolidated in place with vibratory equipment. The mineral aggregate shall be of such size that the percentage composition by dry weight as determined by laboratory sieves will conform to the following gradation:

Sieve Size	Percentage Passing Sieve
1"	100
¾"	90 – 100

No. 4	0 – 10
No. 200	0 – 5

- The concrete water:cement ratio should be 0.45 or less. Mid-range plasticizers may be used to increase concrete workability and facilitate pumping and placement.
- Water should not be added after initial batching unless the slump is less than specified and/or the resulting water:cement ratio will not exceed 0.45.
- Polishing the concrete surface with metal trowels is not recommended.
- Where floor coverings are planned, all concrete surfaces should be properly cured.
- Water vapor emission levels and concrete pH should be determined in accordance with ASTM F1869-98 and F710-98 requirements and evaluated against the floor covering manufacturer's requirements prior to installation.

9.3 DRIVEWAYS AND EXTERIOR FLATWORK

Driveways should be at least 5 inches thick and supported on at least 4 inches of Class 2 aggregate base overlying subgrade prepared in accordance with the "Earthwork" recommendations of this report. Exterior concrete flatwork subject to pedestrian loading should be at least 4 inches thick and may be constructed directly over subgrade soil prepared in accordance with the "Earthwork" recommendations of this report. To help reduce the potential for uncontrolled shrinkage cracking, adequate expansion and control joints should be included. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness. Flatwork should be isolated from adjacent foundations or retaining walls except where limited sections of structural slabs are included to help span irregularities in retaining wall backfill at the transitions between at-grade and on-structure flatwork.

SECTION 10: RETAINING WALLS

10.1 STATIC LATERAL EARTH PRESSURES

The structural design of any site retaining wall should include resistance to lateral earth pressures that develop from the soil behind the wall, any undrained water pressure, and surcharge loads acting behind the wall. Provided a drainage system is constructed behind the wall to prevent the build-up of hydrostatic pressures as discussed in the section below, we recommend that the walls be designed for the following pressures:

Table 5: Recommended Lateral Earth Pressures

Sloping Backfill Inclination (horizontal:vertical)	Lateral Earth Pressure*	
	Unrestrained – Cantilever Wall	Restrained – Braced Wall

Level	45 pcf	45 pcf + 8H
3:1	55 pcf	55 pcf + 8H
2½:1	60 pcf	60 pcf + 8H
2:1	65 pcf	65 pcf + 8H
Additional Surcharge Loads	⅓ of vertical loads at top of wall	½ of vertical loads at top of wall

* Lateral earth pressures are based on an equivalent fluid pressure
** H is the distance in feet between the bottom of footing and top of retained soil

Lower and mid-level foundation walls should be designed as restrained walls. If adequate drainage cannot be provided behind the wall, an additional equivalent fluid pressure of 40 pcf should be added to the values above for both restrained and unrestrained walls for the portion of the wall that will not have drainage. Damp proofing or waterproofing of the walls may be considered where moisture penetration and/or efflorescence are not desired.

10.2 SEISMIC LATERAL EARTH PRESSURES

The 2019 California Building Code (CBC) states that lateral pressures from earthquakes should be considered in the design of basements and retaining walls. We checked seismic earth pressures for the proposed restrained and unrestrained (cantilever) retaining walls in accordance with CBC 1803.5.12 and ASCE 7-16 Section 11.6.3 using the Design level earthquake.

Because the lower and mid-level walls are restrained, or will act as a restrained wall, and will be designed for 45 pcf (equivalent fluid pressure) plus a uniform earth pressure of 8H psf, based on current recommendations for seismic earth pressures, it appears that active earth pressures plus a seismic increment do not exceed the fixed wall earth pressures. Therefore, an additional seismic increment above the design earth pressures is not required as long as the walls are designed for the restrained wall earth pressures recommended above in accordance with the CBC. Exterior cantilevered site walls are anticipated to be 6 feet high or less; therefore, are not required to be designed to resist seismic earth pressures.

10.3 WALL DRAINAGE

Adequate drainage should be provided by a subdrain system behind all walls. This system should consist of a 4-inch minimum diameter perforated pipe placed near the base of the wall (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 approved equivalent. The upper 2 feet of wall backfill should consist of compacted on-site soil. The subdrain outlet should be connected to a free-draining outlet or sump.

Miradrain, Geotech Drainage Panels, or equivalent drainage matting can be used for wall drainage as an alternative to the Class 2 Permeable Material or drain rock backfill. Horizontal strip drains connecting to the vertical drainage matting may be used in lieu of the perforated pipe and crushed rock section. The vertical drainage panel should be connected to the perforated pipe or horizontal drainage strip at the base of the wall, or to some other closed or through-wall system such as the TotalDrain system from AmerDrain. Sections of horizontal drainage strips should be connected with either the manufacturer's connector pieces or by pulling back the filter fabric, overlapping the panel dimples, and replacing the filter fabric over the connection. At corners, a corner guard, corner connection insert, or a section of crushed rock covered with filter fabric must be used to maintain the drainage path.

Drainage panels should terminate 18 to 24 inches from final exterior grade. 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.

10.4 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.

Consideration should be given to the transitions from on-grade to on-structure. Providing subslabs or other methods for reducing differential movement of flatwork or pavements across this transition should be included in the project design.

10.5 FOUNDATIONS

Retaining walls may be supported on drilled pier foundations designed in accordance with the recommendations presented in the "Foundations" section of this report.

SECTION 11: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of Mr. Dong Zhang specifically to support the design of the Broadway Terrace Residence project in Oakland, California. The opinions, conclusions, and recommendations presented in this report have been formulated in accordance with accepted geotechnical engineering practices that exist in Northern California at the time this report was prepared. No warranty, expressed or implied, is made or should be inferred.

Recommendations in this report are based upon the soil and ground water conditions encountered during our subsurface exploration. If variations or unsuitable conditions are encountered during construction, Cornerstone must be contacted to provide supplemental recommendations, as needed.

Mr. Dong Zhang may have provided Cornerstone with plans, reports and other documents prepared by others. Mr. Dong Zhang understands that Cornerstone reviewed and relied on the information presented in these documents and cannot be responsible for their accuracy.

Cornerstone prepared this report with the understanding that it is the responsibility of the owner or his representatives to see that the recommendations contained in this report are presented to other members of the design team and incorporated into the project plans and specifications, and that appropriate actions are taken to implement the geotechnical recommendations during construction.

Conclusions and recommendations presented in this report are valid as of the present time for the development as currently planned. Changes in the condition of the property or adjacent properties may occur with the passage of time, whether by natural processes or the acts of other persons. In addition, changes in applicable or appropriate standards may occur through legislation or the broadening of knowledge. Therefore, the conclusions and recommendations presented in this report may be invalidated, wholly or in part, by changes beyond Cornerstone's control. This report should be reviewed by Cornerstone after a period of three (3) years has elapsed from the date of this report. In addition, if the current project design is changed, then Cornerstone must review the proposed changes and provide supplemental recommendations, as needed.

An electronic transmission of this report may also have been issued. While Cornerstone has taken precautions to produce a complete and secure electronic transmission, please check the electronic transmission against the hard copy version for conformity.

Recommendations provided in this report are based on the assumption that Cornerstone 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, Cornerstone cannot assume any responsibility for any potential claims that may arise during or after construction as a result of misuse or misinterpretation of Cornerstone's report by others. Furthermore, Cornerstone will cease to be the Geotechnical-Engineer-of-Record if we are not retained for these services.

SECTION 12: REFERENCES

ATC Hazards by Location, Hazards by Location, 2020, https://hazards.atcouncil.org/

ASCE 7-16. American Society of Civil Engineers. (2016). *Minimum Design Loads and Associated Criteria for Buildings and Other Structures.*

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California Department of Conservation Division of Mines and Geology, 1998, Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada, International Conference of Building Officials, February, 1998.

California Division of Mines and Geology (2008), “Guidelines for Evaluating and Mitigating Seismic Hazards in California, Special Publication 117A, September.

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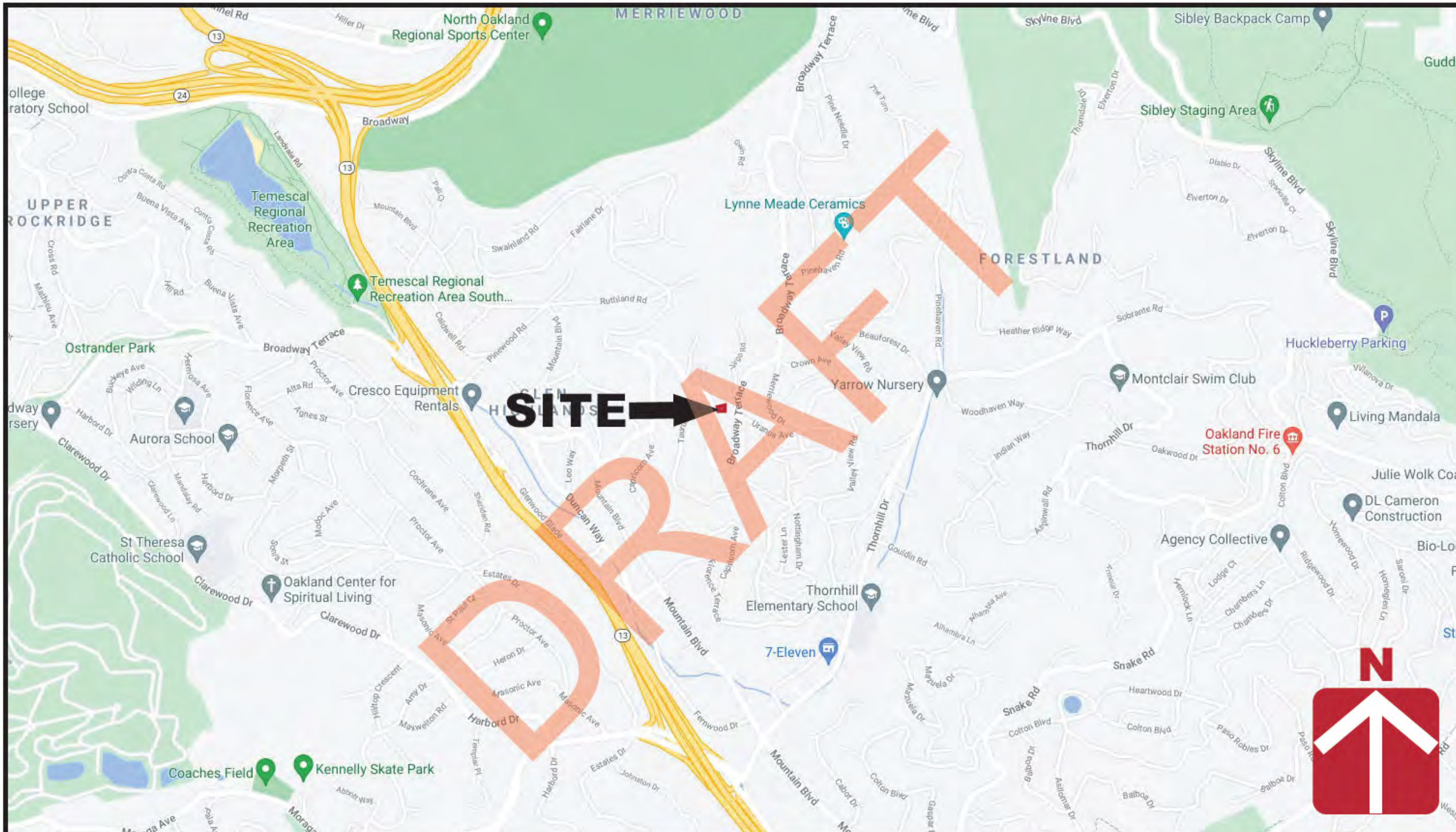
CGS. (2021). *Earthquake Zones of Required Investigation.* CGS Homepage. https://maps.conservation.ca.gov/cgs/EQZApp/app/

Working Group on California Earthquake Probabilities, 2015, The Third Uniform California Earthquake Rupture Forecast, Version 3 (UCERF), U.S. Geological Survey Open File Report 2013-1165 (CGS Special Report 228). *KMZ files available at:* www.scec.org/ucerf/images/ucerf3_timedep_30yr_probs.kmz

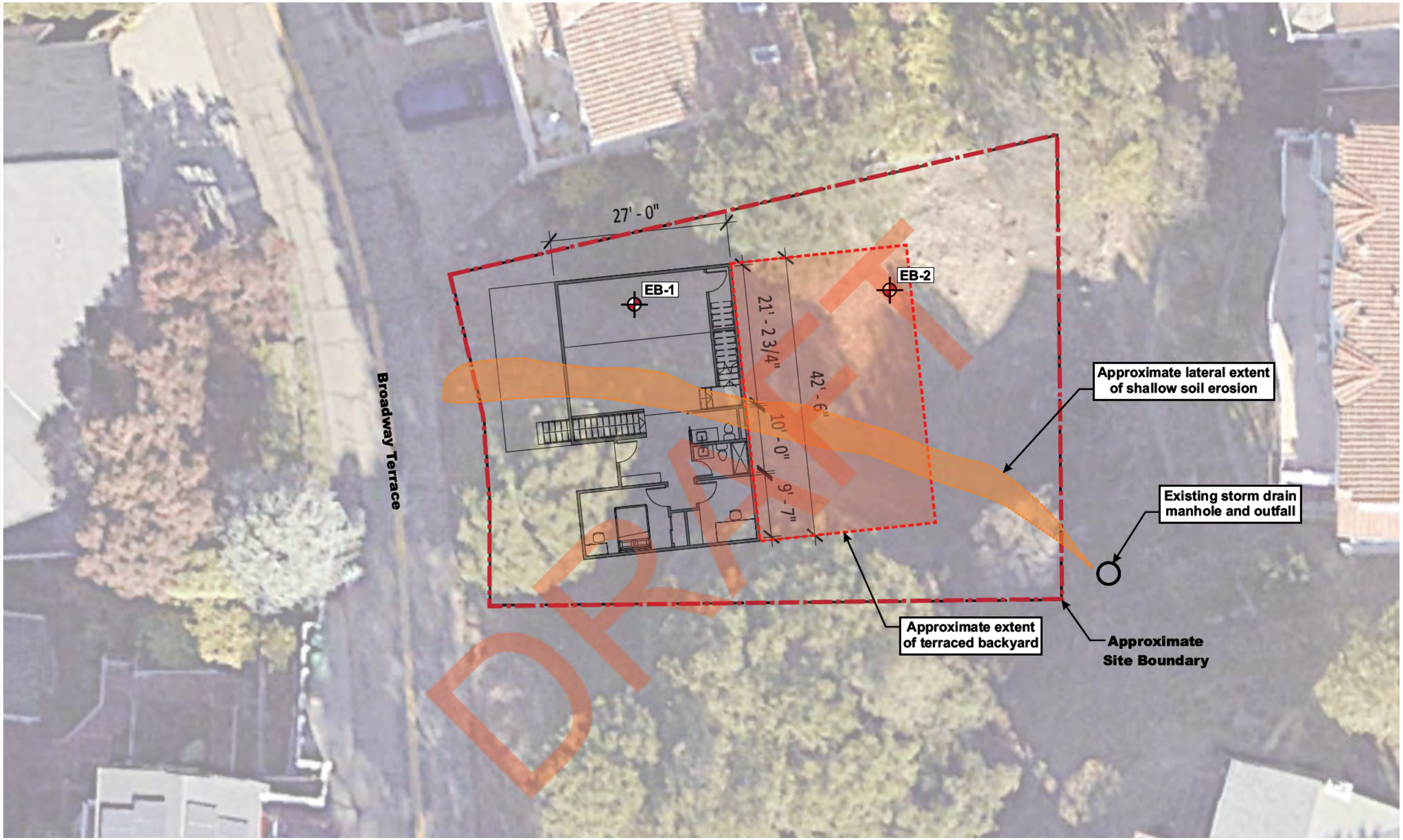


CORNERSTONE EARTH GROUP

Vicinity Map

10040 Broadway Terrace Residential
Oakland, CA

Project Number	1300-1-1
Figure Number	Figure 1
Date	August 2021
Drawn By	RRN



Legend

Approximate location of exploratory boring (EB)

N

0 15 30

APPROXIMATE SCALE (FEET)

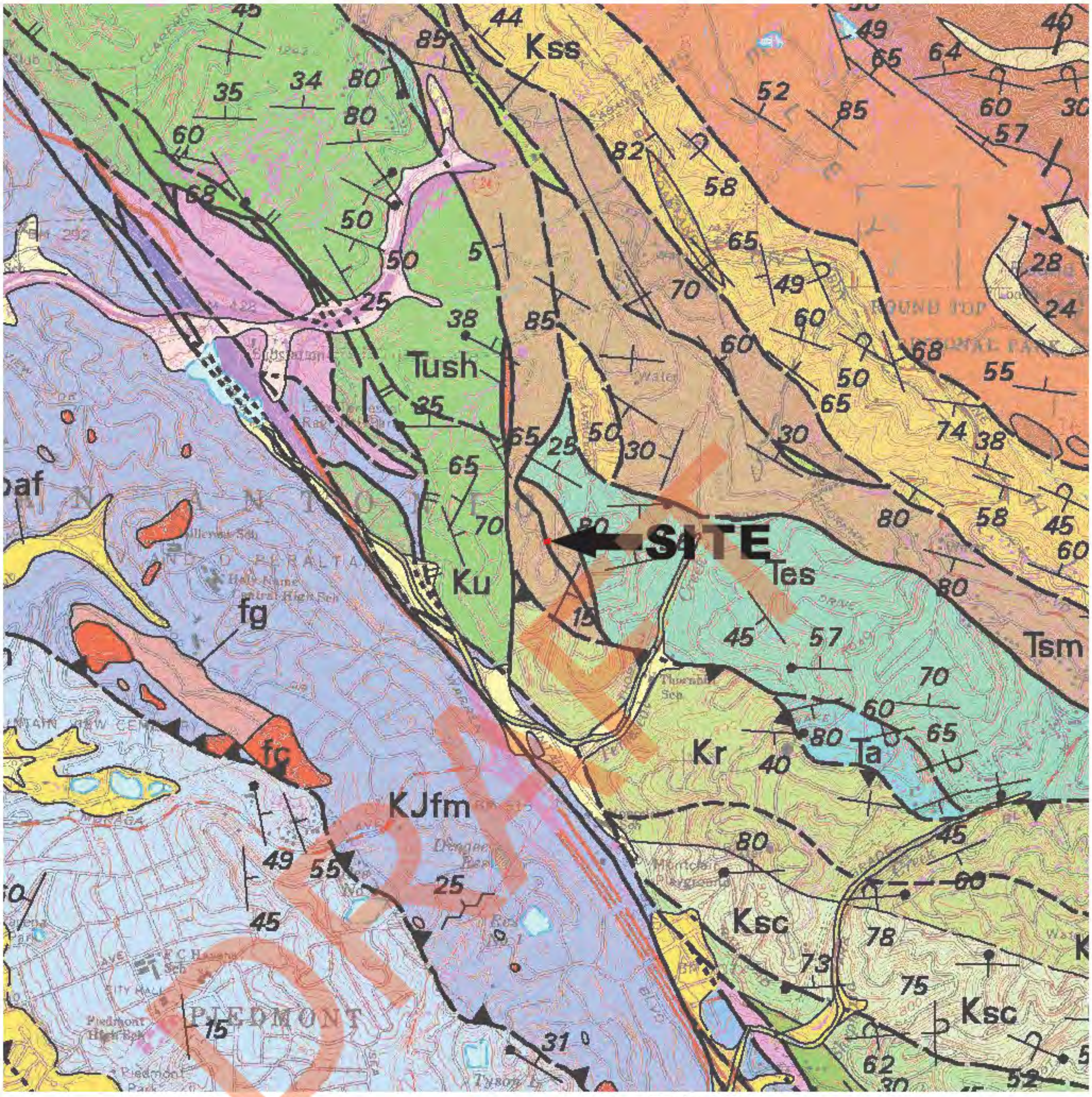
Base by Google Earth, dated 08/06/2020
Overlay by HPA, Inc., Floor Plans, Second Level Plan - A2, undated

Project Number 1300-1-1
Figure Number Figure 2
Drawn By RRN
Date August 2021

Site Plan

10040 Broadway Terrace Residential
Oakland, CA

CORNERSTONE EARTH GROUP



Geologic Units	
Tush	Unnamed gray mudstone (early Miocene)
Tsm	Unnamed glauconitic mudstone (Miocene and Oligocene)
Tes	Unnamed mudstone (Eocene)
Kss	Unnamed lithic sandstone (Cretaceous)
Ku	Unnamed sedimentary rocks (Late Cretaceous, Turonian and Cenomanion)
Kr	Undivided Great Valley complex rocks (Cretaceous)

Explanation

Contact - dashed where approximate, dotted where concealed

Fault - dashed where approximate, dotted where concealed

N

0 2000 4000

APPROXIMATE SCALE (FEET)

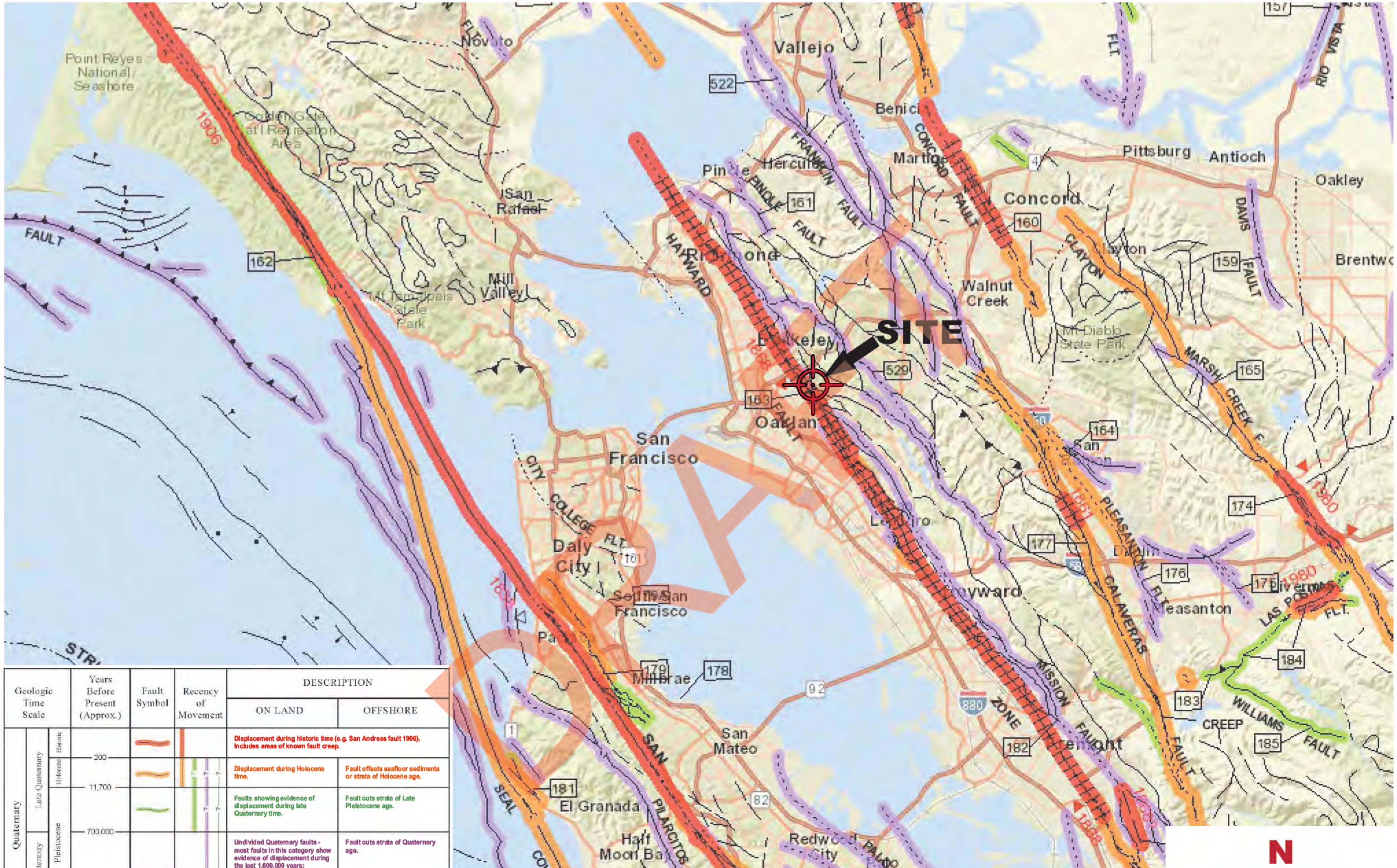
Base: USGS, Geologic Map and Map Database of the Oakland Metropolitan Area, Alameda, Contra Costa, and San Francisco Counties, California, by Graymer, 2000

CORNERSTONE EARTH GROUP

Vicinity Geologic Map

10040 Broadway Terrace Residential
Oakland, CA

Project Number	1300-1-1
Figure Number	Figure 3
Date	August 2021
Drawn By	RRN



Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION: ON LAND	DESCRIPTION: OFFSHORE
Quaternary	Late Quaternary	Historic				Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
		Holocene	200			Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
		Pleistocene	11,700			Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
	Early Quaternary		700,000			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.
Pre-Quaternary			1,600,000 – 4.5 billion (Age of Earth)			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.

Base by California Geological Survey - 2010 Fault Activity Map of California (Jennings and Bryant, 2010)

N

0 5 10

APPROXIMATE SCALE (MILES)

Project Number: 1300-1-1

Figure Number: Figure 4

Date: August 2021

Drawn By: RRN

Regional Fault Map

10040 Broadway Terrace Residential
Oakland, CA

CORNERSTONE EARTH GROUP

APPENDIX A: FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using limited-access solid stem Minute-man drilling equipment. Two 4-inch-diameter exploratory borings were drilled on July 16,2021 to depths of 6½ to 8½ feet. The approximate locations of exploratory borings are shown on the Site Plan, Figure 2. The soils encountered were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). Boring logs, as well as a key to the classification of the soil and bedrock, are included as part of this appendix.

Boring locations were approximated using existing site boundaries, a hand held GPS unit, and other site features as references. Boring elevations were not determined. The locations of the borings 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. The standard penetration resistance blow counts were obtained by dropping a 140-pound hammer through a 30-inch free fall. The 2-inch O.D. split-spoon sampler was driven 18 inches and the number of blows was recorded for each 6 inches of penetration (ASTM D1586). 2.5-inch I.D. samples were obtained using a Modified California Sampler driven into the soil with the 140-pound hammer previously described. Unless otherwise indicated, the blows per foot recorded on the boring log represent the accumulated number of blows required to drive the last 12 inches. The various samplers are denoted at the appropriate depth on the boring logs.

Field tests included an evaluation of the unconfined compressive strength of the soil samples using a pocket penetrometer device. The results of these tests are presented on the individual boring logs at the appropriate sample depths.

Attached boring 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 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.

UNIFIED SOIL CLASSIFICATION (ASTM D-2487-10)

MATERIAL TYPES	CRITERIA FOR ASSIGNING SOIL GROUP NAMES			GROUP SYMBOL	SOIL GROUP NAMES & LEGEND
COARSE-GRAINED SOILS >50% RETAINED ON NO. 200 SIEVE	GRAVELS >50% OF COARSE FRACTION RETAINED ON NO 4. SIEVE	CLEAN GRAVELS <5% FINES	Cu>4 AND 1<Cc<3	GW	WELL-GRADED GRAVEL
			Cu>4 AND 1>Cc>3	GP	POORLY-GRADED GRAVEL
		GRAVELS WITH FINES >12% FINES	FINES CLASSIFY AS ML OR CL	GM	SILTY GRAVEL
			FINES CLASSIFY AS CL OR CH	GC	CLAYEY GRAVEL
	SANDS >50% OF COARSE FRACTION PASSES ON NO 4. SIEVE	CLEAN SANDS <5% FINES	Cu>6 AND 1<Cc<3	SW	WELL-GRADED SAND
			Cu>6 AND 1>Cc>3	SP	POORLY-GRADED SAND
		SANDS AND FINES >12% FINES	FINES CLASSIFY AS ML OR CL	SM	SILTY SAND
			FINES CLASSIFY AS CL OR CH	SC	CLAYEY SAND
FINE-GRAINED SOILS >50% PASSES NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT<50	INORGANIC	PI>7 AND PLOTS>"A" LINE	CL	LEAN CLAY
			PI>4 AND PLOTS<"A" LINE	ML	SILT
		ORGANIC	LL (oven dried)/LL (not dried)<0.75		ORGANIC CLAY OR SILT
	SILTS AND CLAYS LIQUID LIMIT>50	INORGANIC	PI PLOTS >"A" LINE	CH	FAT CLAY
			PI PLOTS < LINE	MH	ELASTIC SILT
		ORGANIC	LL (oven dried) ot drie .75	OH	ORGANIC CLAY OR SILT
HIGHLY ORGANIC SOILS		PRIMARILY ORGANIC MATTER, DARK IN COLOR, AND ORGANIC R		PT	PEAT

OTHER MATERIAL SYMBOLS

- Poorly-Graded Sand with Clay
- Clayey Sand
- Sandy Silt
- Artificial/Undocumented Fill
- Poorly-Graded Gravelly Sand
- Topsoil
- Well-Graded Gravel with Clay
- Well-Graded Gravel with Silt
- Sand
- Silt
- Well Grade ravelly Sand
- Gravelly
- Asphalt
- Boulders d Cobble

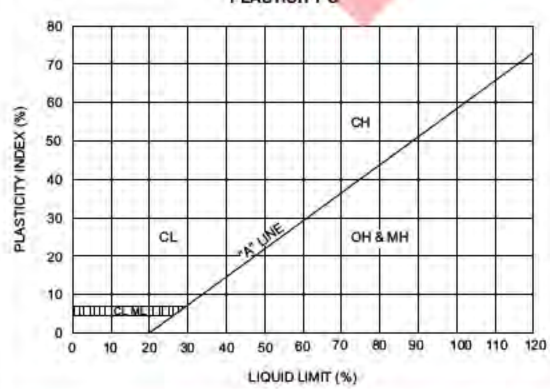
SAMPLER YPES

- SPT
- Modified California (2.5" I.D.)
- Rock Core
- Shelby Tube
- No Recovery
- Grab Sample

ADDITIONAL TESTS

CA	-	CHEMICAL ANALYSIS (CORROSIVITY)	PI	-	PLASTICITY INDEX
CD	-	CONSOLIDATED DRAINED TRIAXIAL	SW	-	SWELL TEST
CN	-	CONSOLIDATION	TC	-	CYCLIC TRIAXIAL
CU	-	CONSOLIDATED UNDRAINED TRIAXIAL	TV	-	TORVANE SHEAR
DS	-	DIRECT SHEAR	UC	-	UNCONFINED COMPRESSION
PP	-	POCKET PENETROMETER (TSF)	(1.5)	-	(WITH SHEAR STRENGTH IN KSF)
(3.0)	-	(WITH SHEAR STRENGTH IN KSF)			
RV	-	R-VALUE	UU	-	UNCONSOLIDATED UNDRAINED TRIAXIAL
SA	-	SIEVE ANALYSIS: % PASSING #200 SIEVE			
▼	-	WATER LEVEL			

PLASTICITY C



PENETRATION RESISTANCE (RECORDED AS BLOWS / FOOT)				
SAND & GRAVEL		SILT & CLAY		
RELATIVE DENSITY	BLOWS/FOOT*	CONSISTENCY	BLOWS/FOOT*	STRENGTH** (KSF)
VERY LOOSE	0 - 4	VERY SOFT	0 - 2	0 - 0.25
LOOSE	4 - 10	SOFT	2 - 4	0.25 - 0.5
MEDIUM DENSE	10 - 30	MEDIUM STIFF	4 - 8	0.5-1.0
DENSE	30 - 50	STIFF	8 - 15	1.0 - 2.0
VERY DENSE	OVER 50	VERY STIFF	15 - 30	2.0 - 4.0
		HARD	OVER 30	OVER 4.0

* NUMBER OF BLOWS OF 140 LB HAMMER FALLING 30 INCHES TO DRIVE A 2 INCH O.D. (1-3/8 INCH I.D.) SPLIT-BARREL SAMPLER THE LAST 12 INCHES OF AN 18-INCH DRIVE (ASTM-1586 STANDARD PENETRATION TEST).

** UNDRAINED SHEAR STRENGTH IN KIPS/SQ. FT. AS DETERMINED BY LABORATORY TESTING OR APPROXIMATED BY THE STANDARD PENETRATION TEST, POCKET PENETROMETER, TORVANE, OR VISUAL OBSERVATION.

CORNERSTONE EARTH GROUP

LEGEND TO SOIL DESCRIPTIONS

Figure Number A-1

HARDNESS

Soft – Reserved for plastic material alone.
Low hardness – Can be gouged deeply or carved easily with a knife blade.
Moderately hard – Can be readily scratched by a knife blade: scratch leaves a heavy trace of dust and is readily visible after the powder has been blown away.
Hard – Can be scratched with difficulty: scratch produces little powder and is often faintly visible.
Very hard – Cannot be scratched with knife blade: leaves a metallic streak.

STRENGTH

Plastic or very low strength.
Friable – Crumbles easily by rubbing with fingers.
Weak – An unfractured specimen of such material will crumble under light hammer blows.
Moderately strong – Specimen will withstand a few heavy hammer blows before breaking.
Strong – Specimen will withstand a few heavy ringing blows and will yield with difficulty only dust and small flying fragments.
Very strong – Specimen will resist heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.

WEATHERING – The physical and chemical disintegration and decomposition of rocks and minerals by natural processes such as oxidation, reduction, hydration, solution, carbonation and freezing and thawing.

Deep – Moderate to complete mineral decomposition extensive disintegration: deep and thorough discoloration: many fractures, all extensively coated or filled with oxides, carbonates and/or clay or silt.
Moderate – Slight change or partial decomposition of minerals: little disintegration: cementation little to unaffected. Moderate to occasionally intense discoloration. Moderately coated fractures.
Little – No megascopic decomposition of minerals little or no effect on normal cementation. Slight and intermittent, or localized discoloration. Few stains or fracture surfaces.
Fresh – Unaffected by weathering agents. No disintegration or discoloration. Fractures usually less numerous than joints.

FRACTURING

Intensity	Size of Pieces in Feet
Very little fractured	Greater than 4.0
Occasionally fractured	1.0 to 4.0
Moderately fractured	0.5 to 1.0
Closely fractured	0.1 to 0.5
Intensely fractured	0.05 to 0.1
Crushed	Less than 0.05

BEDDING OF SEDIMENTARY ROCKS

Splitting Property	Thickness	Stratification
Massive	Greater than 4.0 feet	very thick-bedded
Blocky	2.0 to 4.0 feet	thick-bedded
Slabby	0.2 to 2.0 feet	thin-bedded
Flaggy	0.05 to 0.2 feet	very thin-bedded
Shaly or Platy	0.01 to 0.05 feet	laminated
Papery	less than 0.01 feet	thinly laminated

BORING NUMBER EB-1

CORNERSTONE EARTH GROUP

PROJECT NAME 10040 Broadway Terrace
PROJECT NUMBER 1300-1-1
PROJECT LOCATION Oakland, CA

DATE STARTED 7/16/21 **DATE COMPLETED** 7/16/21
GROUND ELEVATION **BORING DEPTH** 6.3 ft.
DRILLING CONTRACTOR Exploration Geoservices Inc.
LATITUDE 37.840919° **LONGITUDE** -122.215729°
DRILLING METHOD Minuteman, 4 inch Solid Flight Auger
GROUND WATER LEVELS:
LOGGED BY CRS
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

This log is a part of a report by Cornerstone Earth Group, 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.

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
	0.0	**Sandy Lean Clay (CL)** hard, moist, gray and brown mottled, fine sand, low plasticity							
			63	MC-1C	110	18			>4.5 (○)
	2.5		50 5"	MC-2B	98	18			>4.5 (○)
		Unnamed glauconite mudstone [Tsm] low hardness to moderately hard, weak, deep weathering, brown and gray with iron oxide mottles, low to moderate plasticity							
			51	SPT-3		8			
	5.0		50 3"	SPT					
			94 6"	SPT-5		13			
			50 6"	SPT					
		Bottom of Boring at 6.3 feet.							
	7.5								
	10.0								
	12.5								

UNDRAINED SHEAR STRENGTH, ksf: ○ HAND PENETROMETER; △ TORVANE; ● UNCONFINED COMPRESSION; ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL; scale 1.0, 2.0, 3.0, 4.0

BORING NUMBER EB-2

CORNERSTONE EARTH GROUP

PROJECT NAME: 10040 Broadway Terrace
PROJECT NUMBER: 1300-1-1
PROJECT LOCATION: Oakland, CA

DATE STARTED: 7/16/21 DATE COMPLETED: 7/16/21
GROUND ELEVATION: BORING DEPTH: 8.5 ft.
DRILLING CONTRACTOR: Exploration Geoservices Inc.
LATITUDE: 37.840930° LONGITUDE: -122.215844°
DRILLING METHOD: Minuteman, 4 inch Solid Flight Auger
GROUND WATER LEVELS:
LOGGED BY: CRS
AT TIME OF DRILLING: Not Encountered
NOTES:
AT END OF DRILLING: Not Encountered

This log is a part of a report by Cornerstone Earth Group, 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.

DESCRIPTION

DEPTH (ft)	DESCRIPTION
0.0	**Sandy Lean Clay (CL)** very stiff, moist, dark brown, fine sand, low plasticity Liquid Limit = 33, Plastic Limit = 19
2.3	**Sandy Lean Clay (CL)** hard, moist, gray and brown mottled, fine sand, low plasticity
6.5	**Unnamed glauconite mudstone [Tsm]** low hardness to moderately hard, weak, deep weathering, brown and gray with iron oxide mottles, low to moderate plasticity
8.5	Bottom of Boring at 8.5 feet.

N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
	MC-1B	8	15	14		○ (hand penetrometer, ~3.8)
	MC-2B	03	19			>4.5 ○
50 / 6"	MC-3B	103	20			>4.5 ○
50 / 6"	SPT-4B		16			
50 / 6"	SPT-5		20			

UNDRAINED SHEAR STRENGTH, ksf: ○ HAND PENETROMETER; △ TORVANE; ● UNCONFINED COMPRESSION; ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL

APPENDIX B: LABORATORY TEST PROGRAM

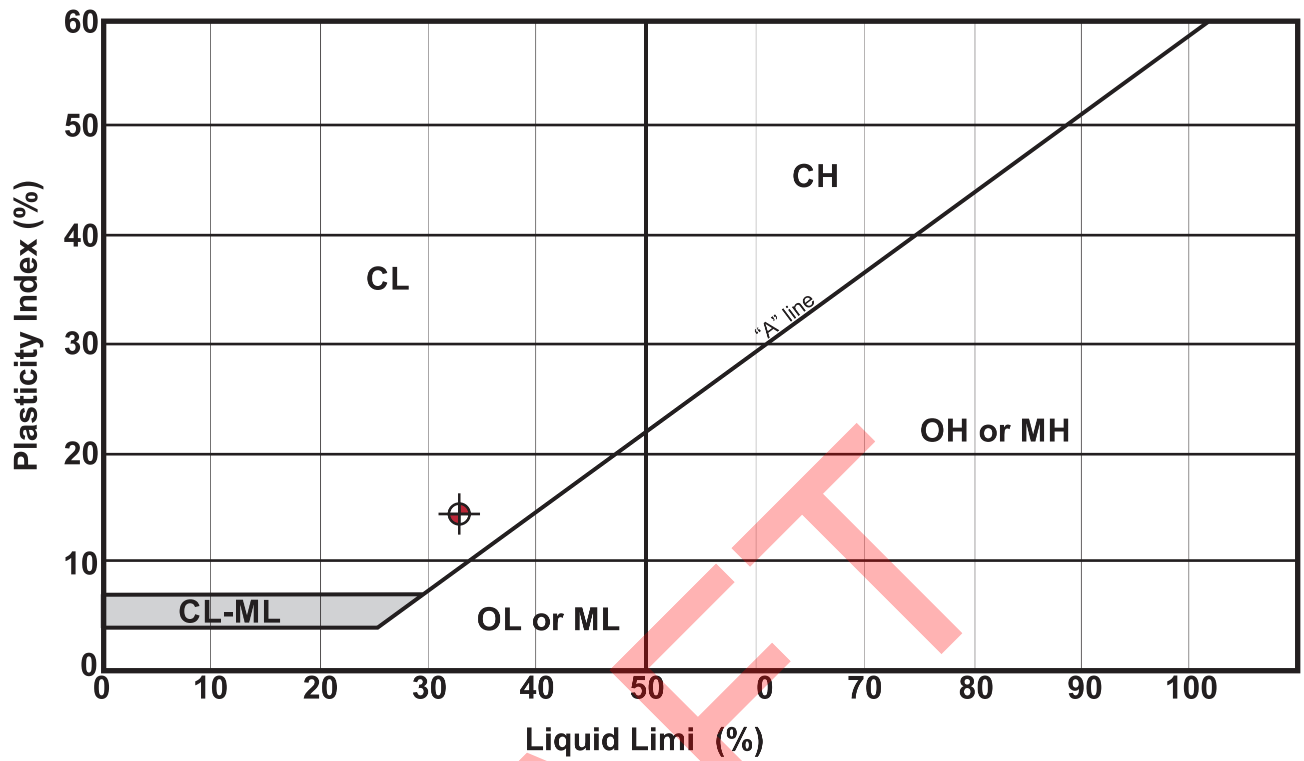
The laboratory testing program was performed to evaluate the physical and mechanical properties of the soils retrieved from the site to aid in verifying soil classification.

Moisture Content: The natural water content was determined (ASTM D2216) on nine samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

Dry Densities: In place dry density determinations (ASTM D2937) were performed on five samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

Plasticity Index: One Plasticity Index determination (ASTM D4318) was performed on a sample of the subsurface soil to measure the range of water contents over which this material exhibits plasticity. The Plasticity Index was used to classify the soil in accordance with the Unified Soil Classification System and to evaluate the soil expansion potential. Results of this test are shown on the boring log at the appropriate sample depth.

Plasticity Index (ASTM D4318) Testing Summary



Symbol	Boring No.	Depth (ft)	Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index	Passing No. 200 (%)	Group Name (USCS - ASTM D2487)
⌖	EB-2	2.0	5	33	19	14	—	Sandy Lean Clay (CL)

Samples prepared in accordance with ASTM D421

CORNERSTONE EARTH GROUP	Plasticity Index Testing Summary 1400Broadway Terrace Oakland, CA	Project Number: 1300-1-1 Figure Number: Figure B1 Date: August 2021 Drawn By: FLL