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GEOTECHNICAL INVESTIGATION REPORT
FOUNDATIONS REPAIR FOR A SINGLE-FAMILY HOME
3120 ELVIDO DRIVE, LOS ANGELES, CA 90049
(APN 4490-004-044)

Prepared for:

RAMON YERA

3120 Elvido Drive
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ZS Engineering #180902

November 19, 2018

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Ramon Yera
3120 Elvido Drive
Los Angeles, CA 90049

Subject: **Geotechnical Investigation Report**
Foundations Repair for a Single-Family Home
3120 Elvido Drive, Los Angeles, CA 90049
(APN 4490-004-044)

Dear Mr. Yera:

In accordance with our proposal, dated September 14, 2018, which was authorized by you, ZS Engineering has prepared this geotechnical report for foundations and slab repair at a single-family residential lot, located at 3120 Elvido Drive in the City of Los Angeles, California. Purposes of this report were to evaluate the subsurface geotechnical condition of the site; assess geologic, seismic hazards at the site; and provide geotechnical design parameters, grading recommendations for foundations and interior slab repair within the problem area in the rear portion of the building.

This lot is within Bel-Air Skycrest community, an upscale residential neighborhood on the south side of Mullholland Drive, on the west side of the 405 Freeway, within the northwestern portion of the City of Los Angeles. Topography within the tract around this lot comprises of gently sloping grades.

This lot is an irregular shaped parcel of about 0.27 acres gross lot area. Currently, it is occupied by a one-story single-family home with an attached 2-car garage. The building contains 4 bedroom, 4.5 bathroom over a total floor area about 3,800 sq. feet. The building sits on a flat pad at an elevation about 6 to 9 feet above the pavement of Elvido Drive. The front lawn leading to the entry door is an upsloping grade from the sidewalk of Elvido Drive, inclined at about 3:1

(horizontal:vertical). Site work in the rear yard include a pool, patio flatwork, a BBQ stand, and a wooden deck. There. In the rear yard (northeast side), flat building pad is followed by a downsloping grade, inclined at about 1.67:1 (horizontal:vertical), which descends to the neighboring lot (16366 Sloan Drive) at an elevation about 15 feet below the subject building pad. This property was developed in 1967 and later, renovated in 1990.

The subject building structure has been undergoing various structural distresses over the recent years, gradually worsening over time. During the site visit, numerous distresses were noticed both at the exterior and within the interior of building structure. These include: cracks on the walls, ceiling; misaligned door and window frames; settlement of the fireplace wall footing; differential settlement across the floor.

Besides the structure, various distresses were also noticed within the exterior flatwork around the building such as several long cracks across the rear yard patio slab, separation of the side yard walkway (north side) from the building wall; vertical drop and separation of the rear yard patio from the side yard walkway at the northeast corner of the building; separation of the patio slab from the rear building wall along the northeast side, etc.

The site is underlain by man-made fill soils to depths varying from about 3 to 5 feet below grade, which are followed by late Miocene age bedrock of Monterey Formation (Tm). Fill soils are silty clay with rock fragments, little amount of fine to medium grained sand. Bedrocks as exposed in the test pits, consist of shaly siltstone with fine grained sandstone interbeds, thin bedded and moderately hard. The bedrock structure strikes northeast and dips generally toward northwest at 20 degree angle.

Based on our geotechnical investigation findings, we conclude that the problem foundations of the existing building shall need to be stabilized with new wall footings and piers that will be embedded into competent bedrock. Exterior (along building perimeter) and interior piers shall be structurally connected by girders underneath the floor frame. Problem flatwork sections (cracks wider than 1/4 inch) shall be saw cut, removed and replaced with new flatwork sections

Existing perimeter footings shall be underpinned by new wall footing sections that will extend minimum 8 inches below the existing footings. New wall footings shall be minimum 18 inches wide and be embedded minimum 18 inches below the exterior soil subgrade (excluding landscaping topsoils). New footing section shall be structurally connected to the existing perimeter footings by dowel rebars.

Pier footings shall be minimum 24 inches in diameter. Embedment for exterior (perimeter) piers shall be minimum 10 feet below the new wall footings or 5 feet into bedrock, whichever is deeper. Embedment for interior piers shall be minimum 8 feet below the soil grade of the crawl space or 5 feet into bedrock, whichever is deeper. Spacing (center to center) between two adjacent piers shall be minimum 8 times the pier diameter.

Design parameters, grading recommendations for the proposed foundations and slab repair are outlined in this report. Based on our findings, there are no geotechnical constraints at the subject site that would adversely impact design and construction of this foundations, flatwork repair project.

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

Respectfully submitted,
ZS ENGINEERING

Zafar Ahmed, PE, GE
Geotechnical Engineer

Fred Aflakian, CEG
Engineering Geologist

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1.0 INTRODUCTION

1.1 Purpose and Scope

This report presents the findings, conclusions and recommendations from geotechnical investigation at the subject single-family residential lot, located at 3120 Elvido Drive in the City of Los Angeles, California. Purposes of this report were to evaluate the subsurface geologic profiles, geotechnical properties; to assess geologic, seismic hazards at the site; and to provide geotechnical design parameters, grading recommendations for repair of building foundations and exterior flatwork. In preparation of this report, we perform the following scope of work:

- Review of the existing geotechnical reports of the neighboring properties; published and unpublished reports and maps pertinent to seismic hazards, local and regional geology for the general area around the site.
- Conduct subsurface exploration consisting of two (2) test pits, excavated and drilled to depths varying from 5 to 7 feet below the grade in the front and rear yards, respectively. Subsurface geologic profiles were logged during field exploration and representative soil samples (bulk and ring) were taken at selected depth intervals.
- Conduct necessary laboratory tests of collected samples in order to characterize the subsurface soils and to obtain necessary geotechnical design parameters.
- Conduct geotechnical evaluations and engineering analyses of the collected data and the laboratory test results. Recommendations for earthwork, design parameters for foundation and slab repair, seismic parameters, other relevant geotechnical parameters and construction guidelines for this project, as presented in this report, are based on the engineering evaluations and analyses.
- Preparation of this report summarizing our findings, conclusions, and recommendations.

1.2 Site and Project Descriptions

This lot is within Bel-Air Skycrest community, an upscale residential neighborhood on the south side of Mullholland Drive, on the west side of the 405 Freeway, within the northwestern portion of the City of Los Angeles. Site location and its vicinities are shown in the attached Figure 1, *Site Location Map*. Topography within the tract around this lot comprises of gently sloping grades.

This lot is an irregular shaped parcel of about 0.27 acres gross lot area. Currently, it is occupied by a one-story single-family home with an attached 2-car garage. The building contains 4 bedrooms, 4.5 bathroom over a total floor area about 3,800 sq. feet. The building sits on a flat pad at an elevation about 6 to 9 feet above the pavement of Elvido Drive. The front lawn leading to the entry door is an upsloping grade from the sidewalk of Elvido Drive, inclined at about 3:1 (horizontal:vertical). Site work in the rear yard include a pool, patio flatwork, a BBQ stand, and a wooden deck. There. In the rear yard (northeast side), flat building pad is followed by a downsloping grade, inclined at about 1.67:1 (horizontal:vertical), which descends to the neighboring lot (16366 Sloan Drive) at an elevation about 15 feet below the subject building pad. Elvido Drive in front of the property is at an average 8 percent up gradient from the northwest to southeast direction. This property was developed in 1967 and later, renovated in 1990.

The subject building structure has been undergoing various structural distresses over the recent years, gradually worsening over time. During the site visit, numerous distresses were noticed both at the exterior and within the interior of building structure. These include: cracks on the walls, ceiling; misaligned door and window frames; settlement of the fireplace wall footing; differential settlement across the floor.

Besides the structure, various distresses were also noticed within the exterior flatwork around the building such as several long cracks across the rear yard patio slab, separation of the side yard walkway (north side) from the building wall; vertical drop and separation of the rear yard patio from the side yard walkway at the northeast corner of the building; separation of the patio slab from the rear building wall along the northeast side, etc. Photos of various distresses of the building structure and exterior flatwork are attached hereafter (see Photos 1 to 12).

Existing building foundations shall need to be stabilized with new foundations - perimeter grade beam and pier footings embedded into bedrock. Problem flatwork sections shall need to be removed and replaced with new flatwork sections. Grading recommendations and design parameters for foundations and flatwork repair are discussed in this report.

1.3 Review of Geotechnical Reports

We reviewed geotechnical reports, response letters to the city's review comments for three (3) of the neighboring properties - 3116 Elvido Drive, 3114 Elvido Drive and 16366 Sloan Drive - located on the adjacent east, east, and the adjacent northeast (rear yard) sides of the subject lot, respectively. These reports and response letters; prepared by AGI Geotechnical, Sassan Geosciences and Vineyard Engineering; are listed in the References. The above neighboring lots are identified in Figure 2, *Site Topography, Exploration & Geology Map*. Based on our review, geologic profiles and geotechnical properties of the subsurface geologic units at the subject lot are found to be fairly consistent with the findings at the adjacent neighboring lots.

1.4 Field Exploration

On September 18, 2018, we conducted subsurface exploration consisting of two (2) test pits, TP-1 and TP-2, excavated to depths about 7 feet and 5 feet below grade at the front and the rear yards, respectively. Approximate locations of the pits are shown on Figure 2, *Site Topography, Exploration & Geology Map*.

Test pits were excavated by manual labor utilizing a power shovel and hand tools. These pits were cut with vertical faces, opening about 3 feet by 3 feet, up to a depth about 3.5 feet. Beyond this depth, drilling to the final depth was utilizing a 6-inch diameter hand auger.

During exploration, subsurface geologic profiles, as encountered within the pits and drilled hole, were logged and representative bulk, ring soil samples were obtained at selected depth intervals. Ring samples were obtained using a drive sampler, 2½-inch inside diameter and 3-inch outside diameter, that holds six (6) modified California rings - each ring ~2.44 inches in inside diameter and 1 inch in height. The sampler was driven into the subsurface soils at target depths by successive drops of a slide hammer that was attached to the drill rods of the hand auger. Slide hammer weighed approximately 20 pounds, dropped over an average height of 18 inches.

Geologic profiles were described in general conformance with the Unified Soil Classification System (USCS) in the ASTM Standard D2487. Logging and sampling were conducted by an engineer and a geologist from our firm. Soil descriptions were entered in the field exploration logs (see Appendix A). Collected soil samples were transported to the laboratory for further evaluation and tests. After logging and sampling, the pits and drilled holes were backfilled with the excavated soil spoils.

1.5 Laboratory Tests

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

- Field moisture and density (ASTM D2216 and ASTM D7263);
- Expansion Index (ASTM D4829);
- Direct Shear (ASTM D3080); and
- Sulfate and chloride contents (California Test Methods 417 and 422).

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

2.0 GEOLOGIC AND GEOTECHNICAL FINDINGS

2.1 Regional Geology

The site is located in the eastern Santa Monica Mountains in the Transverse Ranges Geomorphic Province of California. The Transverse Ranges are an east-west trending series of steep mountain ranges and valleys resulting from north-south tectonic compression and extends from the San Bernardino Mountains in the east to the offshore Channel Islands to the west. The Regional Geology Map for the site shows underlying bedrock as the upper Miocene Modelo Formation. The formation is described as: a silty shale or soft earthy siltstone and interbedded fine- to coarse-grained lithic or arkosic greywacke. Additionally, some of the prominent ridges were mapped as a massive, fine- to coarse-grained sandstone sequences. Location of the site with respect to regional geologic features is shown on Figure 3, *Regional Geology Map*.

2.2 Subsurface Geologic Profile

The site is underlain by man-made fill soils to depths varying from about 3 to 5 feet below grade, which are followed by late Miocene age bedrock of Monterey Formation (Tm). Fill soils consist of light grayish brown color silty clay with rock fragments, little amount of fine to medium grained sand.

Bedrocks as exposed in the test pits, consist of shaly siltstone with fine grained sandstone interbeds, thinly bedded and moderately hard. Shale units are found to be dark grayish brown with orange stains along bedding, while the sandstone units are found to be light brown. The bedrock structure strikes northeast and dips generally toward northwest at about 20 degree angle.

A geologic cross section A-A’, presented in Figure 4, has been drawn across the project site (see Figure 2) and tied to the exploratory test pits in order to illustrate the subsurface geologic profile across the site. Descriptions of subsurface geologic profiles are presented in the field exploration logs (Appendix A). Important characteristics of the subsurface soils, bedrocks that are relevant for this project are discussed briefly below:

2.2.1 Field Moisture and Density

Upper silty, clayey fill soils are found to be medium stiff to stiff and moist.

Underlying bedrock units are found to be moderately hard - gradually stiffer with depth. Field densities of fill soils and bedrock vary from 76.1 to 91.2 pcf within upper 7 feet. Field moisture of the fill soils and shale unit of bedrock within upper 7 feet vary from 21.3 to 24 percent. Field moisture of sandstone units at a depth 3.5 feet below grade in the rear yard is found to be 6.9 percent.

2.2.2 Expansion Potential

Fill soils over bedrock at shallow depths (upper about 3 to 5 feet) are silty clay mixed with little fine to medium sand. Laboratory test results of a representative bulk sample of fill soils indicated low expansion potential (per ASTM D4829) with Expansion Index (EI) value of 45.

2.2.3 Shear Strength Parameters

Conservative estimate of shear strength properties of subsurface fill soils and bedrock units are evaluated from laboratory direct shear tests on ring samples taken from a depth at 4 feet and 7 feet below grade from test pit TP-1. For the bedrock sample, residual value of shear parameters are obtained from shear tests with 4 passes (alternating shear direction after each pass). Laboratory test results of the shear strength parameters - cohesion 345 psf and friction angle 26.7° for fill soils; residual values of cohesion 365 psf and friction angle 27.1° for bedrock - are within the typical range of values for respective geologic units - silty clay fill soils and shaly sitstone of Monterey Formation. These shear parameters will support the design parameters for the proposed shallow grade beam and underlying piers for foundations repair as recommended in this report.

2.2.4 Excavatability

Based on our observation during field exploration, subsurface soils within the anticipated depth of excavation for shallow grade beams are expected to be readily excavatable by conventional earthmoving and trenching equipment in good working condition. Drilling for piers into bedrock are not likely to encounter refusal.

2.2.5 Corrosion Potentials

In general, soil environments that are detrimental to concrete have high concentrations of soluble sulfates and/or pH values of less than 5.5. Section 1904 of the 2017 LABC (Los Angles Building Code) refers to the ACI 318 code for durability requirements of concrete. Section 19.3.2 of ACI318-14 provides guidelines for the concrete mix designs for various exposure levels from soluble sulfate and chloride ions. There are specific requirements on the mix design when the soluble sulfate content of the soil exceeds 0.1 percent by weight or 1,000 parts per million (ppm). As a general practice (e.g., Caltrans guidelines), a threshold limit of chloride ions in the soil environment that may be considered as an external source of chloride to buried concrete is 500 ppm.

Two (2) representative bulk soil samples - one from the upper fill soils and one from the bedrock units - were tested for sulfate and chloride contents. The test results are summarized in Table 1 below and also, presented in Appendix B. These test results indicate that the near surface fill soils have low soluble sulfate and chloride contents (Exposure Classes S0 and C1 per Section 19.3.1 of ACI 318-14). However, bedrocks have moderate amounts of sulfate and low chloride contents (Exposure Classes S1 and C1 per Section 19.3.1 of ACI 318-14). In order to deal with different levels of sulfate exposures, appropriate concrete mix designs for flatwork at surface and buried foundations at depths are discussed in Section 4.7.

Table 1 – Sulfate, Chloride Contents of Geologic Units

Sample Location	Geologic Unit	Sulfate (% by wt.)	Chloride (ppm)
TP-1 @ 0 - 3 ft	Fill soils - silty Clay (CL) w/ little f-m sand	0.0024	12
TP-1 @ 5 - 7 ft.	Bedrock - Shaly siltstone, Monterey Formation (Tm)	0.16	114

2.3 Groundwater

Groundwater was not encountered during our field exploration up to the maximum explored depth of 7 feet below the grade. Historic shallow groundwater at this site and its close vicinities is deeper than 50 feet as documented in the state's seismic hazards report (CGS, 1997). Due to depth, groundwater is not considered as a constraint for design and construction of this foundations repair project.

3.0 FAULTING, SEISMICITY AND SEISMIC HAZARDS

3.1 Faulting and Primary Seismic Hazards

Surface ground rupture along active fault zones and ground shaking represent primary or direct seismic hazards to structures. There are no known active or potentially active faults trending toward or through the site. However, the project site is located in the highly seismic Southern California region within the influence of several faults that are considered to be active or potentially active. An active fault is defined by the State of California as a "sufficiently active and well defined fault" that has exhibited surface displacement within the Holocene time (about the last 11,000 years). A potentially active fault is defined by the State as a fault with a history of movement within Pleistocene time (between 11,000 and 1.6 million years ago).

Nearby known active and potentially active faults for the project site include the following: north terminus of Newport-Inglewood-Rose Canyon Fault Zone (north Los Angeles basin section) at about 7.95 km to the southeast; west terminus of Hollywood fault at about 8.3 km to the southeast; Santa Monica fault at about 9 km to the south; Northridge Hills Fault at about 13 km to the north; Verdugo Fault at about 15.7 km to the northeast; Sierra Madre Fault Zone at about 17.9 km to the north of the site.

With consideration of above proximities to the nearby faults, potential for future surface fault rupture within the project site is considered very low. However, moderate to high ground shaking can be expected at the site sometime during the design life of the subject building structure. Peak ground acceleration at this site is evaluated 0.47g for 10 percent probability in 50 years (475 years return period) based on the Probabilistic Seismic Hazards Assessment Model (CGS, 2008).

3.2 Secondary Seismic Hazards

Secondary seismic hazards for this site, generally associated with severe ground shaking, include liquefaction, seismic settlement, landslide, tsunamis, and seiches. Potentials for these seismic hazards are briefly discussed in the following subsections.

3.2.1 Liquefaction

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

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

The site is not mapped within a liquefaction hazard zone as shown in Figure 5, *Seismic Hazard Zones Map*, which is excerpted from state's seismic hazard zones map (CGS, 1998). Historic shallow groundwater level at this site and its close vicinities is deeper than 50 feet as documented in the state's seismic hazards report (CGS, 1997). Due to deep groundwater, potential for liquefaction does not exist for this site.

3.2.2 Seismic Settlement

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

Seismicity level at this site is relatively high (0.47g Peak Ground Acceleration for 10% probability over 50 years). However, bedrocks at the site are shallow - at depths varying from 3 to 5 feet below the existing grade. Proposed pier footings underneath the building will be embedded minimum 5 feet into bedrock, which will greatly diminish potentials for seismic settlement. With consideration of these factors, structural integrity of the existing building is not likely be adversely impacted if the proposed foundation repair measures as recommended in this report are properly implemented.

3.2.3 Landslide

The site is not mapped within any landslide hazard zone as shown in Figure 5, *Seismic Hazard Zones Map*. The building sits on a flat pad at an elevation about 6 to 9 feet above the pavement of Elvido Drive. The downsloping grade on the rear yard of this lot that descends about 15 feet to a neighboring lot at an inclination about 1.67:1 (horizontal:vertical). During this investigation, this slope was found to be vegetated and stable without any sign of surficial failure, distress, creep, sloughing, and slided mass. With consideration of this topographic setting, the potential for seismically-induced landslides, or debris flows to impact this site is considered very low.

3.2.4 Tsunamis and Seiches

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

Seiches are large waves that are generated in enclosed bodies of water in response to ground shaking. At the present time, no water storage reservoirs are located at higher elevation than this lot within its close vicinities. Therefore, hazards from seiches do not exist for the this site.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 General

Based on our geotechnical investigation findings, we conclude that the problem foundations of the existing building shall need to be stabilized with new wall footings and piers that will be embedded into competent bedrock. Exterior (along building perimeter) and interior piers shall be structurally connected by girders underneath the floor frame. Problem flatwork sections (cracks wider than 1/4 inch) shall be saw cut, removed and replaced with new flatwork sections.

Presented hereafter are our recommendations for site preparation; design parameters for shallow wall footings and deep piers; exterior flatwork repair; seismic parameters, and other relevant geotechnical parameters, construction considerations for this project.

4.2 Earthwork

Earthwork for this project will consist of excavation for shallow wall footings; drilling for the pier footings; and subgrade preparation for exterior flatwork repair. Recommendations for site earthwork are discussed in the following paragraphs.

4.2.1 Site Preparation

Prior to foundations repair around the building perimeter, project area around the perimeter shall be cleared of all weeds, debris, flatwork, and any remnants from previous construction.

For the interior piers, in order to facilitate access of equipment and personnel, sections of raised wood flooring shall be removed at the target locations where new piers will be installed. Prior to installing the interior piers, girders and joists underneath floor frame shall be structurally connected.

Problem flatwork sections (cracks wider than 1/4 inch) shall be neatly saw cut and removed. Any existing utility lines shall be removed and/or rerouted if they interfere with the proposed foundations, slab repair. The cavities resulting from removal of utility lines and any buried obstructions shall be properly backfilled and compacted as recommended in Section 4.2.3 below.

4.2.2 Excavation/Overexcavation

Wall Footings - Excavation for new perimeter wall footings around the building shall be made a neat vertical cut to the design footing bottoms (see Section 4.4) underneath the existing wall footings.

CIDH Piers - Drilling for CIDH piers shall extend to the intended pier tip bottom. No overexcavation below the pier tip will be necessary. Bottom of the drilled hole shall need to be cleaned out of any loose materials, sloughs generated from drilling so that pier tip can be on competent native soils.

Exterior Flatwork - After removal of the problem patio concrete/paver section and underlying base (if any), upper minimum 12 inches of the subgrade soils shall be excavated.

If loose, yielding (pumping) or otherwise unsuitable soils are exposed at the wall footing bottoms and flatwork repair area, problem soils shall be removed until competent bottom soils are reached. Competent removal bottom shall be relatively unyielding (not penetrating more than 2 inches) by hand probing with a cone tipped steel probing rod and shall have minimum dry density of 80 pcf.

4.2.3 Fill Placement and Compaction

For new wall footings and exterior flatwork repair, prior to placement of rebars, soil subgrade underneath the removed flatwork section and footing bottoms shall be scarified, moisture-conditioned (adding water as needed) to minimum 3 percent above the optimum moisture, and recompacted in place to minimum 90 percent (ASTM D1557).

Fill soils shall be placed in thin lifts - loose lift thickness not exceeding 8 inches - moisture-conditioned (adding water as needed) to minimum 3 percent above the optimum moisture, and compacted to minimum 90 percent (ASTM D1557).

Base materials underneath the exterior flatwork and wherever else used shall be placed at minimum 95 percent compaction (ASTM D1557) with placement moisture within 2 percent of the optimum moisture.

During preparation of wall footing bottoms and subgrade for exterior flatwork repair, field density tests shall be taken at the following schedule:

- Minimum one (1) field test for each 50 linear feet of footing excavation for wall footings.

- Minimum one (1) field test for 100 sq. feet area for each one (1) foot lift of fill and at the final soil subgrade for flatwork repair.

- Minimum one (1) field test for each 50 linear feet of trench backfill (for any relocated utility) for each one (1) foot lift of fill and at the final grade.

Field density tests may be taken by utilizing the sand cone (ASTM D1556) method or a combination of both the nuclear (ASTM D6938) and the sand cone methods. In compliance with the LADBS Bulletin P/BC 2017-028, if nuclear method is used at least 1 sand cone test shall be taken for every 10 nuclear method tests. The sand cone test shall be taken at the same general location and elevation as 1 out of the 10 nuclear tests so it can be correlated with the nuclear tests.

4.2.4 Trench Backfill

Any relocated utility trenches shall be backfilled in accordance with Sections 306-12 of the *Standard Specifications for Public Works Construction,* ("Greenbook"), 2018 Edition.

Utility trenches can be backfilled with onsite or import soils that meet the fill soils criteria in Section 4.2.5. Prior to backfilling the trenches, pipes shall be bedded in and covered with granular material that has a minimum Sand Equivalent (SE) value of 40 (ASTM D2419). Due to high fine contents in the onsite soils, bedding sands need to be imported. Bedding sands shall be placed by mechanical compaction; jetting is not recommended. Soil backfill over the pipe bedding zone shall be placed in thin lifts, moisture conditioned (adding water as needed) to minimum 3 percent above the optimum moisture, and mechanically compacted to minimum 90 percent (ASTM D1557).

Wherever mechanical compaction as recommended above is not practical due to space limitations or shallow trench depth, alternative backfill method such as placement of pea gravel (size up to 1/2") or sand-cement slurry (minimum 2 sacks of cement for 1 cubic yard mix in compliance with LADBS Bulletin P/BC 2014-121) may be considered for backfill of utility trenches.

4.2.5 Fill Materials

Onsite soils that are free of organics, debris and oversize particles (larger than 3 inches in the largest dimension) are suitable for use as fill. Import soils, if used, shall be free of organics, corrosion impacts; and shall have low expansion potential (Expansion Index less than 50 per ASTM D4829).

Base materials underneath the new flatwork and wherever else used may consist of crushed miscellaneous base in conformance with 200-2.4 of the *Standard Specifications for Public Works Construction,* (“Greenbook”), 2018 Edition.

Prior to any import, geotechnical consultant shall evaluate the materials submittals and conduct necessary tests in order to confirm the quality of the materials.

4.2.6 Temporary Excavation and Underpinning

Temporary excavations during construction, away from the influence zone of the existing foundations (1:1 projection downward and outward from the footing bottoms), may be constructed according to the slope ratios presented in Table 2 in the following.

Table 2 – Slope Ratio for Temporary Excavation

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

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

In order to excavate for new wall footings around the building, existing perimeter walls will need to be undermined, which will require underpinning the walls. Contractor shall implement appropriate methods and means of underpinning without impacting integrity of the existing building structure.

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

4.3 Seismic Design Parameters

In the absence of data from deep soil borings, this site's subsurface soil profile may be characterized within the category of Site Class D according to Chapter 20 of ASCE/SEI 7-10 as referred in Section 1613.3.2 of the 2017 LABC. Based on the nature of occupancy, proposed residential structure addition will fall into Risk Category II (per Table 1604.5 of the 2017 LABC). Corresponding seismic design parameters for this soil profile, site location (Latitude: 34.1291°N; Longitude: 118.4898°W at the center of the building footprint) and Risk Category are determined in accordance with Section 1613.3 of the 2017 LABC. These parameters, as presented in Table 3 below, are derived from risk-targeted Maximum Considered Earthquake (MCE_R) based spectral response analysis. Proposed foundations repair at the subject residential structure and any structural improvements at this site shall be designed for these seismic parameters.

Table 3 – Seismic Design Parameters

Categorization/Seismic Parameters	Design Value
Site Class	D
Mapped MCE Spectral Acceleration for Short (0.2 Second) Period, S_S	2.140g
Mapped MCE Spectral Acceleration for a 1-Second Period, S_1	0.749g
Short Period (0.2 Second) Site Coefficient, F_a	1.0
Long Period (1 Second) Site Coefficient, F_v	1.5
Adjusted Spectral Response Acceleration at 0.2-Second Period, S_{MS}	2.140g
Adjusted Spectral Response Acceleration at 1-Second Period, S_{M1}	1.123g
Design (5% damped) Spectral Response Acceleration for Short (0.2 Second) Period, S_{DS}	1.426g
Design (5% damped) Spectral Response Acceleration for a 1-Second Period, S_{D1}	0.749g
Seismic Design Category	D

4.4 Foundations Repair

The subject building structure shall need be stabilized with new wall footings, underpinned by CIDH (Cast-in-drilled-hole) piers along the building perimeter and new CIDH piers within the interior of the building footprint. Exterior and interior piers shall be structurally connected by girders underneath the floor frame. Geotechnical design parameters for new wall footings and pier foundations are described in the following subsections.

4.4.1 Shallow Wall Footings

New perimeter wall foundations around the building perimeter shall be bearing on properly compacted soil subgrade, prepared as recommended in Sections 4.2.2 and 4.2.3 of the report. Geotechnical design parameters for wall footings are described in the following subsections.

Footing Dimensions and Embedments – Existing perimeter footings shall be underpinned by new wall footing sections that will extend minimum 8 inches below the existing footings. New wall footings shall be minimum 18 inches wide and be embedded minimum 18 inches below the exterior soil subgrade (excluding landscaping topsoils). New footing section shall be structurally connected to the existing perimeter footings by dowel rebars.

Vertical Bearing - For footings with the minimum embedment as described above and bearing on properly compacted soil subgrade, an allowable vertical bearing capacity of 1,500 psf may be used for design, which may be increased by 300 psf for each additional foot of foundation embedment up to a maximum value of 2,000 psf. These bearing values may be increased by one-third for short-term loads (e.g., seismic, wind loads).

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

4.4.2 CIDH Pier Footings

Geotechnical design parameters for CIDH pier footings are described in the following subsections. Construction guidelines and specifications for drilling the pier holes and placement of concrete are presented in Appendix C.

Footing Dimension and Embedment - Pier footings shall be minimum 24 inches in diameter. Embedment for exterior (perimeter) piers shall be minimum 10 feet below the new wall footings or 5 feet into bedrock, whichever is deeper. Embedment for interior piers shall be minimum 8 feet below the soil grade of the crawl space or 5 feet into bedrock, whichever is deeper. Spacing (center to center) between two adjacent piers shall be minimum 8 times the pier diameter.

Axial Capacity - Axial capacity of pier foundations will be derived end bearing. Allowable end bearing at the pier tip may be considered 6,000 psf for minimum 5 feet of pier embedment into bedrock. This bearing may be increased by 400 psf for each additional foot of embedment into bedrock, up to a maximum bearing value of 7,500 psf. One-third increase for end bearing is allowed for short-term loads (e.g., seismic, wind loads).

Lateral Capacity - Lateral loads may be resisted by soil’s passive resistance and friction between the pile tip and the supporting subgrade. Frictional resistance coefficient of 0.35 may be used at the pier tip. Subgrade soil’s passive resistance may be considered 200 psf per foot of pile embedment up to a maximum value of 1,500 psf. Tributary area for soil's passive resistance for an individual pier may be considered a vertical plane along pier embedment with width 2 times the pile diameter (concrete cross-section). For interior pier footings, upper 12 inches of the embedment below the top of the grade shall be ignored in calculations for passive resistance.

The above friction coefficient and passive resistance values have already been reduced by a factor of safety 1.5. One-third increase of soil's passive resistance is allowed for short-term seismic or wind loads.

4.5 Foundation Setback from Slope

Existing building sits on a flat pad, which is followed by a downsloping grade, inclined at about 1.67:1 (horizontal:vertical), that descends to the neighboring lot (16366 Sloan Drive) at an elevation about 15 feet below the subject building pad (see Figure 4). The building structure maintains a minimum setback of 27 feet to the face of downslope in the rear yard. At the front, the building pad is at an elevation about 6 to 9 feet above the pavement of Elvido Drive. The front lawn leading to the entry door is an upsloping grade from the sidewalk of Elvido Drive, inclined at about 3:1 (horizontal:vertical).

Proposed foundations repair with pier footings, emebedded minimum 5 feet into bedrock, as recommended in this report, will maintain adequate setback to the face of downsloping grades in the front and rear yards in compliance with Section 1808.7 of the 2017 LABC.

4.6 Exterior Flatwork Repair

If width of cracks are 1/4 inch or less, these can be repaired with grout seal. For crack openings wider than 1/4 inch, problem flatwork section encompassing the entire length of the crack shall be neatly saw cut and replaced with a new flatwork section. Saw cut section shall be minimum 2 feet wide. New flatwork section shall be connected to the adjacent existing flatwork slab by dowel rebars.

Minimum concrete section, underlying base thickness, concrete strengths, and minimum reinforcements for repair sections of exterior flatwork are presented in Table 4 below.

Table 4 – Repair Section for Exterior Flatwork

Proposed Improvements	Min. Slab Thickness (inch)	Min. Base Thickness (inch)	Min. Concrete Strength (psi)	Minimum Reinforcement [1]
Exterior flatwork repair	5.0	4.0	2,500	#3 rebars @ 12" o/c, both ways

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

Appropriate joints and saw cuts should be provided for all the concrete flatworks in accordance with either Portland Cement Association (PCA) or American Concrete Institute (ACI) guidelines.

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

4.7 Cement Type and Concrete Properties

Laboratory test results indicate that the soluble sulfate and chloride contents of subsurface fill soils at shallow depths (upper 3 to 5 feet) are low (Exposure Classes S0 and C1 per ACI 318-14). However, soluble sulfate contents of the bedrock (shale) units are found to be moderate and chloride contents are found to be low (Exposure Classes S1 and C1 per ACI 318-14).

For exterior flatwork repair, there is no restriction on the type of cement and minimum concrete strength from the durability standpoint. Conventional Type II cement (ASTM C150) may be used for concrete for this project. Minimum 28-day compressive strength (ASTM C39) of structural concrete should be 2,500 psi.

For new perimeter wall footings and pier footings, concrete shall have minimum specified strength, f'_c, of 4,000 psi and maximum water cement ratio of 0.50, as required by ACI 318-14 for Exposure Class S1. Conventional Type II cement (ASTM C150) may be used for concrete for these structural elements; Type V sulfate resistant cement is preferred.

Water-soluble chloride ion content in the concrete (per ASTM C1218) shall not exceed 0.3 percent of the cement content (by weight) for all elements - flatwork and foundations.

4.8 Corrosion Measures for Buried Metal

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

- Below-grade ferrous metals shall be given a high-quality protective coating, such as 20-mil thick plastic tape, extruded polyethylene, coal-tar enamel, or Portland Cement mortar.

- Below-grade ferrous metals shall be electrically insulated (isolated) from above-grade ferrous metals and other dissimilar metals by means of dielectric fittings in utilities and exposed metal structures breaking grade.

- Reinforcements (rebar, wire mesh) within concrete that will be in direct contact with the site soils shall have at least 3 inches of concrete cover.

4.9 Surface Drainage

In compliance with Section 1804.4 of the 2017 LABC, the ground immediately adjacent to the proposed new foundations shall be sloped away from the building at a gradient not less than 5 percent for a minimum distance of 10 feet from face of the building walls. If any physical obstructions or lot lines prohibit 10 feet of horizontal distance, a 5 percent gradient shall be provided to an approved alternative method of diverting water away from the foundation. Swales used for this purpose shall be sloped at minimum 2 percent wherever located within 10 feet of the building foundation. Impervious surfaces (such as concrete flatwork) within 10 feet of the new foundations shall be sloped at minimum 2 percent away from the building walls.

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

4.10 Geotechnical Services during Construction

During foundation, flatwork repair for this project, geotechnical observation and tests shall be performed at the following stages:

- Excavation bottoms observation and test for shallow wall footings;
- Continuous observation during drilling for piers;
- During preparation of soil subgrade for exterior flatwork repair;
- During any trench backfill; and
- Whenever any unusual or unexpected geotechnical conditions are encountered.

4.11 Limitations

Proposed new wall footings and piers, if designed and installed in compliance with this report, will stabilize the subject building structure against further movement and will arrest progression of interior distresses within the building that are associated with foundation movement. However, this foundation stabilization will not automatically fix already occurred distresses inside the building. Various visible distresses inside the building need to be repaired on case by case basis after installation of the proposed wall footings and piers.

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

Geotechnical investigation and relevant engineering evaluations for this project were performed in substantial conformance with the general practices of geotechnical engineering in southern California at the time of this report. No other warranty is expressed or implied.

5.0 REFERENCES

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Vineyard Engineering, Geotechnical Engineering Report Addendum No. 1, Proposed Residence, 16366 Sloan Drive, Los Angeles, AIN 4490-004-031, Tr. 24452, Lot 1N, Los Angeles, California 90049, LADBS Log #96652, Vineyard Eng. Job #216-269, dated August 23, 2017.

Vineyard Engineering, Geotechnical Engineering Report Addendum No. 2, Proposed Residence, 16366 Sloan Drive, Los Angeles, AIN 4490-004-031, Tr. 24452, Lot FR1, Los Angeles, California 90049, LADBS Log #96652, Vineyard Eng. Job #216-269, dated November 1, 2017.

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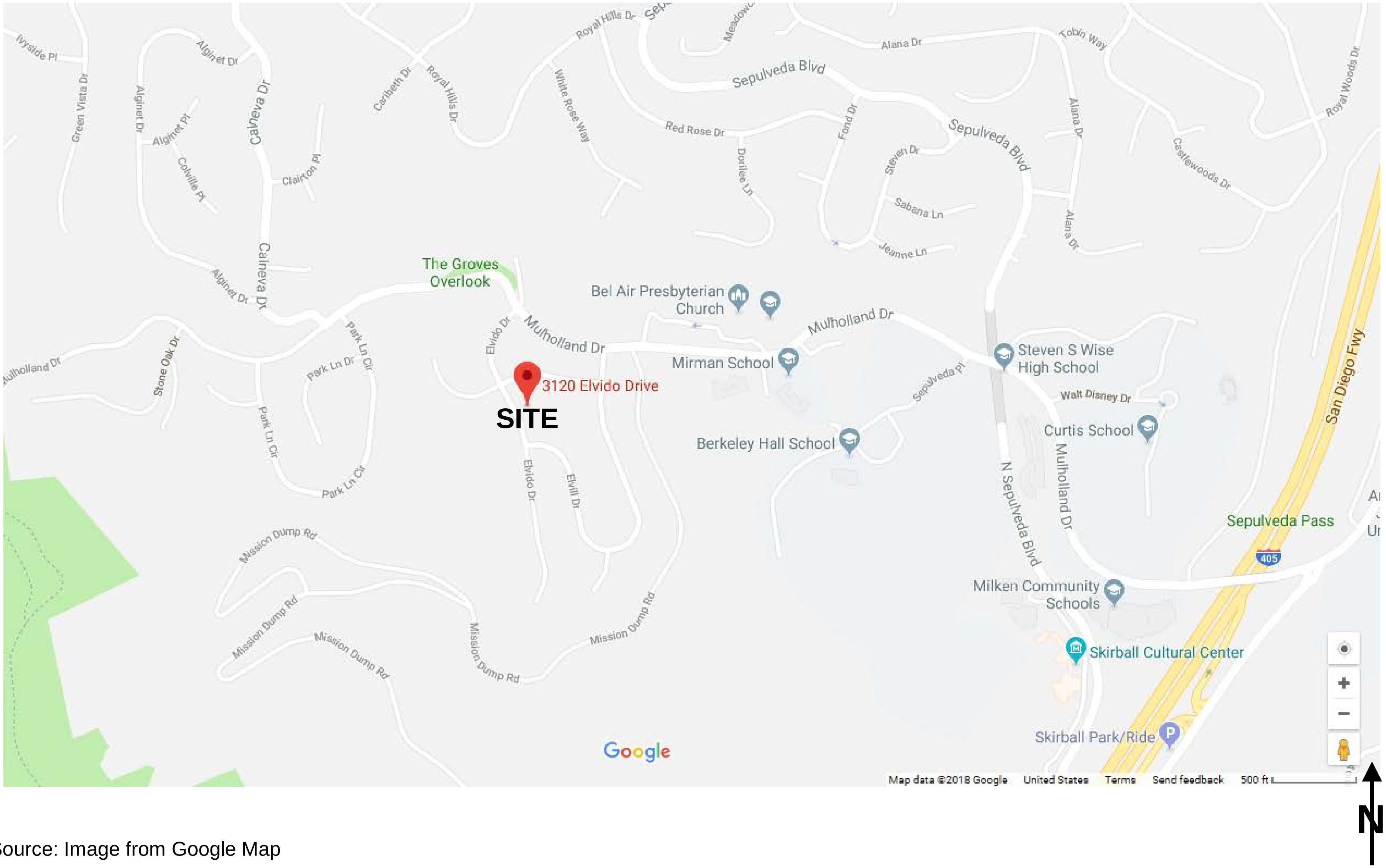


Source: Image from Google Map

	Figure 1 – Site Location Map **Residential Lot at 3120 Elvido Drive, Los Angeles, CA 90049**	**ZS Engineering #180902**

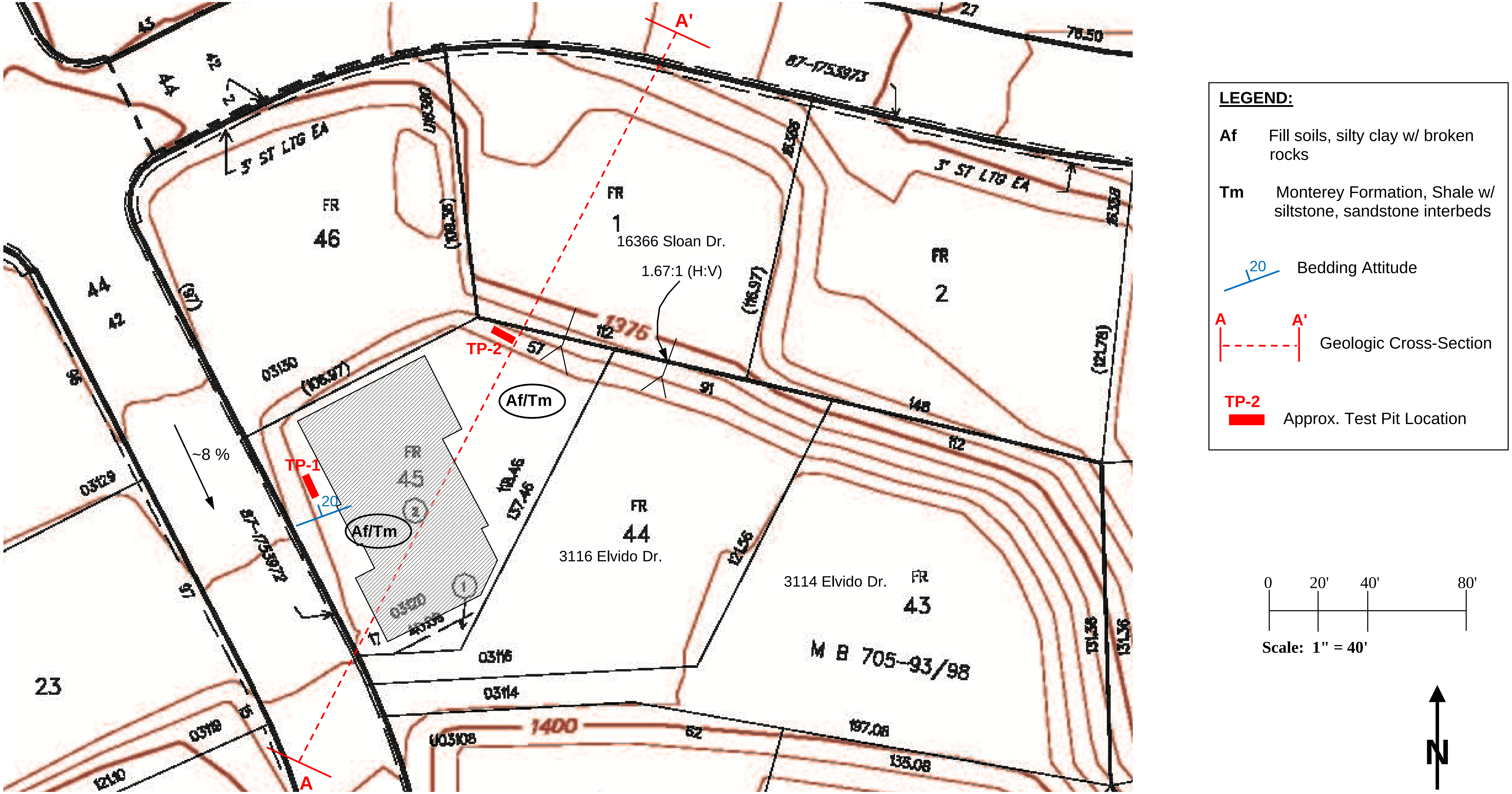
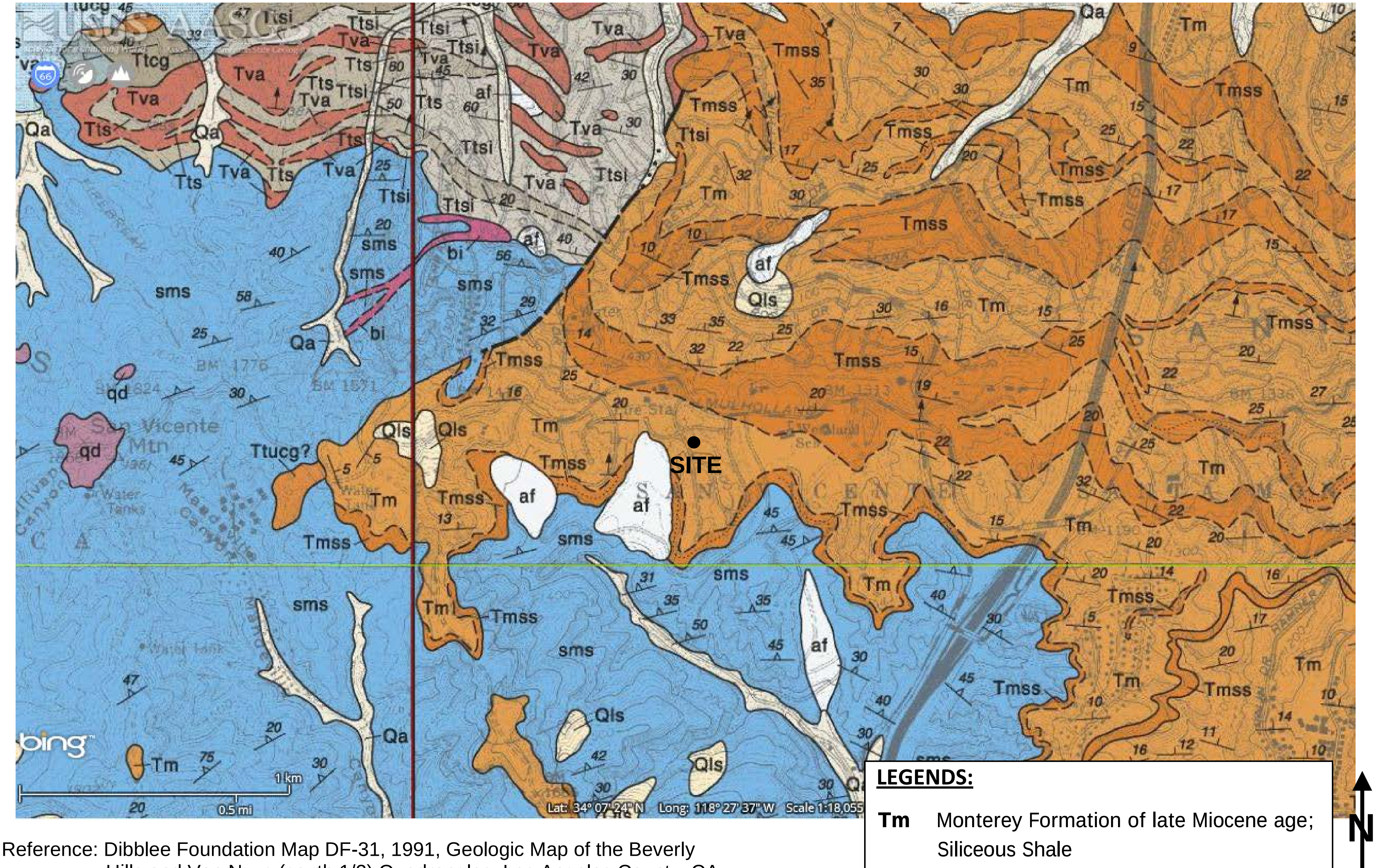


Figure 2 – Site Topography, Exploration & Geology Map

Residential Lot at 3120 Elvido Drive, Los Angeles, CA 90026

ZS Engineering
#180802



Reference: Dibblee Foundation Map DF-31, 1991, Geologic Map of the Beverly Hills and Van Nuys (south 1/2) Quadrangles, Los Angeles County, CA.

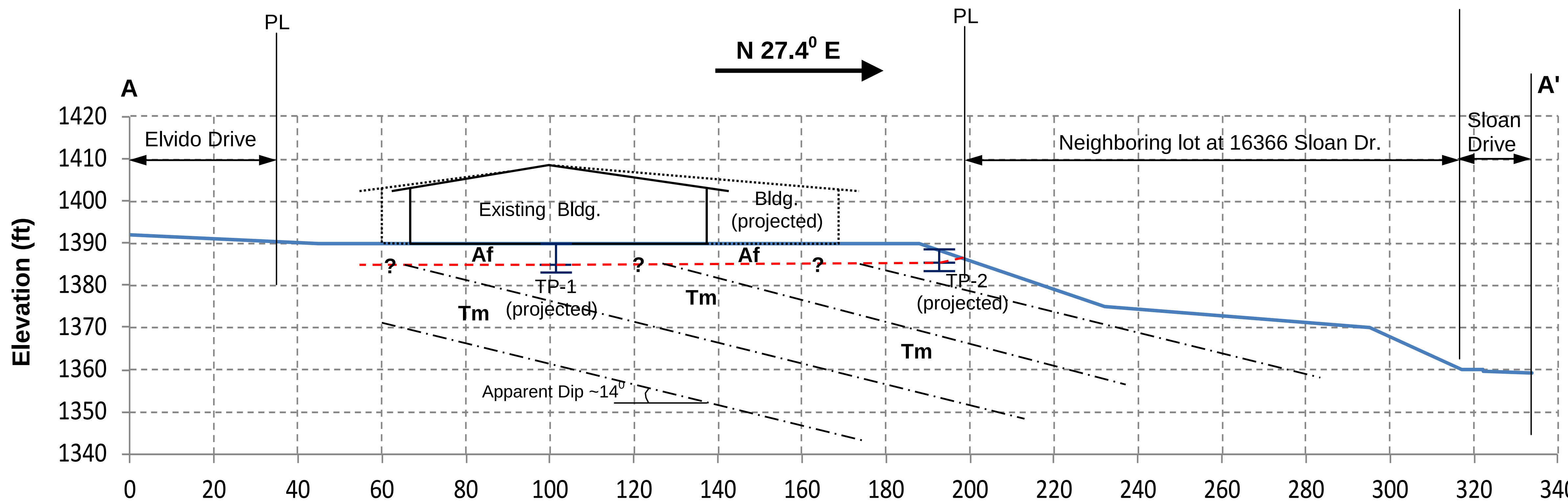
LEGENDS:

Tm Monterey Formation of late Miocene age; Siliceous Shale

Figure 3 – Regional Geology Map

Residential Lot at 3120 Elvido Drive, Los Angeles, CA 90049

ZS Engineering #180902



LEGEND:

Af Fill soils, silty clay w/ broken rocks

Tm Monterey Formation, Shale w/ siltstone, sandstone interbeds

?- - - - - - ? Geologic Contact

0 15' 30' 60'

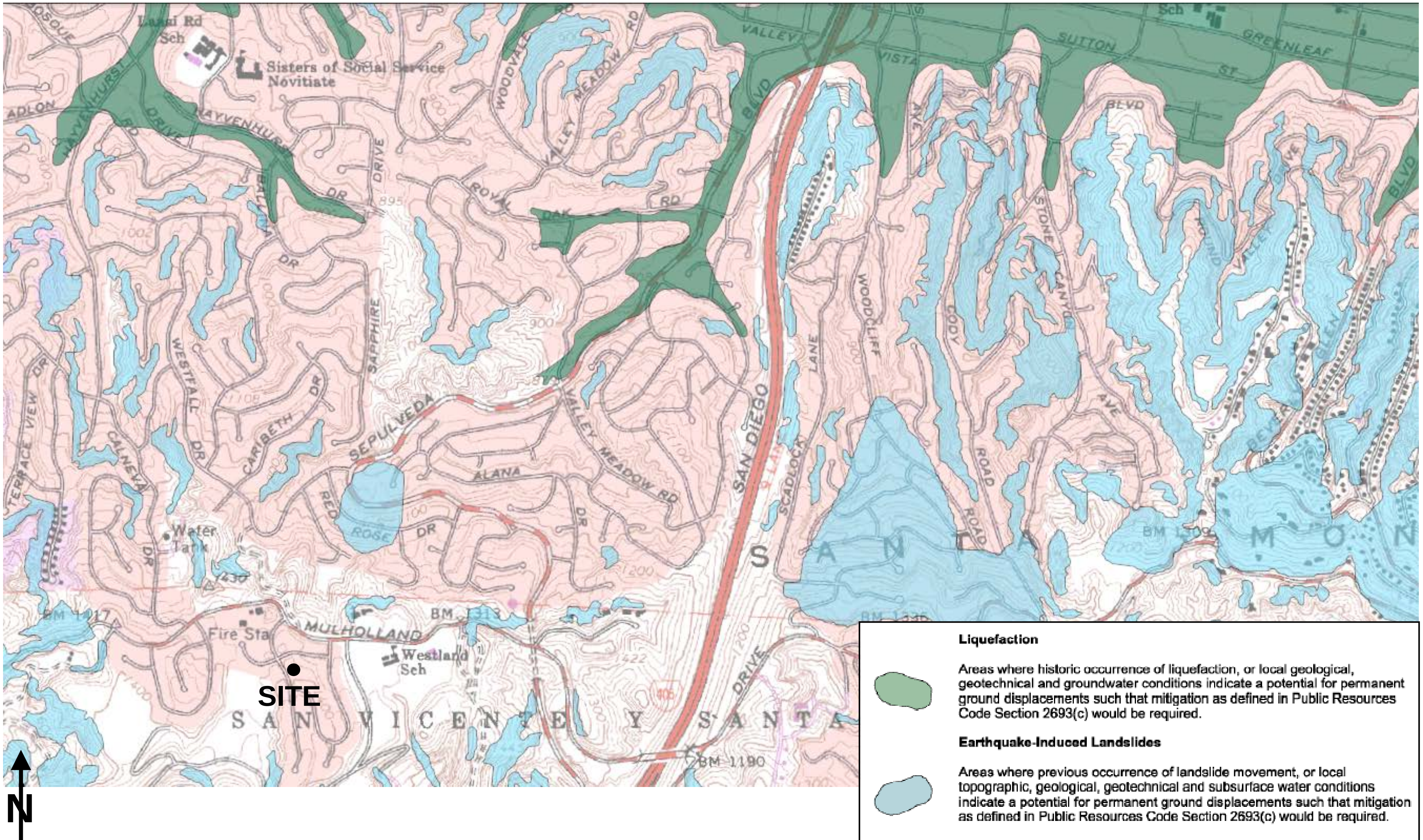
Scale: 1" = 30'

N

Figure 4 – Geologic Cross Section A-A'

Residential Lot at 3120 Elvido Drive, Los Angeles, CA 90026

ZS Engineering #180802



Source: State of California Seismic Hazard Zones Map for Van Nuys Quadrangle, revised on 2/1/1998.

Figure 5 – Seismic Hazard Zones Map

Residential Lot at 3120 Elvido Drive, Los Angeles, CA 90049

ZS Engineering #180902

APPENDIX A

Field Exploration Logs

ZS Engineering

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

DATE: 9/18/18 EXPLORATION METHOD: Power Shovel, Hand Auger (6" Dia.) w/ Drive Sampler

LOGGED BY: ZA/FA ELEVATION: ~1,390 ft. LOCATION: See Fig. 2, Site Plan & Exploration Map

FOUNDATIONS REPAIR AT A RESIDENTIAL LOT
3120 ELVIDO DRIVE, LOS ANGELES, CA 90049

TEST PIT NO. **TP-1**

DEPTH (FEET)	SAMPLE NUMBER	RING SAMPLE	BULK SAMPLE	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	SOIL DESCRIPTIONS	SOIL TESTS
			X			**Fill (Af):** 0 - 5': Surface covered w/ synthetic turf, light grayish	Expansion Index;
2	B-1		X			brown silty Clay (CL) w/ fine to med. sand, broken rocks,	Sulfate, Chloride
			X			medium stiff to stiff, moist.	
4	R-1	X		21.3	78.2		Direct Shear
5							
	B-2		X			**Monterey Formation (Tm):** @ 5': Dark gray shaly siltstone w/ ora	Sulfate, Chloride;
7	R-2	X	X	24.0	76.1	orange stains along bedding, thinly bedded, moderately hard,	Direct Shear
						moist. (B) N 70 E, 20 NW.	

- Depth of excavation ~7 feet below the existing grade.
- Bedrock encountered at 5 ft below grade.
- After logging & sampling, excavated pit was backfilled with the soil cuttings on 9/18/18.

ZS Engineering

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

DATE: 9/18/18 EXPLORATION METHOD: Power Shovel, Hand Auger (6" Dia.) w/ Drive Sampler

LOGGED BY: ZA/FA ELEVATION: ~ 1,388 ft. LOCATION: See Fig. 2, Site Plan & Exploration Map

FOUNDATIONS REPAIR AT A RESIDENTIAL LOT
3120 ELVIDO DRIVE, LOS ANGELES, CA 90049

TEST PIT NO. **TP-2**

DEPTH (FEET)	SAMPLE NUMBER	RING SAMPLE	BULK SAMPLE	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	SOIL DESCRIPTIONS	SOIL TESTS
	B-1		X			**Fill (Af):** 0 - 3': Surface covered w/ dirt, weeds. Light grayish brown silty Clay (CL) w/ fine to med. sand, broken rocks, medium stiff to stiff, moist.	
3							
3.5	R-1	X		6.9	91.2	**Monterey Formation (Tm):** @ 3': Lt. brown Sandstone w/ fine to corase sand grains, massive, moderately hard, slightly moist to moist.	
5							
						- Depth of excavation ~5 feet below the existing grade. - Bedrock encountered at 3 ft below grade. - After logging & sampling, excavated pit was backfilled with the soil cuttings on 9/18/18.	

ZS ENG. #180902 **LOG OF TEST PIT TP-2**

APPENDIX B

Laboratory Test Procedures and Test Results

Laboratory Test Procedures and Test Results

We retained Cal Land Engineering's Laboratory to conduct all the necessary laboratory tests for this geotechnical report. Cal Land Engineering is listed among the City of Los Angeles' certified materials testing laboratories. A certified report of the laboratory test results from Cal Land's Engineering Manager is attached in this appendix. Brief description of the laboratory test procedures and test results are presented hereafter.

Field Moisture and Density: Field moisture contents and dry densities of subsurface soils at selected depths were determined from the ring samples in accordance with the ASTM Test Methods D2216 and D7263, respectively. These test results are presented in the field exploration logs (Appendix A).

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

Sample Location	Soil Description	Expansion Index	Expansion Potential
TP-1 @ 0 - 3 ft	Silty Clay (CL) w/ little f-m sand	45	Low

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

For a conservative estimate of shear strength parameters of the bedrock samples, ultimate shear strengths as described above were obtained from direct shear tests with multiple passes (alternating shear direction after each pass). Shear strength envelope at the 4th pass, which got stabilized after 3 passes, was considered for the design parameters of residual shear strength.

Graphical plots of the shear tests for fill soils and bedrock unit are presented in the *Direct Shear* figures in this appendix.

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

Sample Location	Soil/Bedrock Description	Sulfate (% by wt.)	Chloride (ppm)
TP-1 @ 0 - 3 ft	Silty Clay (CL) w/ little f-m sand	0.0024	12
TP-1 @ 5 - 7 ft.	Shaly siltstone, Monterey Formation (Tm)	0.16	114

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

November 19, 2018
ZS Engineering #180902

Subject: **Acceptance of Laboratory Test Results**
Foundations Repair for a Single-Family Home
3120 Elvido Drive, Los Angeles, CA 90049

To Whom It May Concern

Cal Land Engineering's Laboratory was retained to conduct all the necessary laboratory tests that are documented in this geotechnical investigation report for the proposed foundations repair for the subject single-family home. Cal Land Engineering is listed among the City of Los Angeles' certified materials testing laboratories. A signed and stamped letter from Cal Land Engineering's Engineering Manager is attached hereafter that certifies the enclosed laboratory test results.

The undersigned geotechnical engineer, the author of this report, assumes responsibilities for utilizing all the enclosed laboratory test results, conducted by Cal Land Engineering, for the engineering analyses and evaluations that we performed in preparation of this report. This acceptance letter along with the enclosed signed, stamped letter from Cal Land's Engineering Manager fulfill the requirements of valid laboratory test results as outlined in the LADBS Bulletin P/BC 2017-113.

Respectfully submitted,
ZS ENGINEERING

REGISTERED PROFESSIONAL ENGINEER
ZAFAR AHMED
No. 2720
EXP. 6/30/19
GEOTECHNICAL
STATE OF CALIFORNIA

PROFESSIONAL GEOLOGIST
FRED AFLAKIAN
No. 2051
Certified Engineering Geologist
STATE OF CALIFORNIA

Zafar Ahmed, PE, GE
Geotechnical Engineer

Fred Aflakian, PG, CEG
Engineering Geologist

Cal Land Engineering, Inc.
dba Quartech Consultants
Geotechnical, Environmental, and Civil Engineering

September 24, 2018

ZS Engineering
113 Tomato Springs
Irvine, CA 92618

Attn: Mr. Zafar Ahmed

RE: **LABORATORY TEST RESULTS/REPORT**
Project: Foundations Repair - 3120 Elvido Drive, Los Angeles, CA
QCI Project No.: 18-213-009b

Gentlemen:

We have completed the testing program conducted on samples for the above project. The tests were performed in accordance with testing procedures as follows:

TEST	METHOD
Moisture Content and Density	ASTM D2216, D2937
Expansion Index	ASTM D4829
Direct Shear	ASTM D3080
Sulfate, Chloride	CT-417, CT-422

Summary of the above laboratory test results are attached hereafter.

We appreciate the opportunity to provide testing services to ZS Engineering. Should you have any questions, please call the undersigned.

Respectfully submitted,

Cal Land Engineering, Inc. (CLE)
dba Quartech Consultants (QCI)

REGISTERED PROFESSIONAL ENGINEER
JACK C. LEE
No. 40870
EXP. 3/31/2019
CIVIL
STATE OF CALIFORNIA

Jack C. Lee
Engineering Manager

Enclosure: Lab Test Results

576 East Lambert Road, Brea, California 92821; Tel: 714-671-1050; Fax: 714-671-1090

ZS Engineering
113 Tomato Springs
Irvine, CA 92618

QCI Project No.: 18-213-009b
Date: September 19, 2018
Summarized by: MW

Project Name: Foundations Repair - 3120 Elvido Drive, Los Angeles, CA
ZS Engineering No.: 180902

Summary of Laboratory Testing Data
(Moisture Content & Density, ASTM D2216 & D2937)

Bore Hole No.	Sample Depth (ft)	Dry Density (pcf) ASTM D2937	Moisture Content (%) ASTM D2216
TP-1	4	78.2	21.3
TP-1	7	76.1	24.0
TP-2	3	91.2	6.9

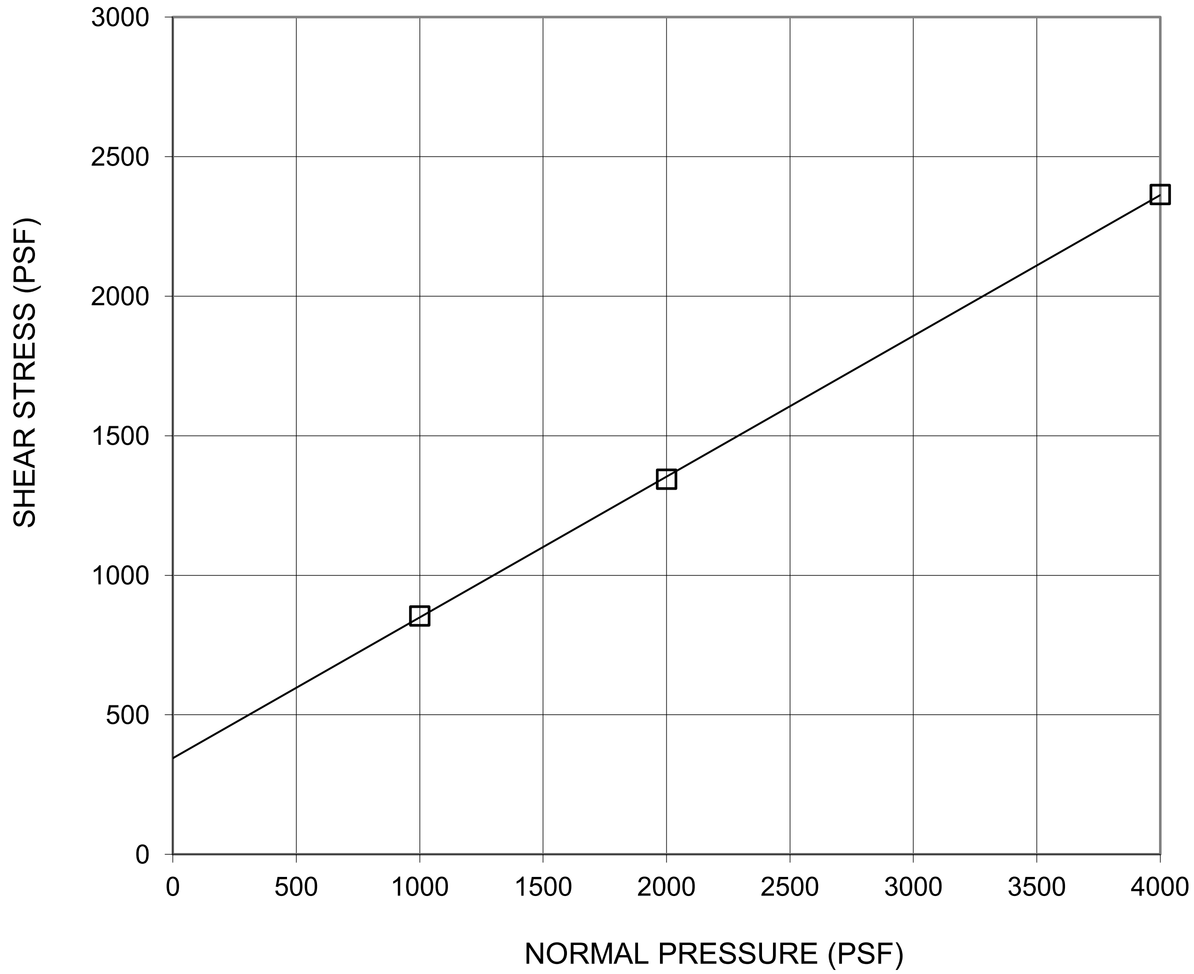
ZS Engineering
113 Tomato Springs
Irvine, CA 92618

QCI Project No.: 18-213-009b
Date: September 21, 2018
Summarized by: MW

Project Name: Foundations Repair - 3120 Elvido Drive, Los Angeles, CA
ZS Engineering No.: 180902

Expansion Index Test Results (ASTM D4829)

Bore Hole No.	Sample Depth (ft)	Visual Classification	Expansion Index (ASTM D4829)	Expansion Potential
TP-1	0 - 3	Silty Clay (CL)	45	Low



Date: September 19, 2018

SHEAR STRENGTH	BORING NO.	DEPTH (FT)	SAMPLE TYPE	SOIL TYPE	COHESION (PSF)	FRICTION ANGLE (DEG)
Ultimate	TP-1	4	Undisturbed Ring	Silty Clay (CL)	345	26.7

Avg. Initial Dry Density = 78.2 pcf

Vertical Loads (PSF)	Moisture Before Test (%)	Moisture After Test (%)
1000	21.8	25.6
2000	21.2	26.4
4000	20.9	25.9

CalLand Engineering, Inc.
Dba Quartech Consultants

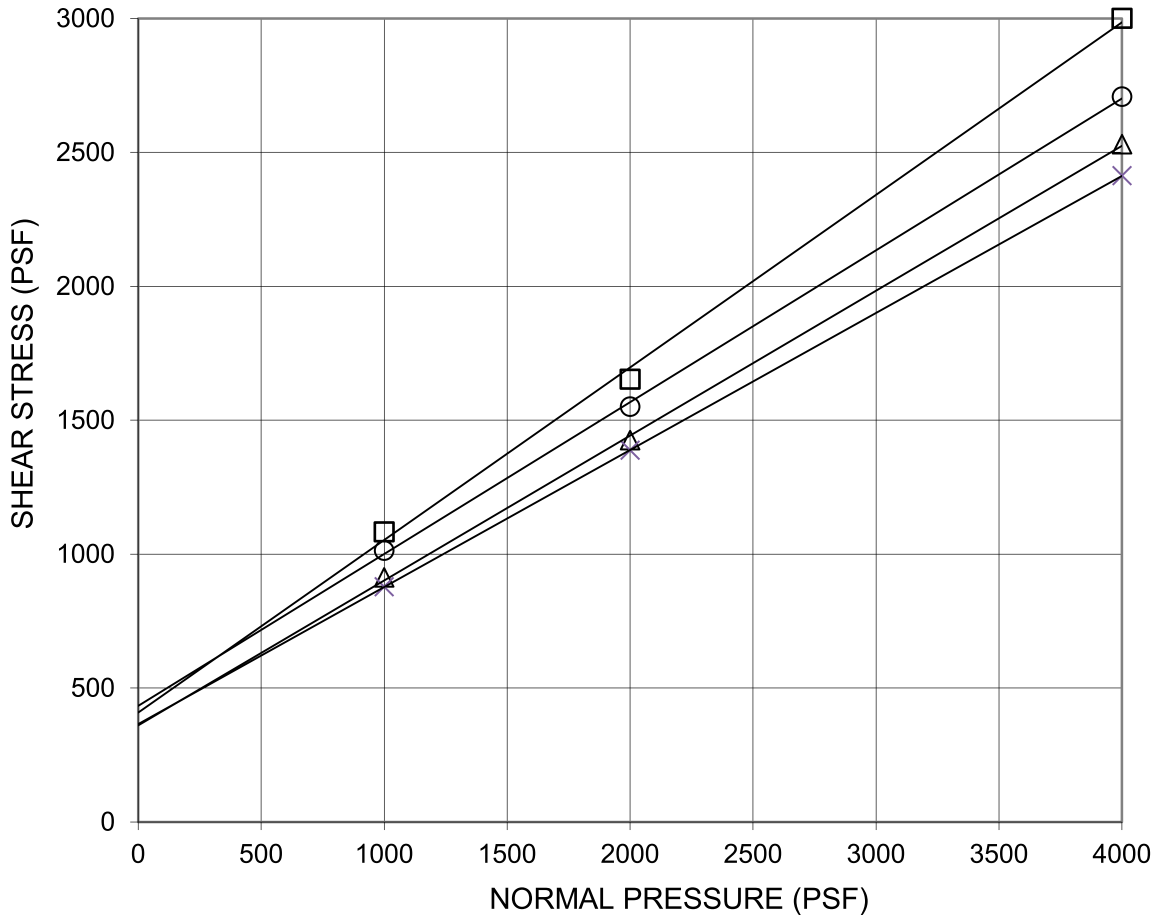
Geotechnical, Environmental & Civil Engineering Services

Project Name/Address:

Foundations Repair
3120 Elvido Drive
Los Angeles, CA

DIRECT SHEAR TEST

(ASTM D3080)



Date: September 19, 2018

SYMBOL	SHEAR STRENGTH	DEPTH (FT)	SAMPLE TYPE	SOIL TYPE	COHESION (PSF)	FRICTION ANGLE (DEG)
1st Pass □ 2nd Pass ○ 3rd Pass △ 4th Pass X	Residual (Ultimate)	7	Undisturbed Ring	Shaly Siltstone, Bedrock	365	27.1

Avg. Initial Dry Density = 76.1 pcf

Vertical Loads (PSF)	Moisture Before Test (%)	Moisture After Test (%)
1000	22.5	30.9
2000	24.2	31.5
4000	25.3	33.1

CalLand Engineering, Inc.
Dba Quartech Consultants

Geotechnical, Environmental & Civil Engineering Services

Project Name/Address:

Foundations Repair
3120 Elvido Drive
Los Angeles, CA

DIRECT SHEAR TEST

(ASTM D3080)

ZS Engineering
113 Tomato Springs
Irvine, CA 92618

QCI Project No.: 18-213-009b
Date: September 24, 2018
Summarized by: MW

Project Name: Foundations Repair - 3120 Elvido Drive, Los Angeles, CA
ZS Engineering No.: 180902

Summary of Laboratory Testing Data
(Sulfate & Chloride Contents)

Test Pit No.	Sample Depth (ft)	Chloride CT-422 (ppm)	Sulfate CT-417 (% by Weight)
TP-1	0 - 3	12	0.0024
TP-1	5 - 7	114	0.16

APPENDIX C

Specifications Guidelines for Drilled Pier Installation

Specifications Guidelines for Drilled Pier Installation

1. Pursuant to Section 1705.8 of the 2017 LABC, continuous observation by a representative of the Geotechnical Consultant shall be performed during drilling holes for the CIDH (Cast-In-Drilled-Hole) piers in order to confirm that the dimensions, embedments of the installed piers are compliant with the approved foundation plans, and that pier installation has been performed as specified. The contractor shall provide access and necessary facilities, including droplights, at contractor's expense, to accommodate observations inside the drilled hole.

2. Pier installation shall be performed such that compliance with all safety rules and requirements is achieved. Drilling equipment, casing, reinforcement, and other items required for installation shall be kept a safe distance from all overhead lines.

3. Piers shall be located as indicated on the drawings. Any pier installed, having a center more than three inches off plan centerlines will require structural analysis. The cost of such analysis and any work or materials resulting from correcting an error in location of piers shall be borne by the contractor.

4. Pier shafts shall be plumb to a tolerance of not more than 1 inch in 6 feet.

5. Bottoms of the pier footing excavations shall need to be cleaned out of any loose materials, sloughs generated from drilling so that pier tip can be on competent native soils.

6. At the completion of drilling, secure covers shall be placed over pier excavations. Concrete placement shall begin within 4 hours after completion of drilling. Pier shafts spaced closer than 5 times the pier diameter (center-to-center) shall be drilled and filled with concrete alternately, allowing at least 12 hours after concrete placement in one shaft before drilling of an adjacent shaft.

7. Concrete shall not be allowed to fall freely more than 4 feet. Concrete pumps, tremies or other such devices that are used for concrete pour shall comply with this requirement. Concrete placement shall continue until concrete extends to the top of the pier shaft. The tremie or concrete pump pipe may be raised slowly as the pier shaft is filled with concrete, provided that the bottom of the pipe is never more than 4 feet above the level of the concrete.

8. If caving is encountered during drilling, metal casings shall be placed in the drilled hole for support against caving. In case of casing, concrete placement and casing pull out shall be

done simultaneously with the bottom of the casing not being pulled above the top of concrete at any time during the entire process.

9. Reinforcement (rebar cage) shall be rigidly installed and secured to prevent movement or dislodgement during concrete placement.

10. In the event that pier installation procedures specified above are not adhered to, the contractor may be required to core the concrete pier to confirm that a continuous concrete pier has been installed. The cost of such coring shall be borne by the contractor.

11. Any piers deemed defective shall be replaced with substitute piers as directed by the Structural Engineer. The cost of installation of such substitute piers shall be borne by the contractor. Costs associated with analysis and design of substitute piers shall also be borne by the contractor.