Bay City Geology, Inc.

SOILS ENGINEERING INVESTIGATION

PROPOSED NEW 4-STORY AT-GRADE SELF-STORAGE BUILDING & TWO NEW DETACHED 1-STORY AT-GRADE SELF-STORAGE BUILDINGS

Tract: 18349, Lot: 26 & 27
APN: 2746-009-023

9143 N. De Soto Avenue
Chatsworth, California 91311

for

9143 De Soto Investments LLC

Project 2557

August 11, 2023

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De Soto Investments, LLC

Subject: **SOILS ENGINEERING INVESTIGATION**
Proposed New 4-Story At-Grade Self-Storage Building &
Two New Detached 1-Story At-Grade Self-Storage Buildings
9143 N. De Soto Avenue
Chatsworth, California

Ladies/Gentlemen:

Bay City Geology, Inc. is pleased to submit this geotechnical engineering report to provide recommendations for the proposed project.

Based on this investigation, it is our opinion that the proposed construction is feasible from a geotechnical engineering standpoint provided the recommendations contained herein are incorporated into the project design plans and specifications. This report should be reviewed in detail prior to proceeding further with the planned development. When final plans for the site development become available, or if the proposed construction is revised, the plans should be forwarded to this office for review and comment.

The scope of this investigation is limited to the project area as depicted on the Plot Map(s) herein. This report is not a comprehensive evaluation of the entire property and may not contain sufficient information for other than the intended use. Prior to use by others, Bay City Geology, Inc. should be consulted to determine if additional work is required. If the project is delayed more than one year, this office should be contacted to verify current site conditions and prepare an update report.

We appreciate the opportunity of serving you on this project. If you have any questions pertaining to our report, or if we can be of further service, please do not hesitate to contact us. This report may not be copied. If you wish additional copies, you may order them from this office.

Respectfully submitted,
BAY CITY GEOLOGY, INC.

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GEOTECHNICAL SOILS ENGINEERING INVESTIGATION
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INTRODUCTION

This report details the results of a limited Soils Engineering Investigation on a portion of the subject property. The purpose of this investigation has been to ascertain the subsurface conditions pertaining to the design and construction of the proposed new 4-story, at-grade storage building and two new 1-story, at-grade detached storage buildings. Review of the project included reconnaissance mapping, description of earth materials, determining soil structure, obtaining representative earth material samples, performing laboratory testing, engineering analyses, and preparation of this report. Findings, conclusions and appropriate recommendations are included herein.

SCOPE

The scope of this investigation includes the following:

- Review of preliminary plans by James Goodman Architecture.
- Review of (4) test pit explorations. Explorations were backfilled with the excavated materials.
- Preparation of the enclosed Plot Map(s), (see Appendix I).
- Sampling of representative earth materials, laboratory testing and analyses, (see Appendix II).
- Review of reference materials and available public reports at the City of Los Angeles, Department of Building & Safety, (see Appendix V).
- Presentation of findings, conclusions, and recommendations for the proposed project.

A plot map was prepared from data available online (http://navigatela.lacity.org) and utilized as a base map for this investigation. Preliminary building plans were prepared by John Goodman Architecture and also utilized as a base map. Both maps consist of one sheet plotted to a scale of one-inch equals forty feet.

The scope of this investigation is limited to the project area explored as depicted on the enclosed Plot Map(s). This report is not a comprehensive evaluation of the entire property. This report has not been prepared for use by other parties or for other purposes, and may not contain sufficient information for other than the intended use. Prior to use by others, Bay City Geology, Inc. should be consulted to determine if additional work is required. If the project is delayed more than one year, this office should be contacted to verify current site conditions and prepare an update report.

PROPOSED DEVELOPMENT

It is our understanding from discussions with the project architect, that the site will be developed with a new 4-story, at-grade self-storage building and two new detached 1-story at grade self-storage buildings. Grading will consist of conventional removal and re-compaction methods to create an engineered building pad. Final building plans have not been prepared and await the conclusions and recommendations of this investigation.

SITE DESCRIPTION

Location and Description

The site is essentially flat-lying and accessed from De Soto Avenue via Nordhoff Street in the Chatsworth neighborhood of the City of Los Angeles. The site previously consisted of three commercial buildings, a swimming pool with spa, and surface parking. All of these previous structures and improvements have been demolished and wasted from the property and the site is currently vacant. Details of the site are depicted on the enclosed Location Map and Plot Map(s) in Appendix I.

The adjacent property to the north of the site is developed with commercial buildings and asphalt parking, with driveways located along the property line. The adjacent property to the south of the subject site is developed with concrete buildings which were partially demolished at the time of the investigation, (August 2, 2023). An approximately (8)- to (10)-foot high concrete masonry unit (CMU) freestanding wall is located along the common property boundary with the adjacent property to the south of the site.

Drainage

Surface water at the site consists of direct precipitation onto the property. Much of this water drains as sheet flow to low-lying areas, offsite and/or to the street.

Groundwater

No active surface groundwater seeps or springs were observed on the subject site. The subsurface exploration did not encounter groundwater to a depth of (9) feet below adjacent site grades. The historic high groundwater level was obtained from review of the California Division of Mines and Geology (CDMG) *Seismic Hazard Zone Report (SHZR 007) for the Canoga Park 7.5-Minute Quadrangle* (1997, 2005). Review of this report indicates that the historically highest groundwater level is on the order of about (70) feet below site grade. Seasonal fluctuations of groundwater levels may occur by varying amounts of rainfall, irrigation and recharge. Groundwater is not anticipated to pose a problem to the proposed project.

SUMMARY OF FINDINGS

Previous Work

The subject property was originally developed circa 1965, after the City of Los Angeles Grading Ordinance. No geology and/or geotechnical reports were found on file at the City of Los Angeles Department of Building and Safety (LADBS) covering original development of the site. The previous structures onsite were demolished in 2022.

The client provided us with a compaction report prepared by Rybak Geotechnical, Inc. (RG), and dated May 2, 2023. The RG compaction report details the removal and backfilling of the previously onsite swimming pool with spa. Our office reviewed the Rybak Geotechnical compaction report prior to beginning work on this project.

Rybak Geotechnical, Inc. (RG), reports that the pool and spa shells were removed. The former pool bottom was excavated exposing firm, competent natural soils and was observed and approved by a RG representative. The RG report indicates that the maximum vertical depth of the compacted former pool backfill is (4.5) feet. The RG indicates that the recently placed engineered, compacted pool backfill was certified as ‘Second Structural’ fill for support of concrete flatwork.

Stratigraphy

The site is underlain by non-marine sedimentary soils of Pleistocene time which are covered by minor anthropogenic artificial fill materials. The earth materials encountered on the subject property are briefly described below. Approximate depths and more detailed descriptions are given in the enclosed Exploration Logs (see Appendix I).

Artificial Fill (Af)

Undocumented anthropogenic artificial fill materials were presumably placed during original and/or previous development of the subject property. The existing artificial fill materials were encountered within each of the four test pit explorations performed onsite and were observed to range in thickness from (1.0) to (2.0) feet. The existing fill materials generally consist of light-brown silty sand with gravel.

Engineered, Compacted Pool Backfill Materials
Additionally, recent grading on has resulted in the placement of engineered, compacted pool backfill materials, (see Previous Works section, above). The engineered, compacted pool backfill materials were placed as backfill for the former swimming pool and spa onsite and were certified as 'Secondary Structural' fill for support of concrete flatwork. The engineered, compacted pool backfill materials were not encountered within the test pit explorations. However, the limits of the engineered, compacted pool backfill materials, (as reported by the previous geotechnical consultant responsible for monitoring of the pool backfilling), are shown on the enclosed Plot Map(s).

Quaternary Alluvium (Qa)
Alluvium is weathered bedrock materials, (AKA- soils), that have eroded from natural ascending slopes and accumulated in generally flat-lying areas. The native alluvial soils were encountered within each of the four test pit explorations performed onsite with an observed thickness greater than (8) feet. The native alluvial soils consist of medium-brown, dense, silty sand and fine sand with pebbles and gravels up to about (2) inches in length.

Seismicity
There are several active and/or potentially active faults within Southern California. Any future movement on these faults could possibly affect structures in the built environment due to seismic induced ground shaking, acceleration, and/or rupture. The time, location, magnitude, amount of fault displacement, and shaking duration of an earthquake cannot be accurately predicted.

Ground motion caused by an earthquake is likely to occur at the site during the lifetime of the development due to the proximity of several active and potentially active faults. The American Society of Civil Engineers (ASCE) Structural Engineering Institute (SEI) in conjunction with the United States Geological Survey (USGS) have developed the most recent *Standard ASCE/SEI 7-16 - Minimum Design Loads and Associated Criteria for Buildings and Other Structures: Provisions and Commentary* (2017) and maintain the *ASCE 7 Hazards Tool*, (https://asce7hazardtool.online/), website and search engine for the prediction of peak and design-level earthquake ground motions. The earthquake induced ground motions anticipated for the subject site, as determined from the *ASCE 7 Hazards Tool* website search engine output, are provided in the Appendix III. Generally, on a regional scale, quantitative predictions of ground motion values are linked to peak acceleration and repeatable acceleration, which is a response to earthquake magnitudes relative to the fault distance from the subject property.

This seismic evaluation is designed to provide the client with current, rational, and believable seismic data that could affect the property during the lifetime of the proposed improvements. The minimum design acceleration for a project is determined from the *ASCE 7 Hazards Tool* website search engine. It is recommended that the structural design of the proposed project be based on current design acceleration practices of similar projects in the area.

Santa Susana – San Fernando - Sierra Madre – Cucamonga Fault System
The Santa Susana - Sierra Madre - Cucamonga Fault System, within the central and eastern portions of the Transverse Ranges Geomorphic Province, refers to the entire 125-km-long complex system of mechanically related thrust and reverse faults that grossly demarcate the mountain fronts of the Santa Susana Mountains in the west to the San Gabriel Mountains in the east. The fault system includes the Santa Susana Fault Zone, San Fernando fault, Sierra Madre fault, and Cucamonga fault.

San Fernando Fault
The San Fernando fault is a left-lateral/reverse frontal fault that extends along the southern margin of the Santa Susana Mountains. According to Tsutsumi and Yeats (1999), the San Fernando fault is a flexural-

slip fault that formed on the south flank of the Mission Hills syncline and Merrick syncline during folding deformation. The 1971 San Fernando (Sylmar) earthquake produced a surface rupture approximately 15 kilometers in length, that is now recognized as the San Fernando Fault Zone.

In 1976, the State of California classified the San Fernando Fault Zone and an eastern portion of the Santa Susana Fault Zone as active faults and provided the active portions of the faults with Earthquake Fault Zones, in accordance with the Alquist-Priolo ("AP") Earthquake Fault Zoning Act of 1972. The subject site is located approximately (6.3) miles southeast of the San Fernando Fault Earthquake Fault Zone, as delineated by the California Geological Survey (CGS). Seismically induced ground rupture at the site is not anticipated due to an earthquake occurring on the San Fernando fault.

Mission Hills Fault
The Mission Hills fault is a north-dipping (40 to 50 degrees) reverse fault that extends east-west along the southern edge of Granada Hills and Mission Hills. The Mission Hills fault is considered to be an active fault, with the most recent seismic event occurring in the late-Quaternary to Holocene. The Mission Hills fault is believed to be the southwestern extension of the San Fernando fault. Both Granada Hills and Mission Hills have been uplifted by long term reverse displacement of the hanging wall. The fault extends eastward toward the eastern edge of the hills near the I-5 freeway. There the fault is believed to turn southeastward toward the Verdugo fault (Tsutsumi and Yeats, 1999).

The Mission Hills fault has not been provided with an Alquist-Priolo ("AP") Earthquake Fault Zone by the State of California. The subject site is located approximately (3) miles southwest of the mapped surface trace of the Mission Hills fault, (Dibblee, 1992). Seismically induced ground rupture at the site is not anticipated due to an earthquake occurring on the Chatsworth fault.

Simi - Santa Rosa Fault
The Simi - Santa Rosa fault is comprised of a zone of high-angle north dipping reverse fault splays with some left-lateral oblique motion which trends northeast-southwest for approximately 40 kilometers, from the northeastern end of Simi Valley to the Oxnard plain, along portions of the southeastern Santa Susana Mountains, the southern Las Posas Hills, southern and western Camarillo Hills. The Simi - Santa Rosa fault is considered to be an active fault capable of producing a magnitude M7 earthquake, with the most recent ground rupture occurring in the Holocene epoch, (Hitchcock *et al.*, 2003).

The Simi - Santa Rosa fault has been provided with an Earthquake Fault Zone, in accordance with the Alquist-Priolo (AP) Earthquake Fault Zoning Act of 1972. The subject site is located approximately (7.3) miles southeast of the Simi - Santa Rosa Fault Earthquake Fault Zone, as delineated by the California Geological Survey (CGS). Seismically induced ground rupture at the site is not anticipated due to an earthquake occurring on the Simi - Santa Rosa fault.

Northridge Hills Fault
The Northridge Hills fault is comprised of several north-dipping reverse fault splays which trend approximately east-west to northwest-southeast for approximately (15) kilometers, from Porter Ranch, (west of the terminus of the Mission Hills fault), across Browns Canyon, Devils Canyon, and Blind Canyon in the southeastern Santa Susana Mountains, and terminating in northeast Simi Valley, (east of the terminus of the Simi-Santa Rosa fault). The Northridge Hills fault may possibly be a structural link between the Simi - Santa Rosa fault and the Mission Hills fault. The Northridge Hills fault is considered to be a potentially active fault capable of producing a magnitude M6+ earthquake, with the most recent ground rupture occurring in the late-Quaternary period, (Baldwin *et al.*, 2000).

The Northridge Hills fault has not been provided with an Alquist-Priolo ("AP") Earthquake Fault Zone by the State of California. The subject site is located approximately (2) miles southwest of the surface geomorphic features inferred to be located along the Northridge Hills fault, (Dibblee, 1992). Seismically induced ground rupture at the site is not anticipated due to an earthquake occurring on the Northridge Hills fault.

Chatsworth Fault
The Chatsworth fault is comprised of several north-dipping reverse fault splays which trend northeast-southwest to roughly east-west, extending from western Porter Ranch through Chatsworth and West Hills and into Ventura County in the eastern Simi Hills, where the central, main fault splay converges with the Burro Flats fault, (Langinheim *et al.*, 2011). The Chatsworth fault is considered to be a potentially active fault capable of producing a magnitude M6.0 - M6.8 earthquake, with the most recent ground rupture occurring in the late-Quaternary period, (SCEDC, 2021).

The Chatsworth fault has not been provided with an Alquist-Priolo ("AP") Earthquake Fault Zone by the State of California. The subject site is located approximately (2) miles east-southeast of the inferred surface trace of the Chatsworth fault, (Dibblee, 1992). Seismically induced ground rupture at the site is not anticipated due to an earthquake occurring on the Chatsworth fault.

Ground Acceleration and Shaking
Significant ground acceleration and shaking should be anticipated due to the relatively close proximity of the site to several active and/or potentially active faults within the local area. Generally, all of southern California is located in a seismically active region and some areas have a higher potential for seismic damage than other areas. The nearer a property is to an active fault, then a greater probability for significant ground shaking and higher ground acceleration should be anticipated. Thus, earthquake insurance and building code upgrades are suggested.

Liquefaction
Liquefaction is a process by which sediments below the water table temporarily lose strength and behave as a viscous liquid rather than a solid. The types of sediments most susceptible are clay-free deposits of sand and silts; gravel only occasionally liquefies. The actions in the soil which produce liquefaction are as follows: seismic waves, primarily shear waves, passing through saturated granular layers, distort the granular structure, and cause loosely packed groups of particles to collapse. The pore-water pressure between grains increases if drainage cannot occur. If the pore-water pressure rises to a level approaching the weight of the overlying soil, the granular soil layer temporarily behaves as a viscous liquid rather than a solid.

In the liquefied condition, soil may deform with little shear resistance; deformations large enough to cause damage to buildings and other structures are called ground failures. The ease with which a soil can be liquefied depends primarily on the looseness of the material, the depth, thickness, and areal extent of the liquefied layer, the ground slope, and the distribution of loads applied by buildings and other structures.

The State of California has prepared Seismic Hazard Zone Reports and Maps to regionally map areas where historic occurrence of liquefaction, or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacement. The maps may not identify all areas that have potential for liquefaction, strong ground shaking, or other earthquake-related geologic hazards.

The subject site is not located within a Liquefaction Hazard Zone as designated on the California Geological Survey (CGS) *Canoga Park Quadrangle (EZRIM) Earthquake Zones of Required Investigation Map,* (1998), (Plate 3). The historic high groundwater level was obtained from review of the California Division of Mines and Geology (CDMG) *Seismic Hazard Zone Report (SHZR 007) for the Canoga Park 7.5-Minute Quadrangle* (1997, 2005). Review of this report indicates that the historically highest groundwater level is on the order of (70) feet below site grade, (Plate 4). Additionally, review of the Dibblee *Geologic Map of* *the Oat Mountain and Canoga Park (North ½) Quadrangles, Los Angeles County, California* (1992), (Plate 2), indicates that the subject site is underlain Pleistocene-age alluvial soils (Qa). Therefore, due to the age, density and consistency of the earth materials underlying the subject site, it is the opinion of this firm that the potential for liquefaction is very low.

CONCLUSIONS

1. Based on the results of this investigation and a thorough review of the proposed development, as discussed, the project is suitable for the intended use providing the following recommendations are incorporated into the design and subsequent construction of the project. Also, the development must be performed in an acceptable manner conforming to the Building Code requirements of the controlling governing agency.

2. The subject site is not located within a Liquefaction Hazard Zone as designated on the California Geological Survey (CGS) *Canoga Park Quadrangle (EZRIM) Earthquake Zones of Required Investigation Map* (1998), (Plate 3). The historic high groundwater level was obtained from review of the California Division of Mines and Geology (CDMG) *Seismic Hazard Zone Report (SHZR 007) for the Canoga Park 7.5-Minute Quadrangle* (1997, 2005). Review of this report indicates that the historically highest groundwater level is on the order of (70) feet below site grade, (Plate 4). Additionally, review of the Dibblee *Geologic Map of the Oat Mountain and Canoga Park (North ½) Quadrangles, Los Angeles County, California* (1992), (Plate 2), indicates that the subject site is underlain Pleistocene-age alluvial soils (Qa). Therefore, due to the age, density and consistency of the earth materials underlying the subject site, it is the opinion of this firm that the potential for liquefaction is *very low*.

3. The SITE CLASS for the proposed project based upon available geotechnical data and the most recent *ASCE/SEI Standard 7-16* (2017) is the "Site Class 'D' - Default." Additional seismic design values are listed in Appendix III.

4. Based upon field observations, soils laboratory testing, and engineering analysis, the Quaternary alluvial soils (Qa) observed in the test pit explorations and underlying the subject site should possess sufficient strength to support the proposed project, including any new engineered, compacted fill materials for the proposed development.

RECOMMENDATIONS

Specific Recommendations

1. Minor amounts of anthropogenic artificial fill materials (Af) have been observed to blanket the subject site with an approximate thickness of about (1.0) feet. The existing fill materials (Af) are not considered suitable for support of new engineered, compacted fill materials, foundations, concrete slabs-on-grade, and/or concrete hardscape.
2. Grading and earthwork should be utilized to create a new uniform building pad area for support of the proposed new 4-story and 1-story self-storage buildings. The proposed new storage buildings should be supported on shallow conventional foundations. Alternatively, the proposed new storage buildings may be supported on mat foundations. The recommended new conventional and/or mat foundations shall bear entirely into the recommended new properly placed engineered, compacted fill materials.
3. Grading and earthwork to create a new uniform building pad area for the proposed project should include removals of any unsuitable/surficial earth materials, *e. g.* the existing fill materials (Af). All

excavated on-site earth materials should be replaced as new properly placed engineered, compacted fill materials.

Any excavations to remove unsuitable/surficial soils and create new building pad areas should extend laterally at least (3) feet beyond the planned new building-lines. Also, removal excavations should extend to a minimum depth of at least (3) feet below the base of the proposed new foundations.

4. Grading and earthwork should be utilized for subgrade preparation for support of any planned new parking area paving. New parking area paving may consist of either rigid concrete pavement or flexible asphalt-concrete (A/C) pavement. Also, the property owner should be aware that removal of all existing fill materials in the area of new paving is not required under the current Building Code; however, pavement constructed over existing fill materials which have not been removed and replaced as new engineered, compacted fill will most likely have a shorter design life and increased maintenance costs.

   The grading over-excavations to remove unsuitable and/or disturbed earth materials should extend at least (24) inches below any planned new parking area subgrade elevation. The over-excavated on-site earth materials are considered suitable for use as new engineered, compacted fill materials and should be properly placed as new engineered, compacted ("Secondary") fill for support of any planned new parking area paving.

5. All grading and earthwork performed for construction of the proposed development shall be performed as outlined in the Grading & Earthwork section of the enclosed Appendix IV – General Recommendations. Also, numerous utility lines should be anticipated to be encountered during grading in multiple and various locations across the subject site. Special care should be taken during grading to properly locate and delineate the existing utility lines. Also, proper methods and procedures shall be implemented to protect and/or abandon any such utility lines.

6. **The undersigned geologist and soils engineer shall review and approve by signature and stamp the detailed plans PRIOR to issuance of any building and/or grading permits to verify that the plans include the recommendations provided herein**.

7. **All grading over-excavations and new foundation excavations shall be observed and verified by the project engineering geologist and/or soils engineer during construction and/or prior to placing steel and concrete.** The required excavation observations are intended to verify conformance of the excavations with the recommendations provided herein, with the design structural engineering plans, with the current Building Code, and with the specific requirements of the local governmental reviewing agency.

8. **Prior to beginning any site grading, removals, and/or excavations for foundations a project pre-construction / pre-grading meeting should be held on-site.** The project pre-construction / pre-grading meeting should be attended by the project engineering geologist and/or soils engineer and by representatives of the property owner, the project architect, the project structural engineer, the project civil engineer, the general contractor, and/or the grading contractor, and by the local building department official/inspector.

9. The property owner shall maintain the site as outlined in the Drainage and Maintenance section of Appendix IV – General Recommendations.

Conventional Foundations

The proposed new 4-story and 1-story self-storage buildings should be supported on shallow conventional foundations bearing into the recommended new properly placed engineered, compacted fill materials. Conventional foundations should consist of either continuous and/or pad footings and grade beams, (or other suitable structural members).

The minimum conventional foundation design recommendations are given as follows:

- *Allowable Bearing Pressures:*
  - Strip Footings — 2,000 pounds per square foot
  - Column Footings — 2,500 pounds per square foot
  - Maximum Allowable — 4,000 pounds per square foot

- *Allowable Bearing Pressure Increases:*
  - For Additional Footing Width — 200 pounds per square foot, per foot
  - For Additional Footing Depth — 400 pounds per square foot, per foot

- *Minimum Footing Widths:*
  - Strip Footings — 12 inches
  - Column Footings — 24 inches (square)

- *Minimum Footing Embedment Depths:*
  - Strip Footings — 24 inches
  - Column Footings — 24 inches

  - All foundation embedment depths shall be measured into the recommended bearing material, below the lowest adjacent grade.

- *Lateral Resistance Parameters:*
  - Coefficient of Friction — 0.30
  - Passive Earth Resistance (acting as a fluid) — 300 pounds per square foot, per foot
  - Maximum Passive Earth Pressure — 3,600 pounds per square foot

  - Lateral loads may be resisted by friction acting at the base of the footings and/or by passive resistance within the recommended bearing material.

The foundation bearing values provided above are for the total of dead and frequently applied live loads and include a Factor-of-Safety of at least (3). These bearing values may be increased by a factor of (⅓) for temporary loads, such as, wind and seismic forces. The bearing values given above are net bearing values; the weight of concrete below grade may be neglected.

When combining passive earth pressure and friction for lateral resistance, the passive earth pressure component should be reduced by one-third. The coefficient of friction should be applied to dead load forces only.

All continuous footings shall be reinforced with a minimum of (4) #(4) bars, two placed near the top and two near the bottom. Reinforcing recommendations are minimums and may be revised by the project structural engineer.

All foundation excavations are required to be observed by the project soils engineer and/or project engineering geologist. Footing depths will be measured from the lowest adjacent grade of recommended bearing material. Footing depths will not be measured from any proposed elevations or grades. Any foundation excavations that are not the recommended depth into the recommended bearing material will not be acceptable to this office.

Mat Foundation

As an alternative to conventional foundations, the proposed new 4-story and 1-story self-storage buildings may be supported on new mat foundations bearing entirely into the recommended new properly placed engineered, compacted fill materials. Additionally, the recommended mat foundations shall be designed to withstand the anticipated static settlement potentials. The final design of the recommended mat foundation shall be provided by the project structural engineer.

The geotechnical mat foundation design recommendations are given as follows:

❖ *Mat Foundation Design Parameters:*

➢ Soil Subgrade Reaction Modulus:	108 kips per cubic foot
➢ Minimum Mat Foundation Thickness:	15 inches
➢ Allowable Average Bearing Pressure:	1,500 pounds per square foot
➢ Maximum Allowable Bearing Pressure: *(for point loads under mat foundation)*	3,000 pounds per square foot

Increases in the mat foundation allowable bearing pressure, for support of point loads under the mat, are allowable up to the maximum allowable bearing pressure. Additionally, given the recommended minimum mat thickness of (15) inches, the project structural engineer should consider including design measures to ensure that the recommended mat foundation behave as a mat under structural loading and not as a thickened membrane. Suggested design measures include:

- Utilizing reinforced steps located at critical structural loading points, such as load-bearing internal walls or at a division wall between compartments or occupations,
- Utilizing reinforced ribs in the mat to increase mat stiffness and structural support, and/or
- Increasing the mat thickness and reinforcing, especially under areas of critical structural loading.

❖ *Lateral Resistance Parameters:*

➢ Coefficient of Friction	0.30
➢ Passive Earth Resistance (acting as a fluid)	300 pounds per square foot, per foot
➢ Maximum Passive Earth Pressure	3,600 pounds per square foot

➢ Lateral loads may be resisted by friction acting at the base of the mat foundation and/or by passive resistance within the recommended bearing material.

The mat foundation allowable average bearing value provided above is for the total of dead and frequently applied live loads and includes a Factor-of-Safety of at least (3). The bearing value may be increased by a factor of (⅓) for temporary loads, such as, wind and seismic forces.

When combining passive earth pressure and friction for lateral resistance, the passive earth pressure component should be reduced by one-third. The coefficient of friction should be applied to dead load forces only.

The recommended Modulus of Subgrade Reaction value may be scaled using the empirical relations proposed by Terzaghi (1955), and as provided in Naval Facilities Engineering Command, Design Manual 7.02 (1986), given as:

$k_s = (k_1) * [(B+B_1)/(2B)]^2$
(for square footings)

$k_{s,rect.} = (k_s) * [1+(B/2L)]/(1.50)]$
(for rectangular footings)

Where:
'k_1' is the Subgrade Reaction Modulus for a (12)-inch test plate,
'B_1' is the test plate width, typically (12) inches square,
'B' is the foundation width,
'L' is the foundation length, and
'k_s' is the scaled Subgrade Reaction Modulus
(for square footings)

All foundation excavation and embedment depths will be measured from the lowest adjacent grade of recommended bearing material. Foundation depths will not be measured from any proposed elevations or grades. Any foundation excavations that are not the recommended depth into the recommended bearing material will not be acceptable to this office.

Foundation Static Settlement
Static settlement of the proposed new development should be anticipated. Estimated total column foundation loads for the proposed self-storage buildings are on the order of (60) kips, with an estimated dead load component of (20) kips and an estimated live load component of (40) kips. New estimated wall foundation loads for the proposed self-storage buildings are on the order of (7) kips per lineal foot.

The native alluvial soils (Qa) underlying the subject site and the recommended new properly placed engineered, compacted fill materials are anticipated to be dense and competent. Therefore, significant static settlement of the new foundations is not anticipated for the expected foundation loading conditions.

Conservatively, static settlement of the proposed development should be anticipated to occur. Static settlement on the order of (¼) to (½)-inches between walls or piers, within (20) feet or less, of each other, and under similar loading conditions, is considered normal. Total static settlement on the order of less than (½)-inch should be anticipated.

Expansive Soils
Our experience indicates that the earth materials at the site are anticipated to exhibit a VERY LOW to LOW expansion potential. As such, special considerations for expansive soil conditions are not required for the proposed development.

Expansive soils can be a problem, as variation in moisture content will cause a volume change in the soil. Expansive soils heave when moisture is introduced and contract as they dry. During inclement weather and/or excessive landscape watering, moisture infiltrates the soil and causes the soil to heave (expansion). When drying occurs the soils will shrink (contraction). Repeated cycles of expansion and contraction of soils can cause pavement, concrete slabs on grade and foundations to crack. This movement can also result in misalignment of doors and windows. To reduce the effect of expansive soils, foundation systems are usually deepened and/or provided with additional reinforcement design by the structural engineer.

Planning of yard improvements should take into consideration maintaining uniform moisture conditions around structures. Soils should be kept moist, but water should not be allowed to pond. These designs are intended to reduce, but will not eliminate, deflection and cracking and do not guarantee or warrant that cracking will not occur.

Site Drainage
Proper surface drainage is critical to the future performance of the project. Saturation of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the designed engineering properties. Proper site drainage should be maintained at all times.

All site drainage should be collected and transferred to the street, or an approved drainage facility (per 2023 LABC §91.7013.9-10) in non-erosive drainage devices. The proposed development should be provided with roof drainage. Discharge from downspouts, roof drains and scuppers should not be permitted on unprotected soils within (5) feet of the building perimeter. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. Drainage should not be allowed to flow uncontrolled over any descending slope. Planters located within wall backfill and/or near foundations should be sealed to prevent moisture intrusion into the subgrade earth materials.

SUSMP Infiltration
Recently, in compliance with state and federal environmental protection legislation, municipalities have begun requiring that development projects include a "Standard Urban Storm water Mitigation Plan" (SUSMP). The project SUSMP shall include "Best Management Practices" (BMPs) for the mitigation and management of storm water runoff at the project site.

Additionally, the project SUSMP should incorporate a "Low Impact Development" (LID) design strategy. LID is a storm water management strategy that seeks to mitigate the impacts of increases in runoff and storm water pollution as close to its source as possible. LID comprises a set of site design approaches and BMPs that promote the use of natural systems for infiltration, evapotranspiration, and use of storm water.

The primary tenet of the LID strategy is the disposal of a certain amount, typically the first (0.75)-inches, of storm water generated on a site by infiltration into the on-site soils. However, this requirement goes against prudent engineering practice. Increasing the moisture content of a soil can have severe adverse effects including, but not limited to:

- loss of soil internal shear strength,
- increase in soil compressibility,
- changes in the design engineering properties of the on-site soils.

Thus, any overlying structures or improvements, including: buildings, pavements, and concrete flatwork, could sustain significant damage due to saturation of the subgrade soils. Structures serviced by subterranean levels could be adversely impacted by storm water disposal by increasing the design fluid pressures on retaining walls and causing leaks in the walls. Proper drainage is critical to the performance of any structure in the built environment.

The City of Los Angeles Bureau of Sanitation Watershed Protection Division (WPD) and the City of Los Angeles Department of Building and Safety (LADBS) both require a minimum setback of (10) feet from any proposed or existing site improvements, and from adjacent properties. Additionally, infiltration of storm water shall me a minimum of (10) feet above the groundwater table, as per LADBS Information Bulletin P/BC 2020-118.

The County of Los Angeles Department of Public Works Geotechnical and Materials Engineering Division (GMED) requires a minimum setback of (15) feet from any adjacent foundations. Additionally, infiltration of storm water shall be a minimum of (10) feet above the groundwater table, as per GMED Policy GS200.1 (2021).

The California Storm Water Quality Association recommends that infiltration devices should be installed no closer than (6) meters, or (19½) feet, from buildings, slopes and highway pavement. The Storm Water Managers Resource Center recommends that infiltration devices should be sited (25) feet down-gradient of structures. Often, these setback magnitudes are not possible on most urban projects.

The subject site is underlain by unsuitable existing fill materials (Af) which overlie native alluvial soils (Qa) consisting of clayey- to silty sand. Soil mixtures of silts, medium to fine sands and sands with gravels are considered to exhibit variable drainage properties in the "good" to "poor" drainage range. Coefficients of permeability ("*k*") for the soil types observed on-site are typically on the order of (10^{-3} to 10^{-6}) centimeters per second. On-site field percolation testing has yielded an in-situ percolation rate for the on-site soils that is on the order of (2.35) inches per hour.

The infiltration of storm water into the subgrade soils on-site is considered <u>feasible</u> provided all applicable governmental reviewing agency regulations are incorporated into the storm water infiltration system design. <u>The observed in-situ infiltration rate of (2.35) inches per hour may be utilized in storm water infiltration system design</u>.

The native alluvial soils (Qa) underlying the subject site <u>are considered suitable</u> for proposed infiltration of storm water. Establishment of a proposed storm water infiltration facility at the subject site <u>is not</u> anticipated to increase the potential for adverse conditions, including the following:

- Perched groundwater <u>is not</u> anticipated due to storm water infiltration at the subject site,
- Hydro-consolidation of the on-site soils <u>is not</u> anticipated to be significant,
- On-site soils <u>are</u> expected to exhibit a VERY LOW to LOW expansion potential. Thus, soil heaving from expansion due to soil saturation <u>is not</u> anticipated.
- Significant settlement of the native alluvial soils (Qa) due to soil saturation from infiltration <u>is not</u> anticipated at the project site or on neighboring sites,
- Basements and/or retaining walls <u>are not</u> present on-site, <u>are not</u> planned for the proposed project, but <u>may be present</u> on neighboring sites. Due to the relatively high permeability of the native granular soils on-site, saturation from storm water infiltration <u>is not</u> anticipated to adversely affect any neighboring basements or retaining structures provided the planned infiltration facility is located a minimum of (10) feet from both on-site structures and neighboring properties.

Any proposed storm water infiltration facility shall be designed in accordance with LADBS Information Bulletin P/BC 2020-118. Any planned infiltration devices: e. g. infiltration vaults, dry-wells, or infiltration columns, shall be located a minimum of (10) feet from any existing or proposed structure foundations. Also, any existing and/or proposed foundations shall be provided with a minimum clearance distance of (10) feet from the "Zone of Saturation" produced by infiltration of storm water into the subgrade soils at the project site. The "Zone of Saturation" shall be defined by the 1H:1V, (horizontal to vertical), extending downward from top of the permeable portion of the infiltration device. Additionally, any planned infiltration devices shall be provided with a minimum setback of (10) feet from adjacent private properties.

The minimum infiltration depth shall be either (5) feet below existing grade or (5) feet below the base of any newly placed engineered, compacted fill pad, whichever is deeper. Also, a minimum lid depth of (10) below the deepest foundation shall also be incorporated into the infiltration system design.

We recommend that the design team (including the structural engineer, waterproofing consultant, environmental engineer, civil engineer, plumbing engineer, and landscape architect) be consulted regarding the design and construction of SUSMP/LID BMP systems. Also, the client/property owner(s) should be aware that environmental legislation and/or storm water mitigation regulations are relatively new to many building departments, and are very dynamic, and may be subject to change without notice.

Lastly, the storm water mitigation and/or infiltration methods mentioned herein are widely accepted by most building departments. But, the specific requirements of the reviewing agency responsible for the proposed development will also greatly impact the final design of the BMP storm water mitigation system.

Temporary Excavations

Temporary excavations for construction of the proposed new 4-story and 1-story self-storage buildings are anticipated to include grading over-excavations for construction of the recommended new engineered, compacted fill building pad area. Conventional excavation equipment may be used to make these excavations. Excavations should expose dense, competent alluvial soils (Qa) overlain by minor existing fill materials (Af). All temporary excavations should be observed and verified by the project soils engineer and/or project engineering geologist during construction so that modifications can be made if variations in the earth materials occur.

All excavations should be stabilized within (30) days of initial excavation. If this time is exceeded, the project soils engineer must be notified, and modifications, such as shoring or slope trimming may be required. Water should not be allowed to pond on top of, or at the toe of, the excavations, nor to flow toward them. All excavations should be protected from inclement weather. Excavations should be kept moist, not saturated, to reduce the potential for raveling and sloughing during construction. No vehicular surcharge should be allowed within (3) feet of any excavation.

Vertical Excavations

The native alluvial soils (Qa) and/or minor existing fill materials (Af) are considered suitable for vertical excavations up to (5) feet in height. Vertical excavations greater than (5) feet in height should be trimmed and laid-back at a maximum gradient of 1H:1V, (horizontal to vertical), for the full height of the excavation.

Grading Over-excavations

Grading and earthwork over-excavations necessary to construct the proposed new building pad areas may be performed utilizing the A-B-C Slot Cutting Method. The planned grading over-excavations are not anticipated to exceed (5) feet in height and may be located within (5) feet of adjacent structures, neighboring properties, and/or public rights-of-way.

The A-B-C Slot-Cutting Method employs the use of the earth as a buttress and allows the excavation to proceed in phases. The slots are all cut to a maximum width of (8) feet and using an alternating A-B-C sequence.

The initial excavation is made at a slope of 1H:1V (horizontal to vertical). Then the “A” slots are excavated, with a maximum width of (8) feet, leaving the "B" and "C" slots to buttress the excavation. The "A" slots are then backfilled with the recommended new engineered, compacted fill materials. The same procedure is used to excavate and recompact the "B" slots, and then, lastly, the "C" slots.

Earthwork, Bulking, & Compaction Shrinkage

The temporary excavations planned for proposed project should be anticipated to encounter firm to dense, competent alluvial soils (Qa) overlain by minor existing fill materials (Af). The “Ease of Excavation” (or “Diggability Index”) of the on-site earth materials may be anticipated to be “Medium (‘M’).” A “Bulking,” (or “Swell”), factor of approximately (20%) may be utilized in earthwork calculations.

Compaction “Shrinkage” results when a volume of loose earth materials at one density and is placed and compacted to a higher density. A shrinkage factor of approximately (5%) to (15%) should be anticipated when placing the excavated on-site earth materials as new select engineered backfill materials with an average of (92%) relative compaction.

Slabs-on-Grade & Hardscape

Concrete conventional slabs-on-grade and/or outdoor concrete flatwork should be a minimum of (4) inches in thickness. Slabs-on-grade should be reinforced with a minimum of (#4) reinforcing bars, placed at (16) inches on center each way. Conventional slabs-on-grade and concrete flatwork may be supported directly on the dense, competent native alluvial soils (Qa) and/or on new properly placed engineered, compacted fill materials and should be underlain by (4) inches of crusher-run base mechanically compacted into place.

Grading and earthwork for subgrade preparation and slab support should include the following:

- Any existing uncertified/spilled fill and loose fill materials within the footprint of any proposed floor-slab or exterior slab areas should be over-excavated and removed to expose dense, competent native alluvial soils (Qa). The minimum over-excavation for placement of engineered compacted fill materials shall be (24) inches.

- The bottom of the over-excavation should be observed by the project soils engineer and/or engineering geologist.

- The bottom of the over-excavation should be scarified about (6) inches, moisture conditioned, and compacted.

- Engineered fill should be placed in loose lifts of about (4) to (6) inches in thickness and compacted.

- Engineered fill should be moisture controlled to be within (3) percent of the optimum moisture content.

- Engineered fill should be compacted to a minimum of (90) percent of the Modified Proctor Maximum Dry Density, per the latest edition of ASTM D 1557.

- Engineered fill should be tested for compaction with a minimum frequency of at every (2) vertical feet or (500) cubic yards of fill placed, whichever is MORE restrictive.

- Engineered fill should be surface tested for compaction at the proposed subgrade elevation.

- The project soils engineer and/or engineering geologist should be contacted to provide periodic observation of the grading operation and perform compaction testing of the engineered compacted fill placed.

- The project soils engineer and/or engineering geologist should prepare a final report detailing the grading and earthwork performed, placement and testing of the engineered compacted fill, and providing the as-built condition of the project area with respect to engineering geology.

Foundation excavation spoils should either be removed from the slab areas or compacted into place by mechanical means and tested for compaction.

For interior slabs and/or slabs where moisture control is required, a vapor retarder with a minimum thickness of (15)-mil should be placed below the concrete slab. The vapor retarder should conform to ASTM E1745 Class A with water vapor transmission rate <0.01 perms and should be installed in accordance with ASTM E1643. The structural engineer should provide design considerations such as reinforcement to offset potential increase in curling stresses in the slab.

Slabs, walkways, and decking are likely to crack as a result of shrinkage and curing processes of concrete. Typical concrete shrinkage can result in cracks and gaps along control joints and where slabs connect with structures. Slabs should be provided with proper control joints in an effort to control the location of the cracking. The gaps will require periodic caulking to limit infiltration of moisture.

Provisions for cracks should be incorporated into the design and construction of the foundation system, slabs and proposed floor coverings. Concrete slabs should have sufficient control joints spaced at a maximum of approximately (8) feet. Slabs-on-grade should be quartered or saw cut slabs to mitigate cracking and be isolated from the stem wall footing. Exterior slabs planned adjacent to descending slopes or planter areas should be provided with a thickened edge. The thickened edge should be a minimum of (12) inches wide and (24) inches deep and two #(4) bars.

Movement of slabs adjacent to structures can be mitigated by doweling slabs to perimeter footings. Doweling should consist of (#4) bars bent around exterior footing reinforcement. Dowels should be extended at least (2) feet into planned exterior slabs. Doweling should be spaced consistent with the reinforcement schedule for the slab. With doweling, (⅜) inch minimum thickness expansion joint material should be provided. Where expansion joint material is provided, it should be held down about (⅜) inch below the surface. The expansion joints should be finished with a color matched, flowing, flexible sealer (e.g., pool deck compound) sanded to add mortar-like texture. As an option to doweling, an architectural separation could be provided between the main structures and abutting appurtenant improvements.

These recommendations are considered as minimums unless superseded by the project structural engineer.

The on-site earth materials are anticipated to exhibit a VERY LOW to LOW expansion potential. Thus, in accordance with 2022 California Building Code (CBC) §1808.6.4, prior to pouring conventional slabs-on-grade, the existing slab sub-grade earth materials should be pre-saturated to a minimum moisture content of (105) percent of the optimum moisture content, per the latest edition of ASTM D1557. Pre-saturation of the slab sub-grade earth materials shall extend to a minimum depth of (24) inches below grade.

For exterior areas, new hardscape, (e.g.: walkway areas, pool areas, and driveway areas), may consist of flexible paving, including: A/C pavement and/or flexible and permeable paving stones. New flexible paving may be cast over newly placed engineered, compacted fill materials. New flexible paving should be designed for an expansive soil condition. Grading and earthwork, as outlined above and with a (24) inch over-excavation, should be utilized for preparation of subgrade soils for support of new flexible paving.

Additionally, the property owner should be aware that removal of all existing fill materials and/or native soil materials in the area of new flexible paving is not required. However, pavement constructed over native soil/existing fill materials will most likely have a shorter design life and increased maintenance costs. Also, if necessary, a 'Request for Modification' of the Building Code to allow placement of new engineered, compacted fill over competent native soil and/or existing fill materials should be submitted along with this geotechnical report.

Concrete Mix Design
Our experience indicates that the earth materials at the site contain negligible to positive levels of sulfates and should be categorized as sulfate exposure class: S0. Thus, a concrete mix design including: Type II Portland cement is recommended for the project. Also, we recommend that a low permeability concrete be utilized at the site to limit moisture transmission through slabs and foundations. For this purpose, the water/cement (*w/cm*) ratio to be used at the site should be limited to (0.50). Limited use (subject to approval of mix designs) of a water reducing agent may be included to increase workability.

The concrete should be properly cured to minimize risk of shrinkage cracking. The code dictates at least (7) days of moist curing. Two to three weeks is preferred to minimize cracking. One-inch hard rock mixes should be provided. Pea gravel mixes are specifically not recommended but could be utilized for relatively non-critical improvements (e.g., flatwork) and other improvements provided the mix designs consider limiting shrinkage.

Contractors/other designers should take care in all aspects of designing mixes, detailing, placing, finishing, and curing concrete. The mix designers and contractor are advised to consider all available steps to reduce cracking. The use of shrinkage compensating cement or fiber reinforcing should be considered. Mix designs proposed by the contractor should be considered subject to review by the project civil/structural engineer.

Pavement Design
Prior to placing new paving for planned new surface parking areas, all deleterious materials and/or existing fill materials (Af) should be removed down to expose competent native alluvial soils (Qa), anticipated at a minimum depth of (24) inches below existing site grades. The bottom excavation should be observed by the project soils engineer and/or project engineering geologist. At the discretion of the project soils engineer and/or project engineering geologist, the bottom excavation may be deepened further than the minimum (24) inches to achieve exposure of competent earth materials.

The bottom excavation should be scarified to a depth of (12) inches, moistened as required to obtain optimum moisture content, and recompacted to a **minimum of (95)-percent of the modified Proctor maximum dry density**, as determined by the most recent version of ASTM D1557. The property owner should be aware that removal of all existing fill materials in the area of new paving is not required, however, pavement constructed in this manner will most likely have a shorter design life and increased maintenance costs.

Asphalt/Concrete (A/C) Pavement
Preliminary Asphalt/Concrete (A/C) pavement sections are provided below for the assumed Traffic Indices of 5.0, 7.0, and 9.0 to be utilized in the design of any planned new surface parking areas. These A/C pavement recommendations should be confirmed with additional “R-value” testing of representative near-surface soils at the completion of grading and after underground utilities have been installed and backfilled. Final A/C pavement section designs shall be selected by the project civil engineer based upon the projected design traffic index(ices).

The following minimum A/C pavement sections are recommended:

A/C Pavement Section Recommendations			
Service Type	Traffic Index (TI)	A/C Pavement Course Thickness (inches)	Aggregate Base Course Thickness (inches)
Light Traffic Loads: Passenger Cars	5.0	3	0
Moderate Traffic Loads: Passenger Cars, Trucks & Buses	7.0	3	4
Heavy Traffic Loads: Passenger Cars, Trucks & Buses, Trash Trucks	9.0	4	6

Portland Cement Concrete (PCC) Pavement
Preliminary minimum Portland Cement Concrete (PCC) pavement sections are provided below for Traffic Indices of 5.0, 7.0, and 9.0 to be utilized in the design of any planned new surface parking areas.

These PCC pavement recommendations should be confirmed with additional "R-value" testing of representative near-surface soils at the completion of grading and after underground utilities have been installed and backfilled. Final PCC pavement section designs shall be selected by the project civil engineer based upon the projected design traffic index(ices).

The following minimum PCC pavement sections are recommended:

PCC Pavement Section Recommendations			
Service Type	Traffic Index (TI)	PCC Pavement Thickness (inches)	Aggregate Base Course Thickness (inches)
Light Traffic Loads: Passenger Cars	5.0	6	12
Moderate Traffic Loads: Passenger Cars, Trucks & Buses	7.0	9	12
Heavy Traffic Loads: Passenger Cars, Trucks & Buses, Trash Trucks	9.0	12	12

The recommended PCC pavement sections consisting of thicknesses presented above shall be placed over a minimum of (12) inches of properly compacted subgrade earth materials. The concrete should have a minimum compressive strength of (3,500) pounds per square inch at the time the pavement is subjected to traffic loads. To reduce, (but not eliminate), the potential for cracking, paving should provide control joints at regular intervals not exceeding (14) feet in each direction to a depth of (1/3) of the concrete thickness. Contraction and construction joints should include a joint filler/sealer to prevent migration of water into the subgrade earth materials. The type of joint sealer and filler material should be specified by the pavement designer and should be maintained throughout the life of the pavement. Dowels are recommended at joints to reduce potential offsets. The above section does not include steel reinforcement. Steel reinforcement, typically consisting of (#3) rebars placed at (24) inches on-center each way, may be added to reduce the potential for cracking.

The PCC pavement section thicknesses provided are minimum thicknesses. Increasing the thickness of any or all of the above layers will reduce the likelihood of the pavement experiencing distress during its service lifetime. The above recommendations are based on the assumption that proper maintenance and irrigation of the areas adjacent to the roadway will occur through the design life of the pavement. Failure to maintain a proper maintenance and/or irrigation program may jeopardize the integrity of the PCC pavement.

Aggregate Base Course
Aggregate base should be compacted to a to a minimum of (95)-percent of the modified Proctor maximum dry density, as determined by the most recent version of ASTM D1557. Base materials utilized should be a CALTRANS Class II Aggregate Base, and/or conform with Sections 200-2.2 or 200-2.4 of the *Standard Specifications for Public Works Construction*, (Green Book), current edition.

The performance of pavement is highly dependent upon providing positive surface drainage away from the edges. Ponding of water on or adjacent to pavement can result in saturation of the subgrade materials and subsequent pavement distress. If planter islands are planned, the perimeter curb should extend a minimum of (12) inches below the bottom of the aggregate base.

REVIEWS

Plan Review and Plan Notes

The final grading, building, and/or structural plans shall be reviewed and approved by the consultants to ensure that all recommendations are incorporated into the design or shown as notes on the plan.

The final plans should reflect the following:

1. This Soils Engineering Investigation by Bay City Geology, Inc. is a part of the plans.
2. Plans must be reviewed and signed by the project engineering geologist and soils engineer.
3. The project engineering geologist and soils engineer must review all grading.
4. The project engineering geologist and soils engineer shall review all foundation excavations prior to placing steel and concrete.

Construction Review

Onsite reviews will be required to verify all geotechnical work. It is required that all footing excavations, seepage pits, and grading be reviewed by this office. This office should be notified at least two working days in advance of any field reviews so that staff personnel may be made available.

The property owner should take an active role in project safety by assigning responsibility and authority to individuals qualified in appropriate construction safety principles and practices. Generally, site safety should be assigned to the general contractor or construction manager that is in control of the site and has the required expertise, which includes but not limited to construction means, methods and safety precautions.

LIMITATIONS

General

Findings, conclusions and recommendations contained in this report are based upon the surface mapping, subsurface exploration, data analyses, and specific information as described and past experience. Earth materials and conditions immediately adjacent to, or beneath those observed may have different characteristics, such as, earth type, physical properties and strength. Therefore, no representations are made as to the nature, quality, or extent of latent earth materials. Site conditions can and do change from those that were first envisioned. During construction, if subsurface conditions differ from those encountered in the described exploration, this office should be advised immediately so that appropriate action can be taken.

The scope of this investigation is limited to the project area explored as depicted on the enclosed Plot Map. This report is not a comprehensive evaluation of the entire property. This report has not been prepared for use by other parties or for other purposes and may not contain sufficient information for other than the intended use. Prior to use by others, Bay City Geology, Inc. should be consulted to determine if additional work is required. If the project is delayed more than one year, this office should be contacted to verify current site conditions and prepare an update report.

Findings, conclusions and recommendations presented herein are based on experience and background. Therefore, findings, conclusions and recommendations are professional opinions and are not meant to indicate a control of nature.

Potentially expansive soils were encountered on the subject property. Design for foundations, slabs on grade, and retaining walls have been provided to mitigate this soil condition. These designs do not guarantee or warrant that cracking will not occur.

This limited report provides information regarding the geologic findings on the subject property. It is not designed to provide a guarantee that the site will be free of hazards in the future, such as, landslides, slippage, differential settlement, debris flows, seepage, concentrated drainage or flooding. Hillside properties are subject to hazards, which are not found with flatland properties. It may not be possible to eliminate all hazards, but homeowners must maintain their property and improve deficiencies.

CONSTRUCTION NOTICE

Construction can be difficult. Recommendations contained herein are based upon surface reconnaissance and subsurface explorations deemed suitable for the scope of the project.

It is this Corporation's aim to advise you through this report of the general site conditions, suitability for construction, and overall stability. It must be understood that the opinions are based upon testing, analysis, and interpretation thereof.

Quantities for foundation concrete and steel may be estimated, based on the findings provided in this report. However, you must be aware that depths and magnitudes will most-likely vary between the explorations provided in this report.

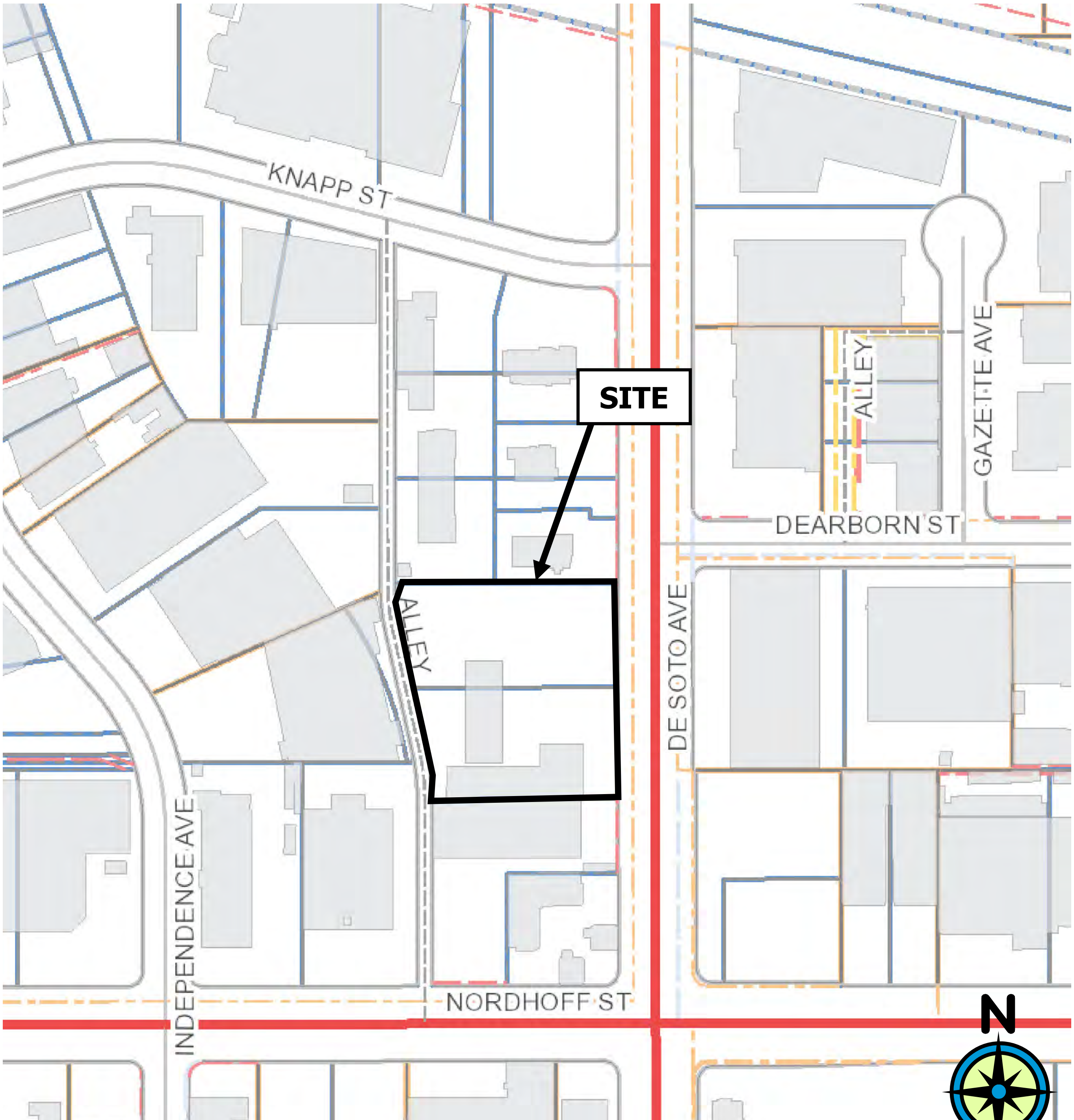
APPENDIX I

Location Maps

Plot Maps

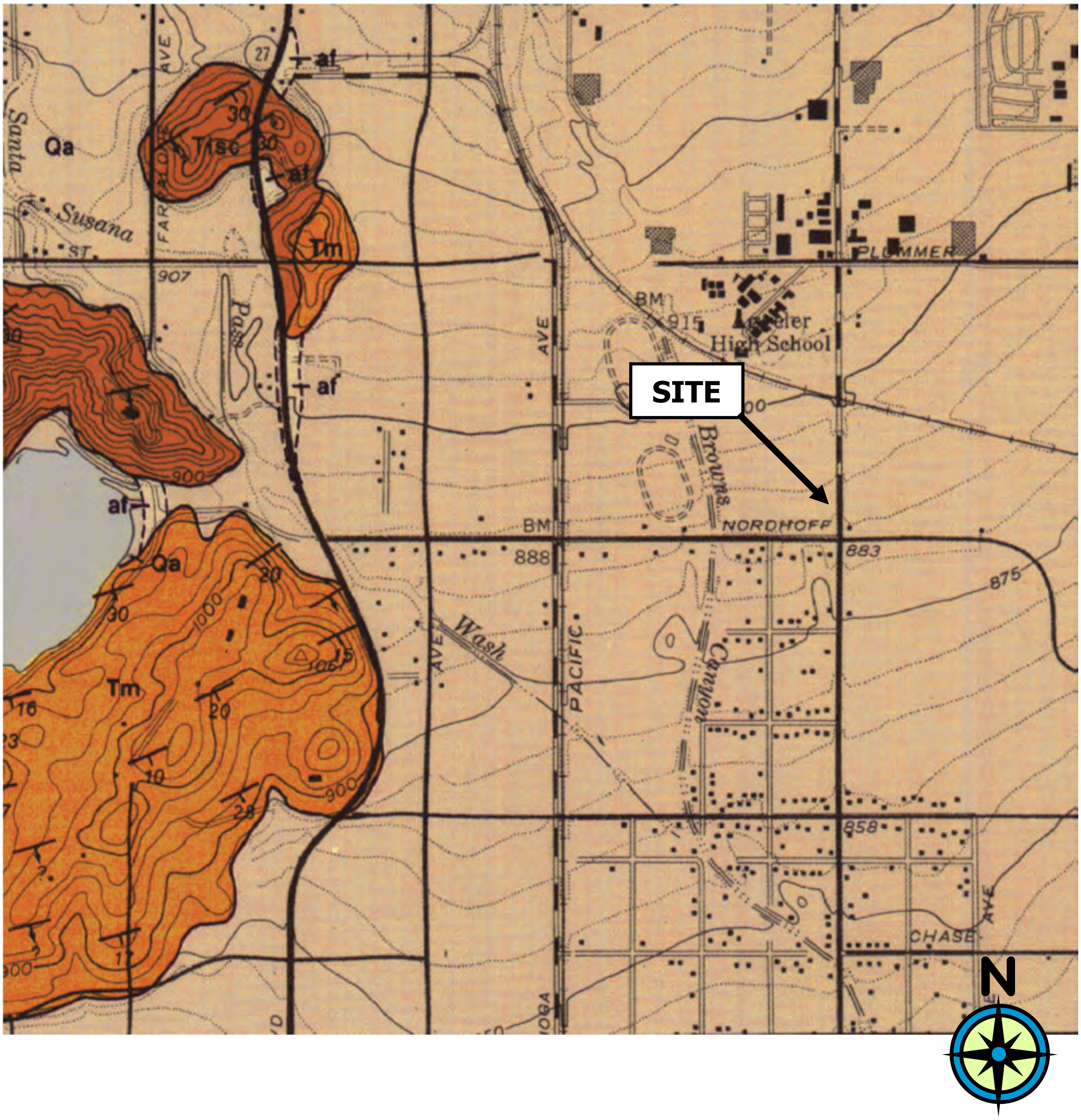
Field Exploration Summary
Exploration Logs 1 through 4

LOCATION



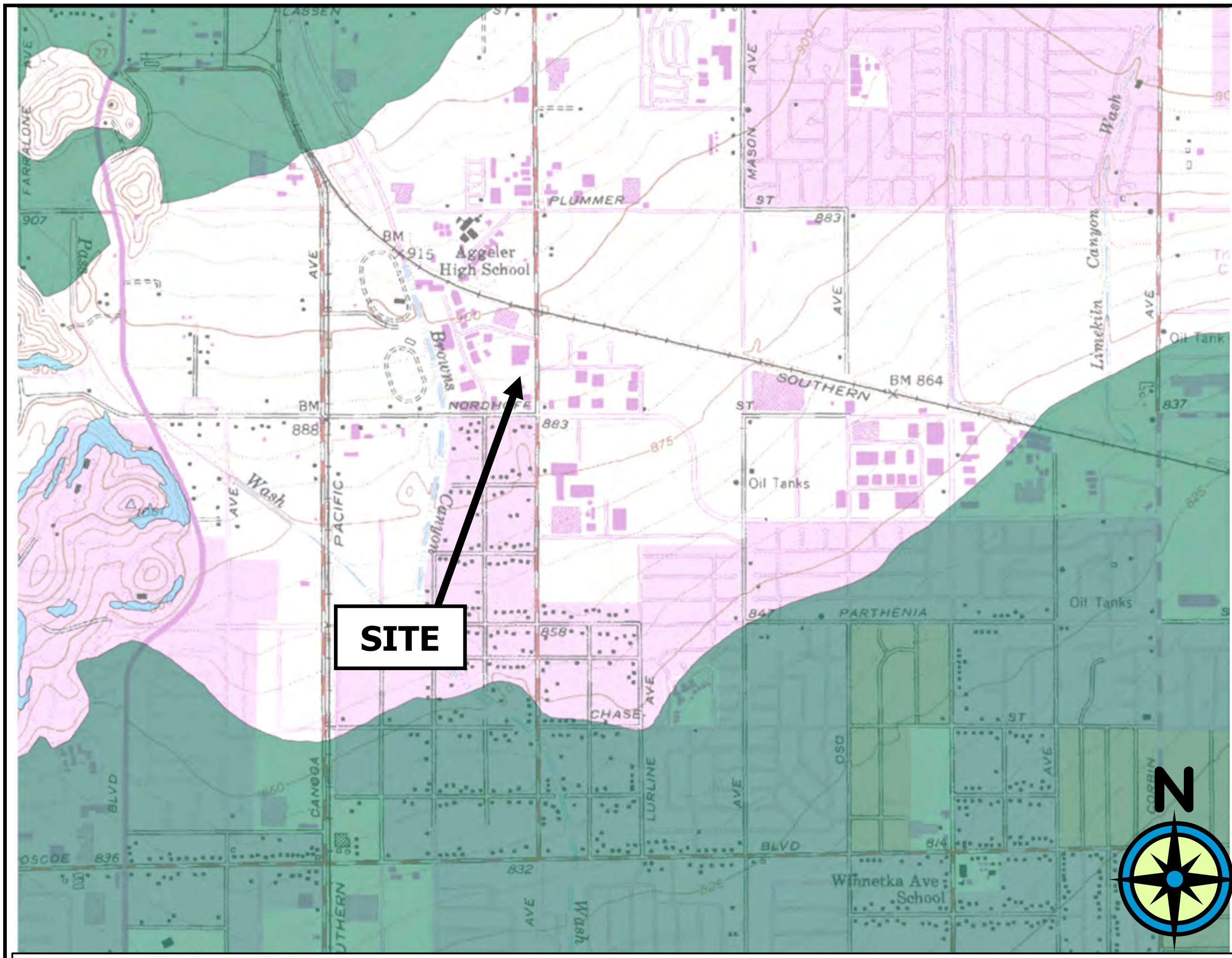
Reference:	Navigate LA (http://navigatela.lacity.org)
Project Address:	9143 De Soto Avenue Chatsworth, California Plate 1

REGIONAL GEOLOGY MAP



Reference:	Dibblee Geologic Foundation
Project Address:	9143 De Soto Avenue Chatsworth, California Plate 2

SEISMIC HAZARD MAP



MAP EXPLANATION

EARTHQUAKE FAULT ZONES

Earthquake Fault Zones
Zone boundaries are delineated by straight-line segments; the boundaries define the zone encompassing active faults that constitute a potential hazard to structures from surface faulting or fault creep such that avoidance as described in Public Resources Code Section 2621.5(a) would be required.

1906 C
? ?

Active Fault Traces
Faults considered to have been active during Holocene time and to have potential for surface rupture: Solid Line in Black or Red where Accurately Located; Long Dash in Black or Solid Line in Purple where Approximately Located; Short Dash in Black or Solid Line in Orange where Inferred; Dotted Line in Black or Solid Line in Rose where Concealed; Query (?) indicates additional uncertainty. Evidence of historic offset indicated by year of earthquake-associated event or C for displacement caused by fault creep.

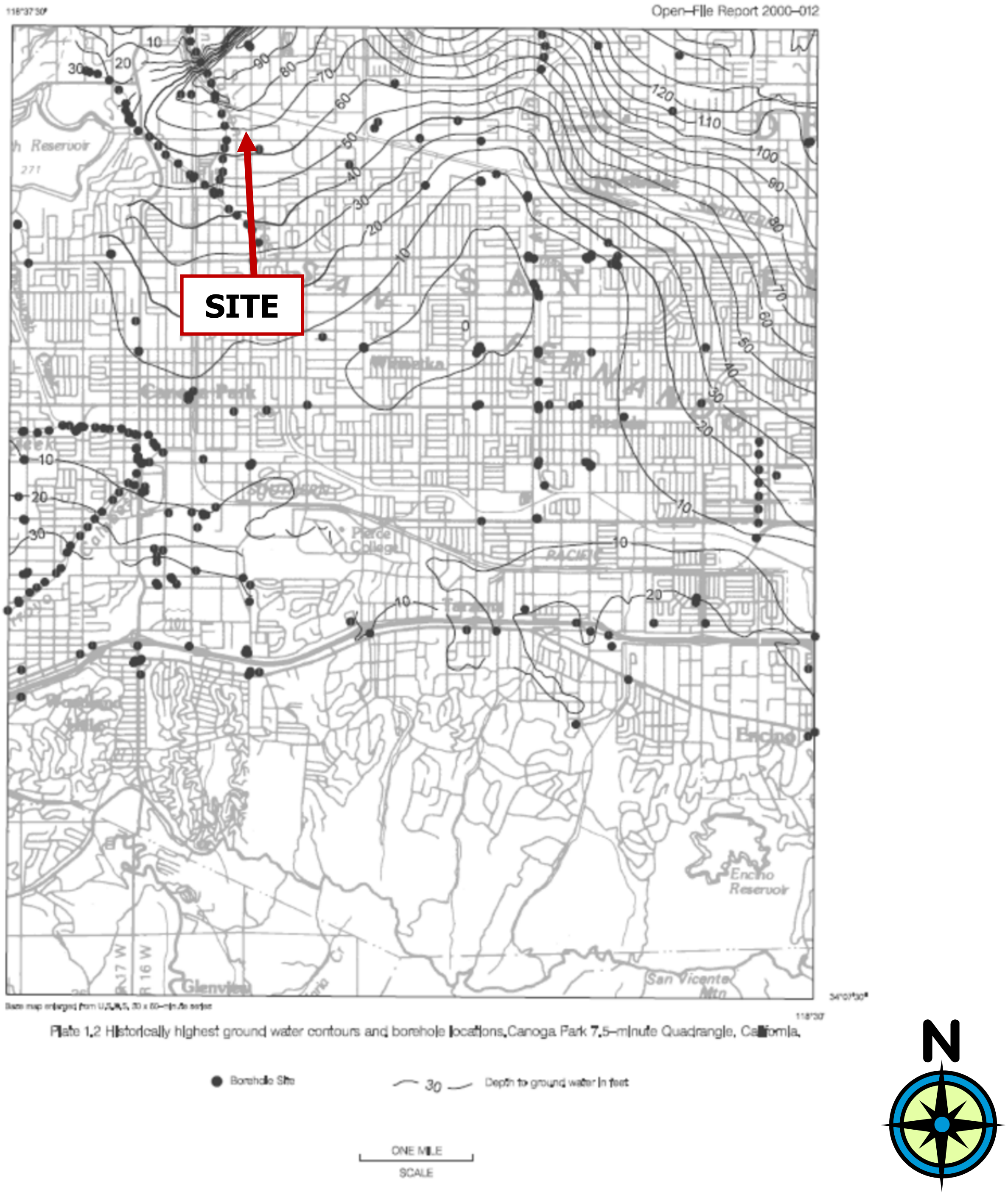
SEISMIC HAZARD ZONES

Liquefaction Zones
Areas where historical occurrence of liquefaction, or local geological, geotechnical and ground water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

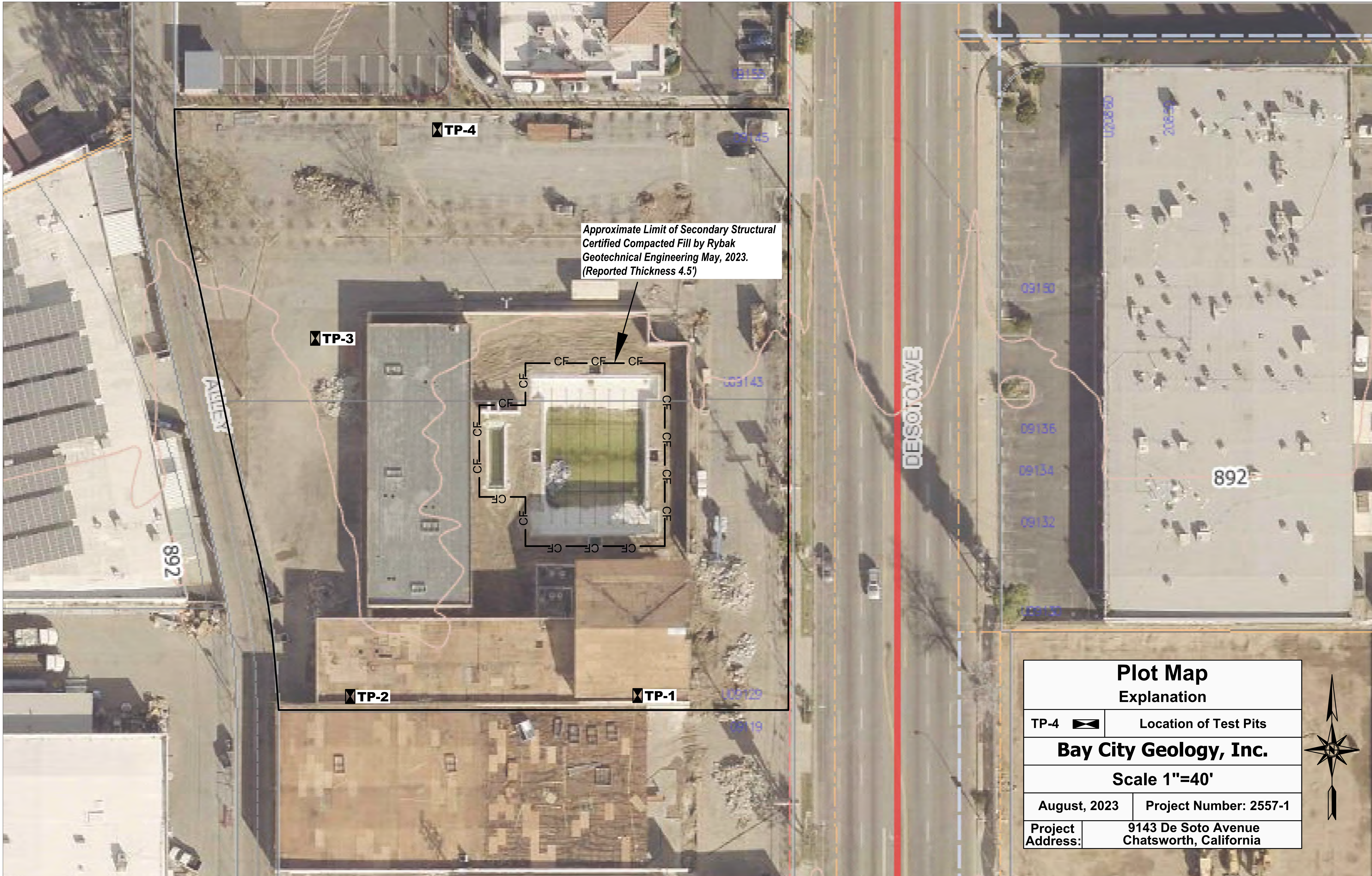
Earthquake-Induced Landslide Zones
Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

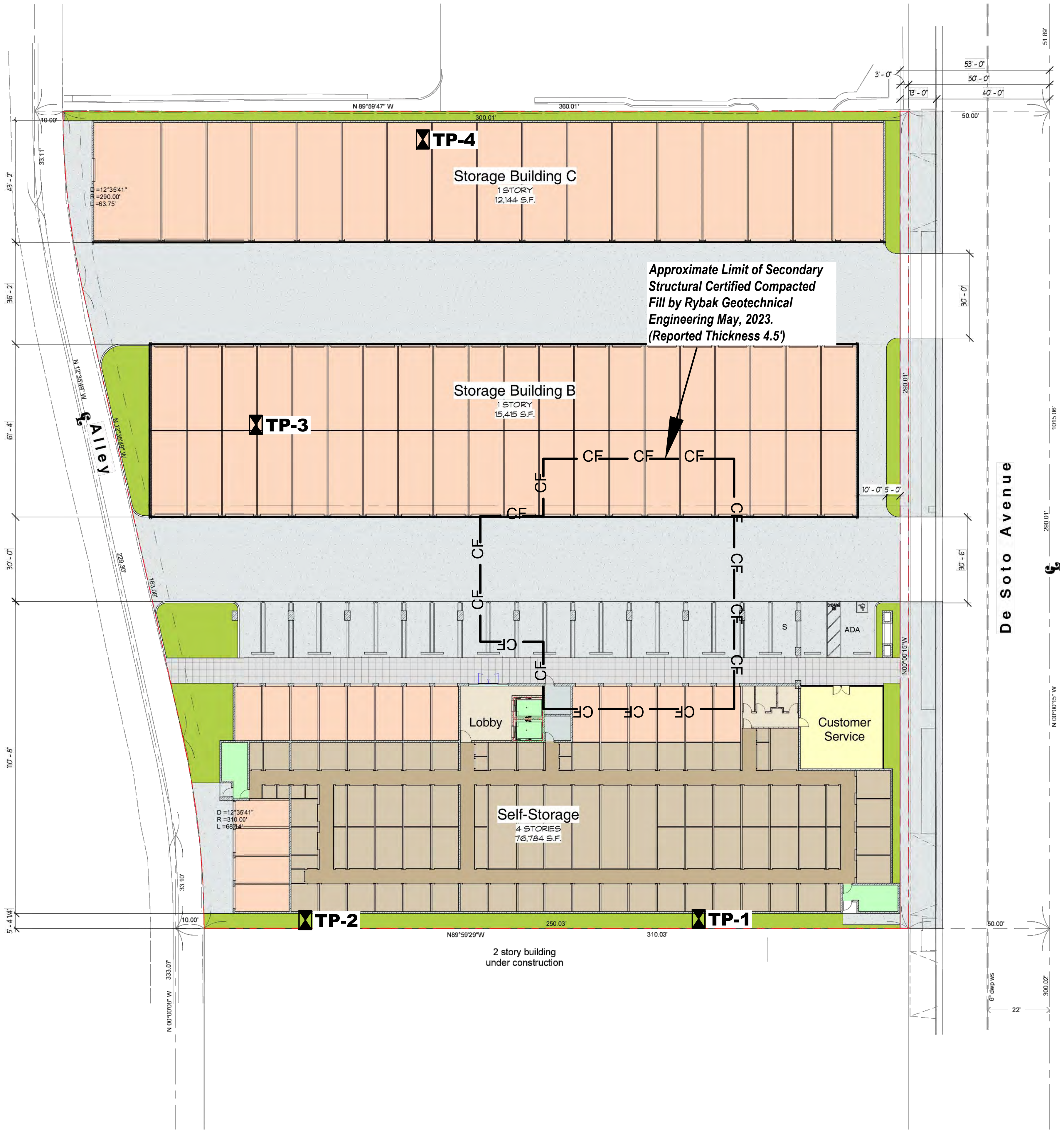
Reference:	State of California EZRIM of the Canoga Park Quadrangle
Project Address:	9143 De Soto Avenue Chatsworth, California — Plate 3

HISTORIC HIGH GROUNDWATER



Reference:	State of California Seismic Hazard Report of the Canoga Park Quadrangle
Project Address:	9143 De Soto Avenue Chatsworth, California Plate 4


TP-4
TP-3
TP-2
TP-1
Approximate Limit of Secondary Structural Certified Compacted Fill by Rybak Geotechnical Engineering May, 2023. (Reported Thickness 4.5')
CF
ALLEY
DE SOTO AVE
892
Plot Map
Explanation
TP-4
Location of Test Pits
Bay City Geology, Inc.
Scale 1"=40'
August, 2023
Project Number: 2557-1
Project Address:
9143 De Soto Avenue
Chatsworth, California



Plot Map

Explanation

TP-4	Location of Test Pits

Bay City Geology, Inc.

Scale 1"=40'

August, 2023	Project Number: 2557-1
Project Address:	9143 De Soto Avenue Chatsworth, California

Building Areas - Self-Storage

Building A	Zoning Code Area							LABC Area					
	Cust Serv	Caretaker Unit	Toilet & Janitor	Loading Lobbies	Dir Access Storage	Int Access Storage	Zoning Subtotal	Stairs & Elevators	Mech	Display Window	Covered Loading	LABC Total	Ext Walls & Attic
Ground Floor	860		278	391	3,469	10,875	15,874	481	174		6,862	23,391	2,440
Second Floor		860				15,772	16,633	529				17,161	1,957
Third Floor						24,384	24,384	529		249		25,161	905
Fourth Floor						24,384	24,384	529		249		25,161	905
Building A Totals	**860**	**860**	**278**	**391**	**3,469**	**75,415**	**81,274**	**2,067**	**174**	**497**	**6,862**	**90,874**	**6,208**

Target: 60,489 Net Rentable Area

Single Story Bldgs	Zoning Code Area							LABC Area					
					Dir Access Storage		Zoning Subtotal					LABC Total	Ext Walls
Building B					15,415		15,415					15,415	0
Building C					12,144		12,144					12,144	0
Single Story Bldgs	**0**		**0**	**0**	**27,559**	**0**	**27,559**	**0**	**0**	**0**	**0**	**27,559**	**0**

Site Statistics

	Provided		Required/Allowed	
Total Gross Site Area (ALTA Survey)	97,155 s.f.			
Total Net Site Area (ALTA Survey)	79,705 s.f.			
Floor Area Ratio Gross	108,833 s.f.	1.12	145,733 s.f.	1.50
Floor Area Ratio Net	108,833 s.f.	1.37	119,557 s.f.	1.50

Parking Analysis **Distance to Nordoff/G-Line Station** **936 feet**

Vehicluar Parking Required - None Required per AB 2097

Parking Provided	
Full Size	21 spaces
Disabled Access	1 spaces
Provided Parking	**22 spaces**

Field Exploration Summary

A field exploration of the site was conducted on August 2, 2023. The geotechnical conditions were mapped by a representative of this office (refer to the Plot Map & Exploration Logs). Subsurface exploration was performed by manually trenching into the underlying earth materials. Explorations were excavated to a maximum depth of (9) feet below adjacent site grades. The Plot Map(s) in Appendix I depict(s) the locations of the subsurface explorations. The explorations were logged by the engineering geologist using both visual and tactile methods.

Representative undisturbed and bulk samples of the on-site earth materials were obtained from the explorations. Hand samples taken from test pits and/or hand-auger explorations were obtained using a (6) inch long brass ring lined, steel barrel hand-sampler that is driven with a slide-safety hammer. The soil is retained in the brass rings of (2½) inches in diameter and (1) inch in height. The samples are transported in moisture tight containers. Locations of earth material samples are indicated on the Exploration Logs.

Project Address: *9143 De Soto Avenue* **Date Logged:** *08/02/23*

Project Number: *2557* **Logged By:** *J. Miller*

EXPLORATION: TP-1

DESCRIPTION

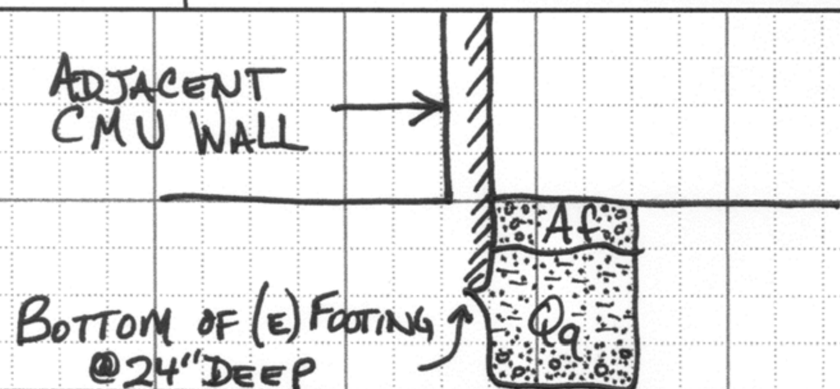
0.0 - 1.0' FILL; Af, silty sand and gravel, light-gray, slightly moist, medium-dense.

1.0 - 4.0' QUATERNARY ALLUVIUM; Qa, silty sand, medium-brown, slightly moist, dense, fine-grained.

@3.0' sand and gravel, fine- to medium-grained, rounded pebbles up to 2-inches in length.

GRAPHIC PROFILE

Scale: 1"=4'



Project Address: *9143 De Soto Avenue* **Date Logged:** *08/02/23*
Project Number: *2557* **Logged By:** *J. Miller*

EXPLORATION: TP-2

DESCRIPTION

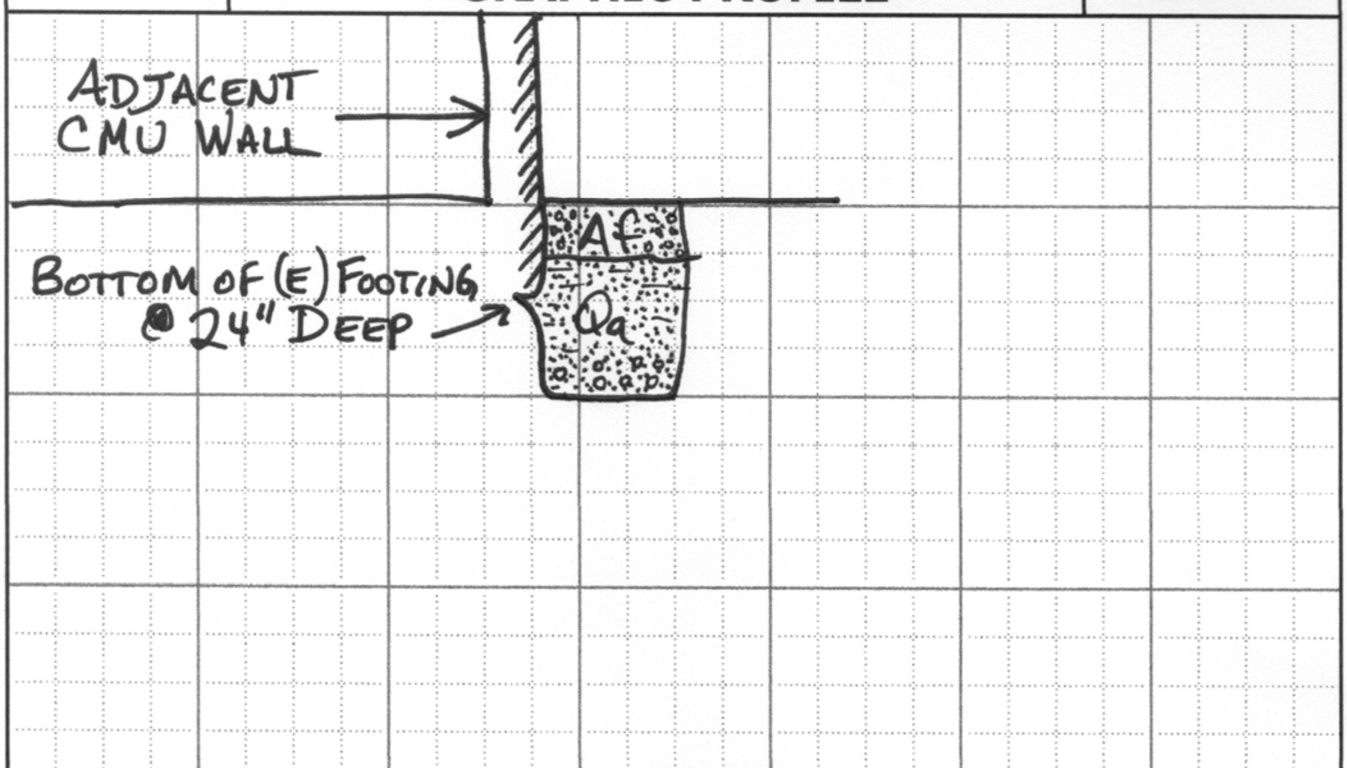
0.0 - 1.0' FILL; Af, silty sand and gravel, light-gray, slightly moist, medium-dense.

1.0 - 4.0' QUATERNARY ALLUVIUM; Qa, silty sand, medium-brown, slightly moist, dense, fine-grained.

@3.0' sand and gravel, fine- to medium-grained, rounded pebbles up to 2-inches in length.

GRAPHIC PROFILE

Scale: 1"=4'



Project Address: *9143 De Soto Avenue* **Date Logged:** *08/02/23*

Project Number: *2557* **Logged By:** *J. Miller*

EXPLORATION: TP-3

DESCRIPTION

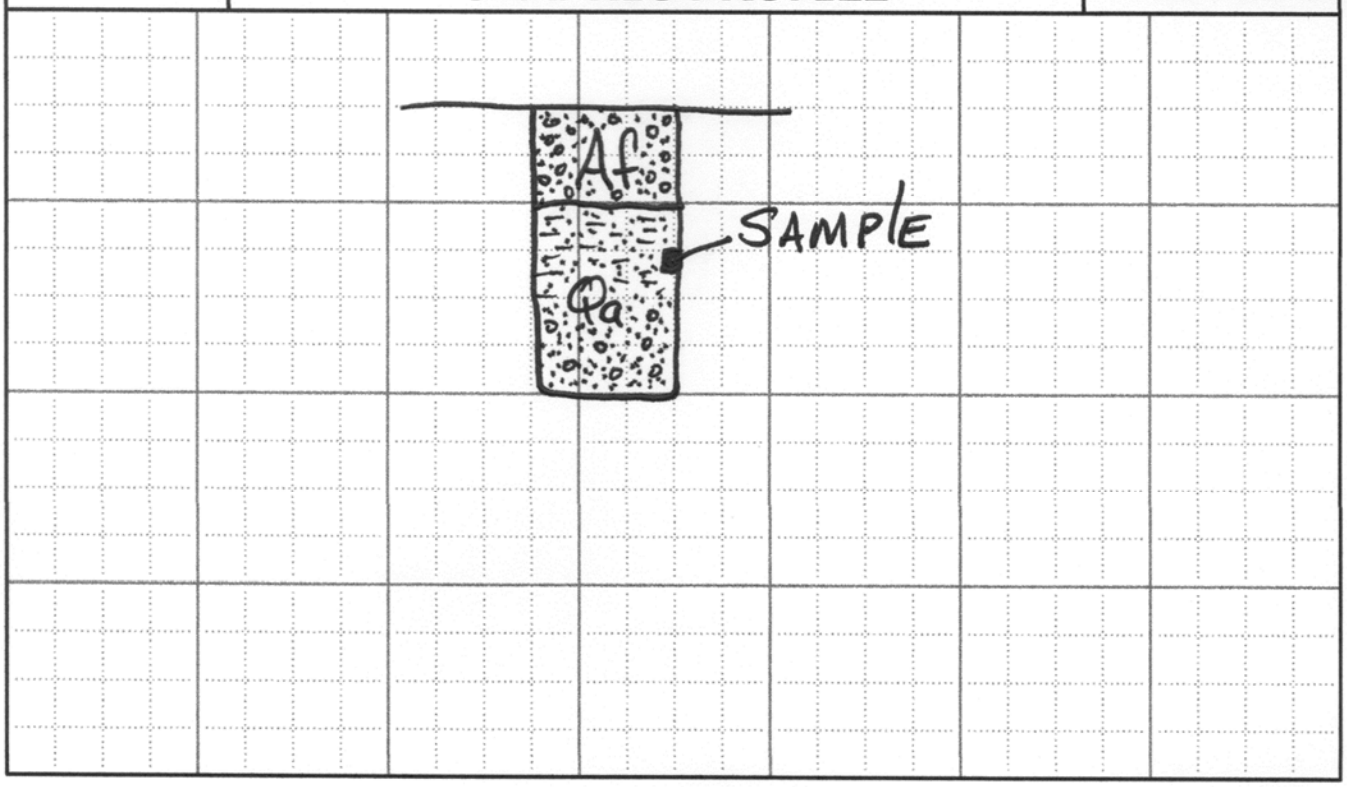
0.0 - 2.0' FILL; Af, silty sand and gravel, light-gray, slightly moist, medium-dense.

2.0 - 6.0' QUATERNARY ALLUVIUM; Qa, silty sand, medium-brown, slightly moist, dense, fine-grained.

@4.0' sand and gravel, fine- to medium-grained, rounded pebbles up to 2-inches in length.

GRAPHIC PROFILE

Scale: 1"=4'



Project Address: *9143 De Soto Avenue*

Project Number: *2557*

Date Logged: *08/02/23*

Logged By: *J. Miller*

EXPLORATION: TP-4

DESCRIPTION

0.0 - 1.0' FILL; Af, silty sand and gravel, light-gray, slightly moist, medium-dense.

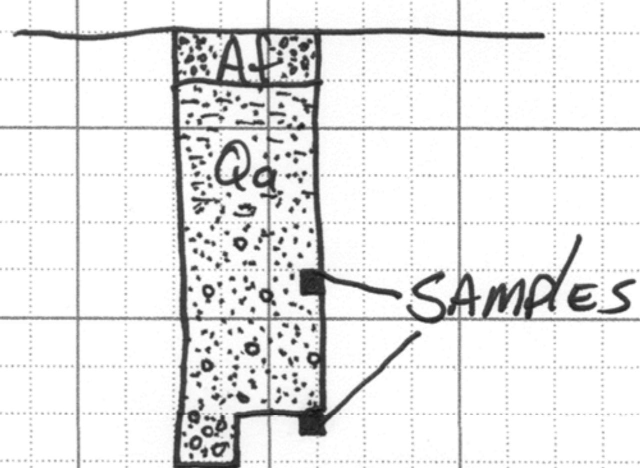
1.0 - 9.0' QUATERNARY ALLUVIUM; Qa, silty sand, medium-brown, slightly moist, dense, fine-grained.

@4.0' sand, fine- to medium-grained, few rounded pebbles up to 2-inches in length.

@8.0' increase in pebbles.

GRAPHIC PROFILE

Scale: 1"=4'



APPENDIX II - LABORATORY TESTING

Laboratory testing was performed on representative samples obtained during our field exploration. Samples were tested for the purpose of estimating material properties for use in subsequent engineering evaluations.

The physical properties of the earth materials were tested at Advanced Materials Testing, LLC (AMT), a City of Los Angeles Approved testing laboratory. In accordance with the 2023 Los Angeles Building Code (LABC) §91.7008.5, we, the undersigned geologist and engineer, have reviewed, concur with, and accept professional responsibility for use of all the laboratory testing data and results provided in the enclosed AMT laboratory testing report.

Laboratory testing was performed on samples obtained as outlined in Appendix I. All samples were sent to the laboratory for examination, testing, and classification using the Unified Soil Classification System (USCS).

AMT

Advanced Materials Testing, LLC

August 7, 2023 Project 2557

Mr. Jonathan Miller
Bay City Geology, Inc.
24736 Calvert Street
Woodland Hills, CA 91367

Subject: **RESULTS OF LABORATORY TESTING**
9143 De Soto Avenue
Los Angeles, California

Mr. Miller:

Pursuant to your request, this is a letter to certify that Advanced Materials Testing, LLC has performed laboratory soil tests for the subject project under the supervision of the undersigned engineer. Services performed by this facility were conducted in a manner consistent with that level of care and skill ordinarily exercised by members of the profession currently practicing in the same locality under similar conditions. No other warranties are expressed nor implied. Interpretation of the laboratory test results and applications of the results on the design and construction of the project are beyond the scope of our services.

Moisture and Density Tests

The dry unit weight and moisture content of the undisturbed samples were determined. The results are tabulated in the Laboratory Recapitulation - Table 1.

Shear Tests

Direct single-shear tests were performed with a direct shear machine. The desired normal load is applied to the specimen and allowed to come to equilibrium. The rate of deflection on the sample is approximately 0.005 inches per minute. The samples are tested at higher and/or lower normal loads in order to determine the angle of internal friction and the cohesion. The results are plotted on the Shear Test Diagrams and the results tabulated in the Laboratory Recapitulation - Table 1. The samples were observed prior to and after shearing to ensure the particle size of the sample did not exceed 10% of the diameter of the test specimen in accordance with ASTM standards. Although the soil was described to include gravels they were not included within the samples tested, therefore, the results provide a conservative estimate of the shear strength of the soil.

Consolidation

Consolidation tests were performed on samples, within the brass ring, to predict the soils behavior under a specific load. Porous stones are placed in contact with top and bottom of the samples to permit to allow the addition or release of water. Loads are applied in several increments and the results are recorded at selected time intervals. Samples are tested at field and increased moisture content. The results are plotted on the Consolidation Test Curve and the load at which the water is added as noted on the drawing.

Moisture-Density Relation (Maximum Density Test)

Compaction tests are performed on representative samples to determine compaction characteristics. A sample at a selected water content is placed in five layers in a mold. Each layer is uniformly compacted with a hammer. The resulting dry density is determined. A sufficient number of samples are tested at varying moisture contents to determine the moisture-density relation. The test results are shown in the Laboratory Recapitulation - Table 1.

Should you have any questions regarding these laboratory test results, please do not hesitate to contact the undersigned at your convenience.

Respectfully submitted,
Advanced Materials Testing, LLC

Raymond Haddad
President
RMH-1
Attachments: Laboratory Test Results

PROJECT NO.: 2557

PROJECT ADDRESS: 9143 De Soto Avenue

LABORATORY RECAPITULATION 1

Explorations	Depth (ft)	Material	Dry Density (p.c.f.)	Moisture Content (%)
TP-3	3.0	Qa	111	6
TP-4	5.0	Qa	111	15
	8.0	Qa	112	11

Maximum Dry Density & Optimum Moisture (D-1557)

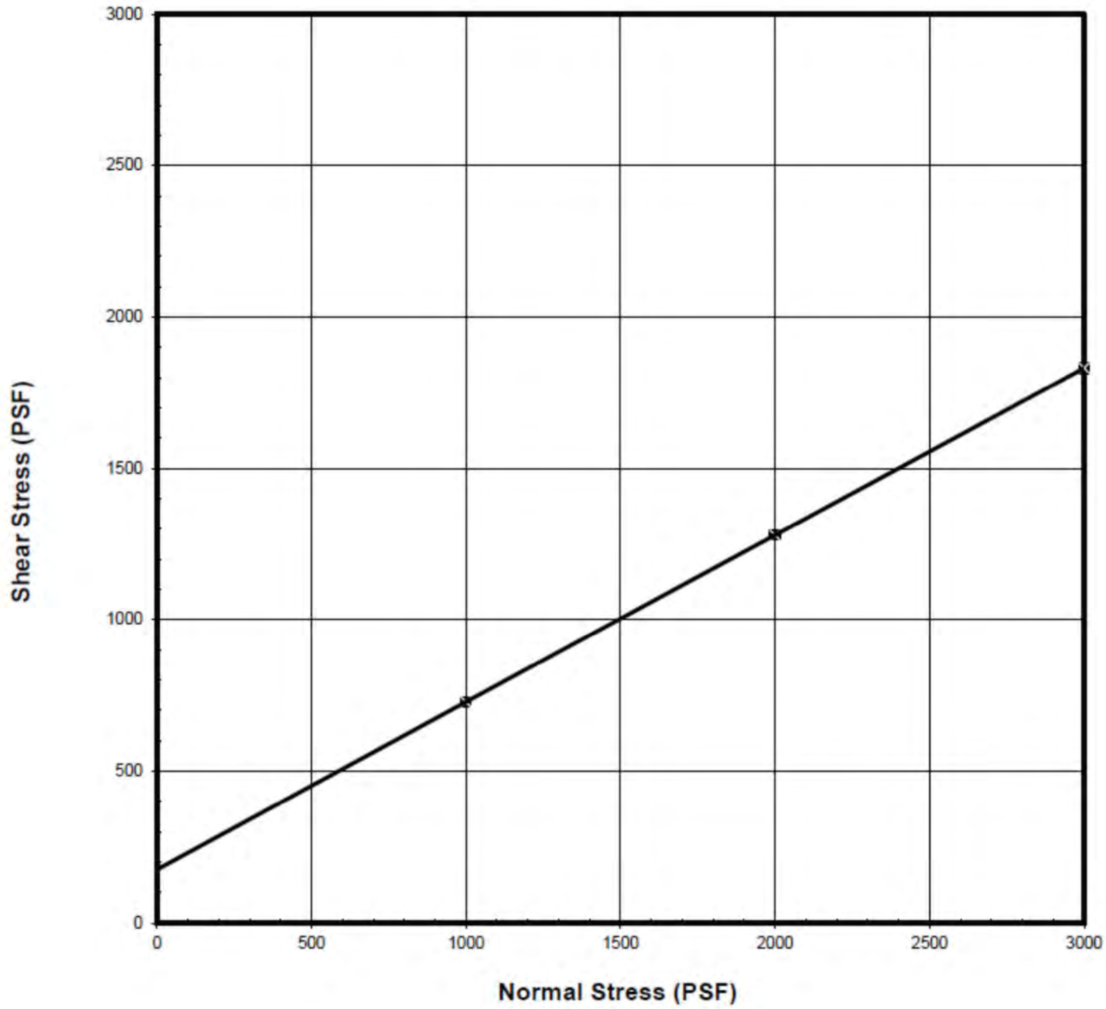
Description	Maximum Density (pcf)	Optimum Moisture (%)
Bulk	117.4	8.2

PLATE: S-1
P.N. 2557

Direct Shear Test Diagram (D-3080)

Sample Description	Sample Identification	Test Type	Sample Test State	Number of Passes
Qa	TP-3 @ 3.0'	Ultimate	Saturated	1

Soil Dry Density (PCF)	111	Shear Strength Values:	
Soil Moisture Content (%)	18	Phi (Degrees)	28.9
Soil Saturation (%)	97.4	Cohesion (PSF)	175.0

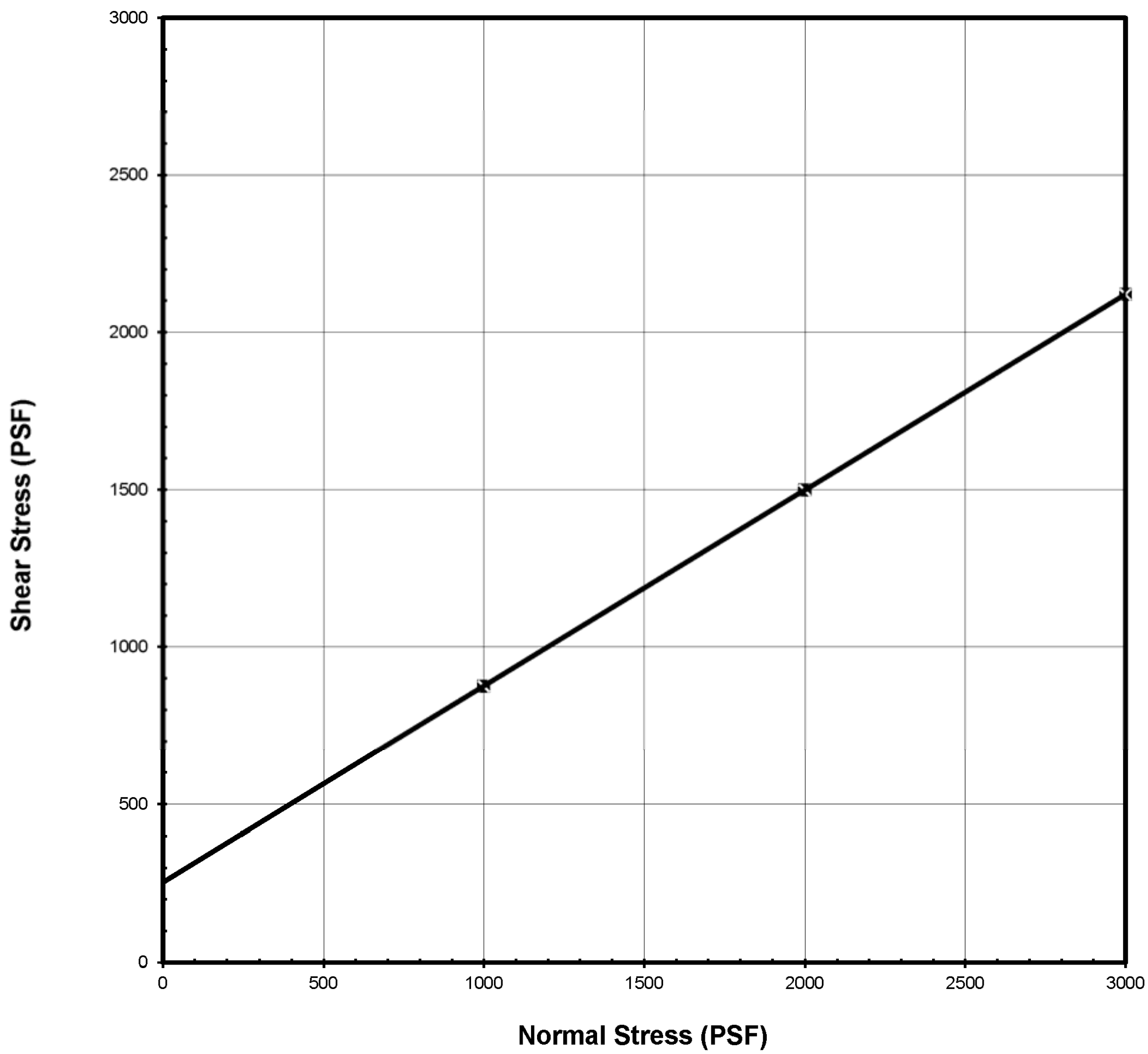


Direct Shear Test Diagram (D-3080)

PLATE: S-2
P.N. 2557

Sample Description	Sample Identification	Test Type	Sample Test State	Number of Passes
Remolded @ 90%	TP-4 @ Bulk	Ultimate	Saturated	1

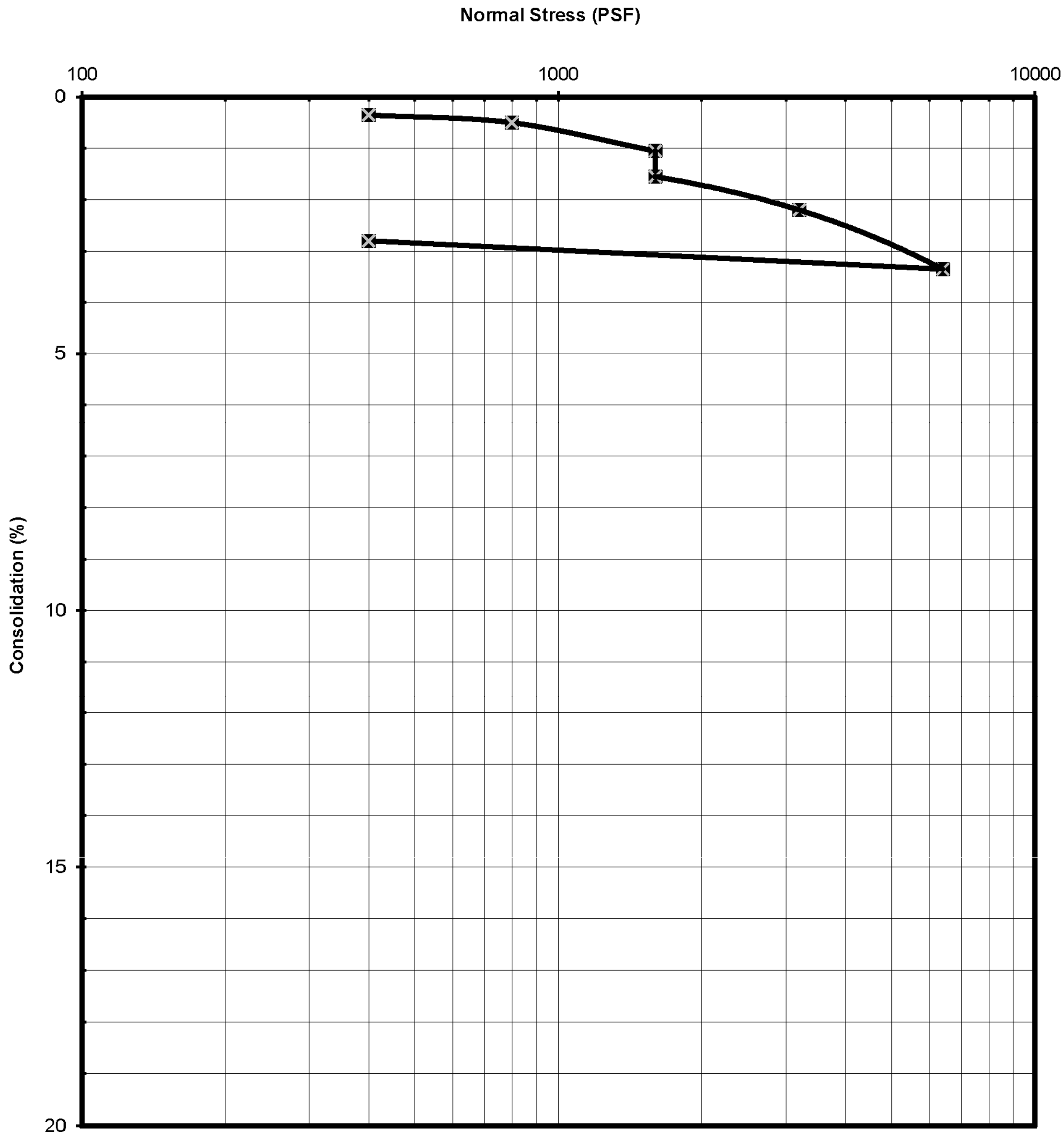
Soil Dry Density (PCF)	106	Shear Strength Values:	
Soil Moisture Content (%)	21	Phi (Degrees)	31.9
Soil Saturation (%)	99.4	Cohesion (PSF)	253.7



Consolidation Pressure Curve (D-2435)

Sample Identification	Sample Description
TP-4 @ 5.0'	Qa

PLATE: C-1
P.N. 2557



APPENDIX III

ENGINEERING ANALYSIS

Foundation Settlement Calculations

Infiltration Procedure and Calculations

Temporary Excavation Calculations

Seismic Design Considerations

Bay City Geology, Inc.

Project: De Soto Av
File No.: 2557

Settlement Calculation - Column Footing

Description: 4' - 3" Square Pad Footing
Gridline:

Soil Unit Weight	130.0	pcf	Column	Footing
Bearing Value	3400	psf	61.41	kips
Depth of Footing	2.00	feet		
Width of Footing	4.25	feet		

* Influence Values are based on Westergaard's Analyses

Depth Below Ground Surface (feet)	Average Depth Below Ground Surface (feet)	Average Depth Below Foundation (feet)	Ratio of Foundation vs. Depth (a/z)	Influence Value	Foundation Influence Pressure (psf)	Natural Soil Pressure (psf)	Total Pressure (psf)	Consolidation Curve Used	Percent Strain [Total] (%)	Percent Strain [Natural] (%)	Percent Strain [Net] (%)	Thickness of Depth Increment (feet)	Net Settlement (inches)
5.0													
	6.0	4.0	1.1	22%	736	780	1516	TP4 @ 5'	0.99	0.51	0.48	5.0	0.288
7.0													
	8.0	6.0	0.7	13%	435	1040	1475	TP4 @ 5'	0.98	0.68	0.30	2.0	0.072
9.0													
	10.0	8.0	0.5	7%	251	1300	1551	TP4 @ 5'	1.00	0.83	0.17	2.0	0.041
11.0													
	12.0	10.0	0.4	5%	162	1560	1722	TP4 @ 5'	1.10	1.04	0.06	2.0	0.014
13.0													
	14.0	12.0	0.4	3%	103	1820	1923	TP4 @ 5'	1.68	1.62	0.06	2.0	0.014
15.0													
	16.0	14.0	0.3	3%	103	2080	2183	TP4 @ 5'	1.82	1.78	0.04	2.0	0.010
17.0													
	18.0	16.0	0.3	1%	43	2340	2383	TP4 @ 5'	1.88	1.85	0.03	2.0	0.007
19.0													
	20.0	18.0	0.2	1%	43	2600	2643	TP4 @ 5'	1.99	1.97	0.02	2.0	0.005
21.0													
	22.0	20.0	0.2	1%	43	2860	2903	TP4 @ 5'	2.10	2.08	0.02	2.0	0.005
23.0													
	24.0	22.0	0.2	1%	22	3120	3142	TP4 @ 5'	2.18	2.17	0.01	2.0	0.002
25.0													
	26.0	24.0	0.2	1%	22	3380	3402	TP4 @ 5'	2.30	2.29	0.01	2.0	0.002
27.0													
	28.0	26.0	0.2	1%	22	3640	3662	TP4 @ 5'	2.39	2.38	0.01	2.0	0.002
29.0													
	30.0	28.0	0.2	1%	22	3900	3922	TP4 @ 5'	2.43	2.43	0.00	2.0	0.000
31.0													
	32.0	30.0	0.1	1%	22	4160	4182	TP4 @ 5'	2.55	2.55	0.00	2.0	0.000
33.0													

REFERENCE: Sowers, G. F. (1979) Introductory Soil Mechanics and Foundations: Geotechnical Engineering.
4th ed. Prentice Hall: New York, NY.

Total Settlement (in):	0.463
Differential Settlement (in):	0.309
-- to --	0.232

Bay City Geology, Inc.

Project: De Soto Av
File No.: 2557

Settlement Calculation - Strip Footing

Description: 24" Deep x 36" Wide Strip Footing
Gridline:

Soil Unit Weight	130.0	pcf	Strip	Footing
Bearing Value	2400.0	psf	7.20	kips/ft
Depth of Footing	2.00	feet		
Width of Footing	3.00	feet		

* Influence Values are based on Westergaard's Analyses

Depth Below Ground Surface (feet)	Average Depth Below Ground Surface (feet)	Average Depth Below Foundation (feet)	Ratio of Foundation vs. Depth (b/z)	Influence Value	Foundation Influence Pressure (psf)	Natural Soil Pressure (psf)	Total Pressure (psf)	Consolidation Curve Used	Percent Strain [Total] (%)	Percent Strain [Natural] (%)	Percent Strain [Net] (%)	Thickness of Depth Increment (feet)	Net Settlement (inches)
5.0													
	6.0	4.0	0.8	29%	698	780	1478	TP4 @ 5'	0.96	0.51	0.45	5.0	0.270
7.0													
	8.0	6.0	0.5	22%	518	1040	1558	TP4 @ 5'	1.00	0.68	0.32	2.0	0.077
9.0													
	10.0	8.0	0.4	13%	315	1300	1615	TP4 @ 5'	1.06	0.83	0.23	2.0	0.055
11.0													
	12.0	10.0	0.3	13%	315	1560	1875	TP4 @ 5'	1.15	1.04	0.11	2.0	0.026
13.0													
	14.0	12.0	0.3	9%	214	1820	2034	TP4 @ 5'	1.75	1.62	0.13	2.0	0.031
15.0													
	16.0	14.0	0.2	9%	214	2080	2294	TP4 @ 5'	1.85	1.78	0.07	2.0	0.017
17.0													
	18.0	16.0	0.2	5%	108	2340	2448	TP4 @ 5'	1.88	1.85	0.03	2.0	0.007
19.0													
	20.0	18.0	0.2	5%	108	2600	2708	TP4 @ 5'	2.00	1.97	0.03	2.0	0.007
21.0													
	22.0	20.0	0.2	5%	108	2860	2968	TP4 @ 5'	2.09	2.08	0.01	2.0	0.002
23.0													
	24.0	22.0	0.1	5%	108	3120	3228	TP4 @ 5'	2.18	2.17	0.01	2.0	0.002
25.0													
	26.0	24.0	0.1	5%	108	3380	3488	TP4 @ 5'	2.30	2.29	0.01	2.0	0.002
27.0													
	28.0	26.0	0.1	5%	108	3640	3748	TP4 @ 5'	2.39	2.38	0.01	2.0	0.002
29.0													
	30.0	28.0	0.1	5%	108	3900	4008	TP4 @ 5'	2.43	2.43	0.00	2.0	0.000
31.0													
	32.0	30.0	0.1	5%	108	4160	4268	TP4 @ 5'	2.55	2.55	0.00	2.0	0.000
33.0													

REFERENCE: Sowers, G. F. (1979) Introductory Soil Mechanics and Foundations: Geotechnical Engineering. 4th ed. Prentice Hall: New York, NY.

Total Settlement (in):	0.500
Differential Settlement (in):	0.334
-- to --	0.250

Infiltration Procedure & Calculations

Percolation testing was conducted on (1) of the onsite test pit explorations. A (1) foot cube was excavated at the bottom of the exploration at (8) feet below grade. The cube was presoaked with water prior to running the percolation test. The percolation test consisted of filling the cube with water and drop measurements were conducted at 30-minute intervals. The test was repeated until a stabilized rate was obtained. The reduction factor was calculated to determine the infiltration rate. The lowest observed percolation rate of (2.35) inches per hour should be utilized for design of the SUSMP.

Test Pit 4 Percolation Test Data

Reading Number	Time Start/End	Elapsed Time	Water Drop (inches)	Percolation Rate (in/hr)
1	08:15 08:45	30	12	24
2	08:45 09:15	30	9	18
3	09:15 09:45	30	5	10
4	09:45 10:15	30	5	10
5	10:15 10:45	30	4	8
6	10:45 11:15	30	3	6
7	11:15 11:45	30	3	6
8	11:45 12:15	30	3	6

$$Rf = \left[\frac{2d1-\Delta d}{DIA}\right]+1$$

Reduction Factor (R_f)
2.56

d1= initial water depth (in.)
Δd = water level drop of stablized level (in.)
DIA = 13.5 in. (equivalent diameter of 1-foot cube)

$$\text{Infiltration Rate} = \frac{\text{Stablized Level }(^{in}/_{hr})}{\text{Reduction Factor }(Rf)} \qquad \frac{6}{2.56}$$

Infiltration Rate (in/hr) = 2.35

Bay City Geology, Inc.

SLOT CUT CALCULATION

File No.: **2557**
Project: **De Soto Av**

ASSUME: Grading Over-Excavations

CALCULATE THE FACTOR OF SAFETY OF SLOT CUT EXCAVATIONS. ASSUME COHESIVE AND FRICTIONAL RESISTANCE ALONG THE SIDES OF SLOTS AS WELL AS THE FAILURE SURFACE. THE HORIZONTAL PRESSURE ON THE SIDES OF THE SLOTS IS THE AT-REST PRESSURE (1-SIN(phi)).

CALCULATION PARAMETERS

EARTH MATERIAL:	Alluvium (SM)		EXCAVATION HEIGHT:	5	feet
SHEAR DIAGRAM:	Lowest Ult. Vals.		BACKSLOPE ANGLE:	0	degrees
COHESION:	175	psf	SURCHARGE:	0	pounds
PHI ANGLE:	28.9	degrees	SURCHARGE TYPE:	U	Uniform
DENSITY:	131	pcf	INITIAL FAILURE ANGLE:	10	degrees
SLOT BOUNDARY CONDITIONS			FINAL FAILURE ANGLE:	80	degrees
SLOT CUT WIDTH:	8	feet	INITIAL TENSION CRACK:	1	feet
COHESION:	87.5	psf	FINAL TENSION CRACK:	10	feet
PHI ANGLE:	14.45	degrees			

Run Calculation

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	57	degrees
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	1.0	feet
DEPTH OF TENSION CRACK	3.5	feet
TOTAL EXTERNAL SURCHARGE	0.0	pounds
VOLUME OF FAILURE WEDGE	33.8	ft^3
WEIGHT OF FAILURE WEDGE	4433.1	pounds
LENGTH OF FAILURE PLANE	1.8	feet
SURFACE AREA OF FAILURE PLANE	15	ft^2
SURFACE AREA OF SIDES OF SLOTS	4.2	ft^2
NUMBER OF TRIAL WEDGES ANALYZED	8928	trials
TOTAL RESISTING FORCE ALONG WEDGE BASE (FrB)	2024.0	pounds
TOTAL RESISTING FORCE ALONG WEDGE SIDES (FrS)	650.5	pounds
RESULTANT HORIZONTAL COMPONENT OF FORCE	**-6.7**	**pounds**
CALCULATED FACTOR OF SAFETY	**1.27**	

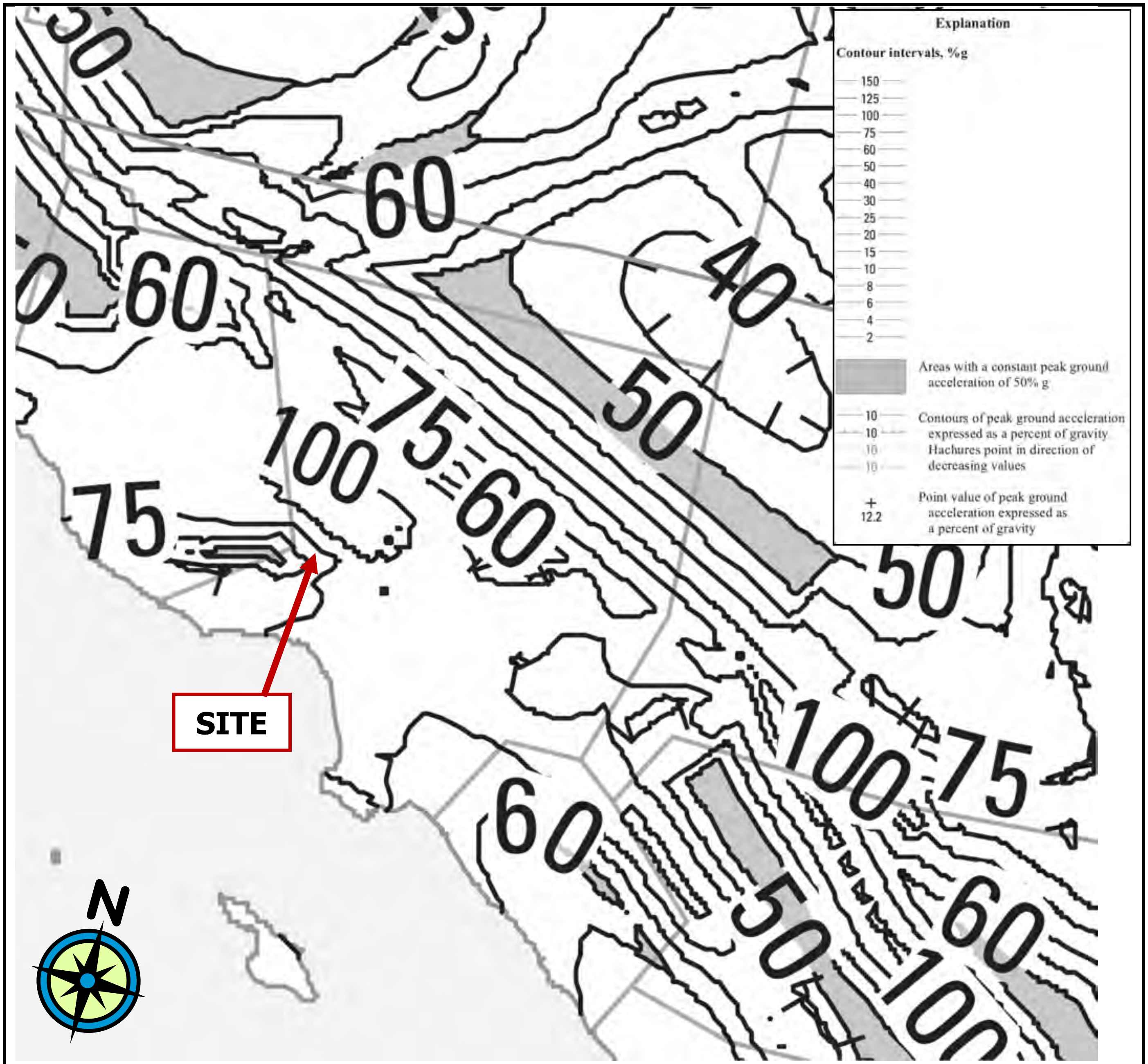
CONCLUSIONS:

THE CALCULATION INDICATES THAT SLOT CUTS UP TO 8 FEET WIDE AND 5 FEET HIGH HAVE A SAFETY FACTOR GREATER THAN 1.25 AND ARE TEMPORARILY STABLE.

Seismic Design Considerations

Any new structures to be developed in the proposed development area should be designed in accordance with the seismic design considerations contained in 2022 California Building Code (CBC) §1613 and *Standard ASCE/SEI 7-16*. The following parameters should be considered for design:

MAXIMUM CONSIDERED EARTHQUAKE GEOMETRIC MEAN



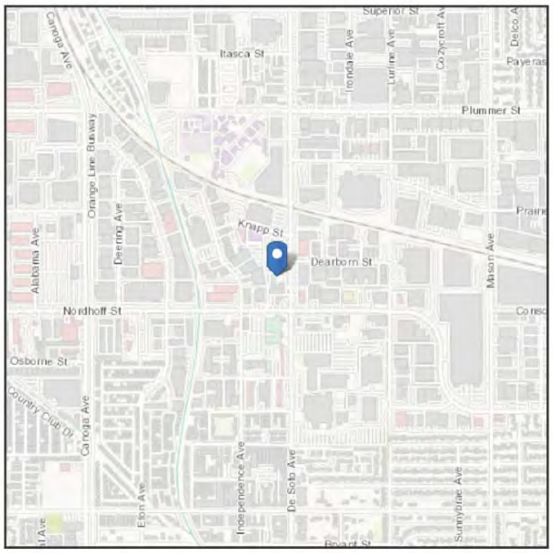
Reference:	Portion of Figure 22-9, ASCE Standard: ASCE/SEI 7-16
Project Address:	9143 De Soto Avenue Chatsworth, California Plate 5

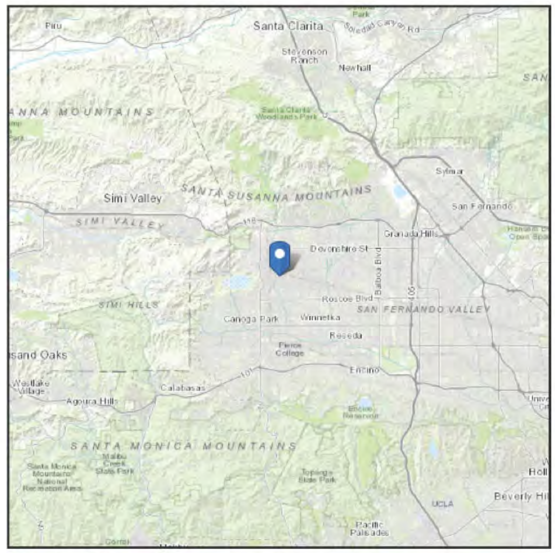
ASCE 7 Hazards Report

Address:
9143 De Soto Ave
Chatsworth, California
91311

Standard: ASCE/SEI 7-16
Risk Category: II
Soil Class: D - Default (see Section 11.4.3)

Latitude: 34.236703
Longitude: -118.589168
Elevation: 891.5951030421974 ft (NAVD 88)





Seismic

Site Soil Class: D - Default (see Section 11.4.3)

Results:

S_S :	1.691	S_{D1} :	N/A
S_1 :	0.6	T_L :	8
F_a :	1.2	PGA :	0.68
F_v :	N/A	PGA_M :	0.816
S_{MS} :	2.029	F_{PGA} :	1.2
S_{M1} :	N/A	I_e :	1
S_{DS} :	1.353	C_v :	1.438

Ground motion hazard analysis may be required. See ASCE/SEI 7-16 Section 11.4.8.

Data Accessed: Wed Aug 09 2023

Date Source: USGS Seismic Design Maps

AMERICAN SOCIETY OF CIVIL ENGINEERS

The ASCE 7 Hazard Tool is provided for your convenience, for informational purposes only, and is provided "as is" and without warranties of any kind. The location data included herein has been obtained from information developed, produced, and maintained by third party providers; or has been extrapolated from maps incorporated in the ASCE 7 standard. While ASCE has made every effort to use data obtained from reliable sources or methodologies, ASCE does not make any representations or warranties as to the accuracy, completeness, reliability, currency, or quality of any data provided herein. Any third-party links provided by this Tool should not be construed as an endorsement, affiliation, relationship, or sponsorship of such third-party content by or from ASCE.

ASCE does not intend, nor should anyone interpret, the results provided by this Tool to replace the sound judgment of a competent professional, having knowledge and experience in the appropriate field(s) of practice, nor to substitute for the standard of care required of such professionals in interpreting and applying the contents of this Tool or the ASCE 7 standard.

In using this Tool, you expressly assume all risks associated with your use. Under no circumstances shall ASCE or its officers, directors, employees, members, affiliates, or agents be liable to you or any other person for any direct, indirect, special, incidental, or consequential damages arising from or related to your use of, or reliance on, the Tool or any information obtained therein. To the fullest extent permitted by law, you agree to release and hold harmless ASCE from any and all liability of any nature arising out of or resulting from any use of data provided by the ASCE 7 Hazard Tool.

Additional Seismic Design Considerations for Site Class D ("Default" / "Stiff Soil")

Under EXCEPTION A of ASCE/SEI 7-16 §11.4.8, A ground motion hazard analysis is not required for structures other than seismically isolated structures and structures with damping systems where the Seismic Response Coefficient "C_S" is determined as provided herein, (below):

Seismic Response Coefficient "C_S":
$C_S = \{(S_{DS})*[1/(R/I_e)]\}$; for: $T \leq (1.5)*(T_S)$ *(eq. 12.8-2, ASCE/SEI 7-16)*
$C_S = \{[(1.5)*(S_{D1})]/[(T)*(R/I_e)]\}$; for: $(1.5)*(T_S) < T \leq T_L$ *(eq. 12.8-3, ASCE/SEI 7-16)*
$C_S = \{[(1.5)*(S_{D1})*(T_L)]/[(T)*(R/I_e)]\}$; for: $T > T_L$ *(eq. 12.8-4, ASCE/SEI 7-16)*

Where:
Site Class is "D" – "Default / Stiff Soil"
S_1: Mapped MCE_R, 5% damped, spectral response acceleration parameter at a 1s period, and "S_1" ≥ 0.2.
S_{DS}: Design, 5% damped, spectral response acceleration parameter at short periods.
S_{D1}: Design, 5% damped, spectral response acceleration parameter at a 1s period.
R: Response modification coefficient.
I_e: Importance Factor.
T: Fundamental period of the structure.
T_S: $T_S = S_{D1}/S_{DS}$
T_L: Long-period transition period.

Under EXCEPTION A of ASCE/SEI 7-16 §11.4.8, the following seismic design parameters may be determined from their respective sections of ASCE/SEI 7-16 without performing a site-specific ground motion hazard analysis, provided that the proposed development meets the requirements of EXCEPTION A:

Long-Period Site Coefficient (at 1.0-s Period), "F_V": $F_V = 1.7$ *(Table 11.4-2, ASCE/SEI 7-16)*

MCE_R, 5% Damped, Spectral Response Acceleration Parameter at 1s Period adjusted for Site Class effects, "S_{M1}":
$S_{M1} = F_V * S_1 = (1.7) * (0.6) = 1.02$ *(eq. 11.4-2, ASCE/SEI 7-16)*

Design, 5% Damped, Spectral Response Acceleration Parameter at 1s Period adjusted for Site Class effects, "S_{D1}":
$S_{D1} = (2/3) * S_{M1} = (2/3) * (1.02) = 0.68$ *(eq. 11.4-4, ASCE/SEI 7-16)*

Seismic Design Category, "SDC": SDC = "D" *(ASCE/SEI 7-16 §11.6)*

APPENDIX IV – GENERAL RECOMMENDATIONS

Drainage and Maintenance

Maintenance of the property and structures located within must be performed to minimize the chance of serious damage and/or instability to improvements. Most problems are associated with or triggered by water. Therefore, a comprehensive drainage system should be designed and incorporated into the final plans. In addition, pad areas should be maintained and planted in a way that will allow this drainage system to function as intended. The following are specific drainage, maintenance, and landscaping recommendations. Reductions in these recommendations will reduce their effectiveness and may lead to damage and/or instability to the improvements. It is the responsibility of the property owner to ensure that the residence and drainage devices are maintained in accordance with the following recommendations and the requirements of all applicable government agencies.

Drainage
Positive pad drainage should be incorporated into the final plans. The pad should slope away from the footings at a minimum five percent slope for a horizontal distance of five feet. In areas where there is insufficient space for the recommended five-foot horizontal distance concrete or other impermeable surface should be provided for a minimum of three feet adjacent the structure. Pad drainage should be at a minimum of two percent slope where water flows over lawn or other planted areas. Drainage swales should be provided with area drains about every fifteen feet. Areas drains should be provided in the rear and side yards to collect drainage. All drainage from the pad should be directed so that water does not pond adjacent to the foundations or flow towards them. Roof gutters and downspouts are required for the proposed structures and should be connected into a buried area drain system. All drainage from the site should be collected and directed via non-erosive devices to a location approved by the building official. Area drains, subdrains, weep holes, roof gutters and downspouts should be inspected periodically to ensure that they are not clogged with debris or damaged. If they are clogged or damaged, they should be cleaned out or repaired.

Landscaping (Planting)
Planters placed immediately adjacent to the structures are not recommended. If planters are proposed immediately adjacent to structures, impervious above-grade or below-grade planter boxes with solid bottoms and drainage pipes away from the structure are suggested. All slopes should be maintained with a dense growth of plants, ground-covering vegetation, shrubs and trees that possess dense, deep root structures and require a minimum of irrigation. Plants surrounding the development should be of a variety that requires a minimum of watering. It is recommended that a landscape architect be consulted regarding planting adjacent to improvements. It will be the responsibility of the property owner to maintain the planting. Alterations of planting schemes should be reviewed by the landscape architect.

Irrigation
An adequate irrigation system is required to sustain landscaping. Over-watering resulting in runoff and/or ground saturation must be avoided. Irrigation systems must be adjusted to account for natural rainfall conditions. Any leaks or defective sprinklers must be repaired immediately. To mitigate erosion and saturation, automatic sprinkling systems must be adjusted for rainy seasons. A landscape architect should be consulted to determine the best times for landscape watering and the proper usage.

Pools/Plumbing
Leakage from a swimming pool or plumbing can produce a perched groundwater condition that may cause instability or damage to improvements. Therefore, all plumbing should be leak-free. Pools located adjacent to descending slopes should be provided with a pool subdrain system.

Grading & Earthwork

General Grading Guidelines

1. Prior to commencement of work, a pre-grading meeting shall be held. Participants at this meeting will consist of the contractor, the owner or his representative, and the soils engineer and/or engineering geologist. The purpose of the meeting is to avoid misunderstanding of the recommendations set forth in this report that might cause delays in the project.
2. Prior to placement of fill materials, all vegetation, rubbish, and other deleterious material should be disposed of off-site. The proposed structures should be staked out in the field by a surveyor. This staking should, as a minimum, include areas for over-excavation, toes of slopes, tops of cuts, setbacks, and easements. All staking shall be offset from the proposed grading area at least (5) feet.

   The proposed construction areas should be excavated down to competent bedrock (or other recommended competent earth material).
3. The excavated grade (or "bottom"), that is determined to be satisfactory for the support of the controlled fill materials, shall then be scarified to a depth of at least (6) inches and moistened as required. The bottom should be compacted to at least (90) percent relative compaction.
4. The controlled fill materials shall consist of earth materials approved by the project soils engineer and/or engineering geologist. These materials may be obtained from the on-site excavation areas, from any other approved source areas, and by blending soils from one or more sources. The controlled fill materials used shall be free from organic matter, vegetation, and other deleterious substances. Also, the controlled fill materials shall not contain rocks greater than (8) inches in diameter, nor of a quantity sufficient to make compaction difficult.
5. The approved controlled fill materials shall be placed in approximately level layers ("lifts") about (4 to 6) inches thick and moistened as required. Each layer shall be thoroughly mixed to attain uniformity of moisture in each layer.

   When the moisture content of the controlled fill materials is found to be (3) percent or more below the optimum moisture content, as specified by the soils engineer, water shall be added and thoroughly mixed in until the moisture content is brought up to the optimum moisture content, and <u>no more than</u> (3) percent above the optimum moisture content.

   When the moisture content of the controlled fill materials is greater than (3) percent above the optimum moisture content, as specified by the soils engineer, the fill material shall be either dried and aerated by scarifying, or it shall be blended with additional drier fill materials and thoroughly mixed until the moisture content is brought down to the optimum moisture content, and <u>no more than</u> (3) percent above the optimum moisture content.

   Each lift of controlled fill materials shall be compacted to a minimum of (90) percent relative compaction (as determined by the modified Proctor maximum dry density - ASTM D 1557), using approved compaction equipment. Where cohesionless soil having less than (15) percent finer than (0.005) millimeters is used for controlled fill materials, the controlled fill material shall be compacted to a minimum of (95) percent relative compaction.
6. Review of controlled fill material placement and compaction should be provided by the soils engineer (or his designee) during the progress of grading. Generally, density tests will be required at intervals not exceeding (2) feet of fill height/depth, or for every (500) cubic yards of controlled fill materials placed.
7. During periods when inclement weather is expected at the project site, all controlled fill materials that have been spread and are awaiting compaction shall be compacted before stopping work, either because of inclement weather or at the end of the work day. The upper surface of the controlled fill area shall

sloped/contoured to drain all precipitation to a single location; where water may be collected and removed from the controlled fill area.

Following inclement weather, work may resume only after the condition of the controlled fill area and materials have been review by the soils engineer, and he has given authorization to resume work. Loose fill materials not compacted prior to the rain shall be removed and aerated so that the moisture content of these controlled fill materials will be <u>not less than</u> and <u>no more than</u> (3) percent above the optimum moisture content.

Surface materials previously compacted before the inclement weather period, shall be scarified, brought to the proper moisture content, and re-compacted prior to placing additional controlled fill materials, if deemed necessary by the soils engineer

8. Review of geotechnical data available for the local vicinity of the site indicates that septic tanks, seepage pits/cesspools, or leach fields may be encountered during site grading. If encountered, these should be drained of effluent or drilled out if they have been backfilled. The cleaned-out area should be inspected by the soils engineer and the local building official prior to backfill. Seepage pits/cesspools may be filled with approved controlled fill materials, lean-mix concrete (or 2-sack slurry), or (¾) inch crushed rock gravel. Whichever backfill material is selected, at least (5) feet of controlled fill materials, placed at a minimum of (90) percent relative compaction should cap the backfilled seepage pit/cesspool.

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