Geotechnical Site Evaluation
and
Storm Water Infiltration Test Report
Proposed 4 Level, Self-Storage Building
Extra Space Storage #1974
1761 West Katella Avenue
(APN 128-542-011)
Anaheim, California

prepared for

Extra Space Storage
2795 East Cottonwood Parkway, #300
Salt Lake City, Utah 84121

GORIAN
& ASSOCIATES, INC.

TABLE OF CONTENTS

1. INTRODUCTION 1
2. PROPOSED DEVELOPMENT 1
3. SCOPE OF SERVICES 2
  3.1. ARCHIVAL REVIEW 2
  3.2. SUBSURFACE EXPLORATION 2
  3.3. INFILTRATION TESTING 2
  3.4. GEOTECHNICAL LABORATORY TESTING 3
  3.5. GEOTECHNICAL ENGINEERING ANALYSIS AND REPORT PREPARATION 3
4. SITE CONDITIONS 3
  4.1. SITE DESCRIPTION 3
  4.2. SUBSURFACE CONDITIONS 3
    4.2.1. Alluvium 4
    4.2.2. Artificial Fill 4
    4.2.3. Asphaltic Concrete and Aggregate Base 4
  4.3. GROUNDWATER 4
  4.4. FAULTING AND SEISMICITY 4
  4.5. FLOOD POTENTIAL 5
  4.6. HYDROCONSOLIDATION 5
5. CONCLUSIONS AND RECOMMENDATIONS 5
  5.1. GENERAL 5
  5.2. GEOTECHNICAL SEISMIC DESIGN 5
  5.3. STORMWATER INFILTRATION TESTING 6
  5.4. BASEMENT EXCAVATION 6
    5.4.1. General 6
    5.4.2. Soil Conditions 7
    5.4.3. Shored Excavation 7
    5.4.4. Surcharge Loading 7
    5.4.5. Soil Pressure 7
    5.4.6. Cantilever Shoring Tilt 8
    5.4.7. Passive Pressure with Soldier Piles 8
    5.4.8. Friction Values for Anchors 8
    5.4.9. Testing of Tiebacks 8
    5.4.10. Soil Nail Walls 9
    5.4.11. Lagging 9
    5.4.12. General Considerations 9
    5.4.13. Barricades 9
    5.4.14. Shoring System Monitoring 9
    5.4.15. Temporary Slopes 10
    5.4.16. Shoring Plan Review and Construction Inspection 10
  5.5. SITE PREPARATION AND GRADING 10
    5.5.1. General 10
    5.5.2. Demolition 10
    5.5.3. Site Clearing 10
    5.5.4. Soil Removals within Basement Excavations 11
    5.5.5. Soil Removals Outside Basement Excavations 11
    5.5.6. Soil Compaction 11
    5.5.7. In-Place Soil Processing 11
    5.5.8. Fill Placement 11
  5.6. SOIL EXPANSIVENESS 11

- 5.7. FOUNDATION DESIGN 11
  - 5.7.1. Design Data 11
  - 5.7.2. Mat Slab Design Data 12
  - 5.7.3. Lateral Resistance 12
  - 5.7.4. Estimated Settlements 12
  - 5.7.5. Footing Excavations 12
  - 5.7.6. Premoistening 13
- 5.8. SLABS-ON-GRADE 13
  - 5.8.1. Site Preparation 13
  - 5.8.2. Slab-on-Grade Design Data 13
  - 5.8.3. Premoistening 13
  - 5.8.4. Concrete Placement and Cracking 13
  - 5.8.5. Moisture Vapor Barrier 13
- 5.9. BASEMENT RETAINING WALLS 14
  - 5.9.1. Foundations 14
  - 5.9.2. Lateral Earth Pressures 14
  - 5.9.3. Lateral Seismic Pressure 15
  - 5.9.4. Waterproofing 15
  - 5.9.5. Drainage 15
  - 5.9.6. Backfilling 15
- 5.10. SOIL CORROSIVITY 16
- 5.11. SITE DRAINAGE 16
- 5.12. GUTTERS AND DOWNSPOUTS 16
- 5.13. PAVEMENT DESIGN 16
- 5.14. PLAN REVIEW(S) 16

6. CLOSURE 16

ATTACHMENTS

References

Figure 1: Vicinity Map

Figure 2: Regional Geologic Map

Figure 3: Seismic Hazard Zone Map

Appendix A: Logs of Subsurface Data

Appendix B: Laboratory Testing

Appendix C: ASCE 7 Hazards Report

Appendix D: Stormwater Infiltration Testing

Plate 1: Boring Location Map

GORIAN
& ASSOCIATES, INC.

Applied Earth Sciences
Geotechnical Engineers
Engineering Geologists
DSA Accepted Testing Laboratory
Special Inspection and Materials Testing

3595 Old Conejo Road
Thousand Oaks
California 91320-2122
805 375-9262

March 30, 2023

Extra Space Storage
2795 East Cottonwood Pkwy
#300
Salt Lake City, Utah 84121

Work Order: 3239-0-0-100

Attention: Mr. Clint Kleppe
Development Manager, Asset Strategy

Subject: **Geotechnical Site Evaluation and Storm Water Infiltration Test Report, Proposed 4 Level, Self-Storage Building, Extra Space Storage #1974, 1761 West Katella Avenue (APN 128-542-011), Anaheim, California**

1. INTRODUCTION

The following report contains the results of our geotechnical site evaluation addressing design and construction of a 4 level building planned at the Extra Space Storage Facility (ESS) #1974 at 1761 West Katella Avenue (APN 128-542-011) in Anaheim, California. In addition, storm water infiltration testing was performed as part of this site evaluation. The address is in the northeast corner of West Katella Avenue and South Humor Drive approximately 1-1/2 block west of Euclid Street as shown on the attached Site Vicinity Map, Figure 1. Based on information provided by ESS, the self-storage building with a 18,377 square foot gross footprint with two above grade levels over two subterranean levels will be constructed in the northern portion of the site as shown on Plate 1.

Geotechnical borings were used to obtain data on the subsurface alluvial soils consisting predominately of silty fine sand interstratified with clayey silt to silty clay to the explored depth of 41 feet as described herein. The field exploration was supplemented with laboratory testing to determine mechanical properties of the encountered soils. In addition, research was performed that indicated the site is not within Earthquake Fault, Liquefaction, or Landslide Zones (CGS, *Earthquake Zones of Required Investigation* website). Based on our site evaluation, the site is suitable for the proposed construction from a geotechnical standpoint provided recommendations presented herein are implemented in the project design and construction. Descriptions of the site and geologic units along with our conclusions and recommendations are presented within the text of this report.

2. PROPOSED DEVELOPMENT

Based on information provided by ESS, the project will consist of a four level self-storage building having a gross footprint of 18,377 square feet. The building will have two above grade levels over two subterranean levels in the northern portion of the site as shown on Plate 1. Construction of the building will include demolition of the existing parking / open air storage area. The site is relatively flat and therefore no significant grade changes are anticipated to the site except possibly for drainage adjacent the completed building. Overall configuration of the site will remain roughly as currently laid out with the driveway off South Humor Drive remaining at the current location as shown on Plate 1a. The planned configuration of the site is shown on Plate 1b.

The building is anticipated to be supported on continuous footings, with individual storage units possibly supported on a mat slab on grade within the interior of the structure. Continuous footings at the perimeter and at the interior are anticipated to be loaded to 7 to 10 kips per linear foot. The steel stud walls spaced on 10-foot centers will be loaded to approximately 5 kips per linear foot and will be supported directly on a thickened interior slab typical of this type of structure. The storage live loads are anticipated to be 125 pounds per square foot.

3. SCOPE OF SERVICES

This office performed a geotechnical evaluation of the site outlined herein in general accordance with the *Scope of Services* presented in our proposal of August 27, 2022 (Proposal Number: 7244-10). Our scope of services included the following:

3.1. ARCHIVAL REVIEW

Pertinent references in our office including regional geologic references applicable to the site were reviewed with respect to the proposed development.

3.2. SUBSURFACE EXPLORATION

Three borings were drilled to a total depth of 41 feet below the existing ground surface within the area of the proposed building to explore the underlying soil and groundwater conditions. The borings were drilled using a subcontractor supplied and operated truck mounted CME 75 hollow stem drill rig equipped with 8-inch diameter augers. A geologist from our office logged the borings and obtained both relatively undisturbed drive and bulk samples for laboratory testing. Drive samples were obtained using an automatic hammer providing a hammer weight of 140 pounds with a fall of 30 inches. The Logs of Subsurface Data are presented in Appendix A and the approximate locations of the points of exploration are shown on the attached Boring Location Map, Plate 1.

At the conclusion of drilling, logging, and sampling the borings were backfilled with the boring cuttings and tamped. However, the backfill may settle over time and the site representative should fill any depression that may occur, as necessary.

3.3. INFILTRATION TESTING

The storm water infiltration testing was conducted by the drilling of two infiltration test borings to total depths ranging from 21 feet (IB-1) to 14 feet (IB-2) below the existing ground surface using a subcontractor supplied and operated truck mounted CME 75 hollow-stem auger drill rig equipped with 8-inch diameter augers. A geologist from our office logged and sampled the subsurface soils at various depths from the infiltration borings for additional soil classification per the design manual.

The hollow-stem auger borings (IB-1 and IB-2) were excavated within the area of the proposed BMP to total depths of 14 to 21 feet below the ground surface and were tested for stormwater infiltration. At the conclusion of logging and soil sampling, the hollow-stem borings were converted into infiltration test wells. 1 foot of medium bentonite chips was placed on the bottom of each boring, then a 2-inch diameter PVC pipe was placed in the boring; with the lower 5 feet of pipe being slotted (0.02). The annular space between the slotted pipe and the wall of the excavation was backfilled using #3 sand to just above the slotted portion of the pipe, then another 1 foot of medium bentonite chips was placed. The upper portion of the annular space was backfilled with soil cuttings from the borings and the test borings were presoaked. At the completion of the infiltration testing (as discussed later herein), the borings were back-filled and topped with rapid set concrete.

3.4. GEOTECHNICAL LABORATORY TESTING

A program of laboratory testing was performed to evaluate geotechnical properties of selected soil samples obtained during the subsurface exploration. Testing was performed to determine compaction characteristics, consolidation potential, shear strength, grain size analysis, and in-situ moisture content and dry density. One sample of the underlying soil was provided to an independent corrosion engineer for corrosion testing. The results of the laboratory testing are presented in Appendix B.

3.5. GEOTECHNICAL ENGINEERING ANALYSIS AND REPORT PREPARATION

The results of our field and laboratory programs were used in engineering evaluations to develop geotechnical recommendations for design and construction of the self-storage structure. The results of our completed scope of services are presented in this geotechnical report that includes:

a) A description of the subsurface conditions encountered in the exploratory excavations, including Logs of Subsurface Data (Appendix A) and a Boring Location Map (Plate 1) showing the approximate excavation locations.

b) A description of the laboratory testing program, including tests results (Appendix B).

c) Discussion and recommendations regarding:
   i) Geologic hazards including seismic setting of the site and faulting;
   ii) Seismic design criteria;
   iii) Seismically induced settlement;
   iv) Soil collapse and expansion potential;
   v) Site preparation and remedial grading;
   vi) Concrete slabs on grade including aggregate base and vapor retarder;
   vii) Modulus of subgrade reaction;
   viii) Conventional and mat foundation design recommendations;
   ix) Estimated settlements;
   x) Pavement and hardscape design recommendations;
   xi) Soil chemistry analysis, by subcontract;
   xii) Lateral earth pressures.
   xiii) Temporary excavations;
   xiv) Shoring.

4. SITE CONDITIONS

4.1. SITE DESCRIPTION

Extra Space Storage Facility (ESS) #1974 is located at 1761 West Katella Avenue (APN 128-542-011)) in Anaheim, California approximately a mile west of Disneyland, (see the attached Site Vicinity Map, Figure 1). The nearly level square property is situated at the northeast corner of West Katella Avenue and South Humor Drive approximately 1.5 blocks west of Euclid Street. The property is developed with a single story storage building in the southern portion of the property with paved parking and open air storage in the northern portion of the site. Drainage of the site is by sheet flow to storm drain inlet structures within the parking and drive areas.

4.2. SUBSURFACE CONDITIONS

The site, as encountered in our subsurface exploration program, is underlain by asphaltic concrete and aggregate base mantling Quaternary-age alluvium to the maximum depth explored, 41 feet (borings B-1 through B-3). However, 17 feet of artificial fill was locally encountered in boring B-2 mantling the underlying alluvial soils consisting of pea gravel (this appears to be a backfill of a prior excavation). Descriptions of these units are presented below and in the attached Logs of Subsurface Data (Appendix A).

4.2.1. Alluvium

Quaternary-age alluvium underlies the entire site to the maximum depth explored, 41 feet (B-1 through B-3) (see the attached Regional Geologic Map, Figure 2). As encountered in the borings, the upper portion of the alluvium generally consists of brown grading to yellowish brown silty fine sand in a damp to moist and medium dense condition. At depth, the alluvium generally consists of yellowish brown to light yellowish brown to light gray silty fine to silty fine to coarse sand with few fine gravels in a damp to moist and medium dense to dense condition. Typically, these sandy soils are friable. The sandy units of the alluvium are commonly interstratified with yellowish brown to light yellowish brown to grayish brown silt to silty clay in a moist and stiff to hard condition.

4.2.2. Artificial Fill

Artificial fill was encountered only in boring B-2 and was observed to be approximately 17 feet in thickness. The fill consists of pea gravel and based on sampling efforts; the pea gravel is in a loose condition. It appears the pea gravel was used to backfill an excavation, however, these materials are not well consolidated resulting in a sag in the parking lot and subsequent standing water condition after a heavy rain storm.

4.2.3. Asphaltic Concrete and Aggregate Base

The surface of the site is generally covered with 2 to 3-inches of asphaltic concrete which is underlain by 4 to 6-inches of aggregate base in a damp and dense condition. However, in the area of B-2 the asphaltic concrete was observed to be 7.5 inches in thickness.

4.3. GROUNDWATER

Groundwater was not encountered in the hollow stem auger borings extended to a maximum depth of 41 feet below the existing ground surface. However, high moisture contents were noted in the borings within the silty soils at 20 feet below grade. Based on the *Seismic Hazard Zone Report* for the Anaheim 7.5-minute Quadrangle, historic groundwater is approximated at 50 feet below the site. Based on California Department of Water Resources *California Groundwater Live*, groundwater measured is at 97 feet in a well roughly 1.4 miles to the northeast of the property. As in any groundwater situation, groundwater levels can fluctuate and groundwater (or perched zones) may be encountered at higher elevations than previously observed in the general area.

4.4. FAULTING AND SEISMICITY

The storage facility, like any other development in greater Southern California, is in a seismically active region prone to occasional damaging earthquakes. The destructive power of earthquakes can be grouped into fault-rupture, ground shaking (strong motion), and secondary effects of ground shaking such as tsunami, liquefaction, settlement, landslides, etc.

The hazard of fault-rupture is generally thought to be associated with a relatively narrow zone along well-defined pre-existing active faults. No doubt there is and will be exceptions to this, because it is not possible to predict the precise location of a new fault where none existed before (CDMG, 1975). Holocene-active faults are not known to cross the site nor is the project site currently within an Alquist-Priolo (A-P) Earthquake Fault Zone as defined by the State Geologist (CGS 2018). The site is between two major, active fault zones: the Newport-Inglewood Fault Zone to the southwest and the Whittier-Elsinore Fault Zone to the northeast, approximately 8.7 and 10.5 miles from the site respectively. Additionally, the site is between the El Modeno and Los Alamitos Faults to the east and west, approximately 8 and 5.6 miles from the site respectively. Potential for surface ground rupture due to faulting onsite during the project lifetime is considered remote.

Although no active or potentially active faults are known to exist within or adjacent the site, the area will be subject to strong ground motion from occasional earthquakes in the region. Significant earthquakes have occurred within a 40-mile radius of the Site within the last 50 years. Additional earthquakes will likely occur in this area within the life of the project and it will experience strong ground shaking from these events.

Probabilistic seismic hazard analyses (PSHA) predict the Design Basis Earthquake having a 2% probability of exceedance in 50 years (2,475-year return period will have a peak ground acceleration estimated to be 0.68g based on a seismic event with a mean magnitude of 6.7 (Mw) at a mean distance of 12.64 km from the site. This is based on the U.S. Geological Survey (USGS) interactive web application, Unified Hazard Tool https://earthquake.usgs.gov/hazards/interactive/ for the D class site.

Secondary effects of strong ground motion include tsunami, seiche, liquefaction, settlement, earthquake triggered landslides, and flooding from dam failures. Tsunamis are impulsively generated water waves that can cause damage to shoreline areas. A seiche is an oscillation wave within an enclosed body of water. The site is not near the ocean or adjacent a body of water and, therefore, is not subject to tsunami and seiche hazards. Furthermore, the site is not prone to earthquake triggered landslides due to the relatively low relief in the area and preponderance of development covered land, nor is the site in the vicinity of any dam failure inundation zone. The site is not within a State designated seismic hazard zone for liquefaction potential (CGS, *Earthquake Zones of Required Investigation* website). See Figure 3, the Seismic Hazards Zone Map.

4.5. FLOOD POTENTIAL

The site is not in an area of flood hazard based on the FEMA flood hazard zone as indicated on the FEMA Flood Zone FIRM Panel: 06059C0137J (effective on12/03/2009).

4.6. HYDROCONSOLIDATION

Hydroconsolidation occurs when the soil structure collapses due to soil wetting resulting in consolidation of the soil column. The consolidation test performed for this evaluation indicate hydroconsolidation is negotiable for the onsite soils tested below the proposed basement level.

5. CONCLUSIONS AND RECOMMENDATIONS

5.1. GENERAL

The site was evaluated from a geotechnical standpoint for construction of a self-storage facility described herein. The bottom of the basement excavation will expose alluvial deposits suitable for support of the structure. Differential settlement should be negligible based on the bearing capacities provided herein. The project may be developed as described earlier in this report provided recommendations presented herein are followed and incorporated into the project design and construction. Excavation of the basement should be completed with care to avoid undermining adjacent structures or facilities.

5.2. GEOTECHNICAL SEISMIC DESIGN

As previously discussed, active faults identified by the State are not onsite nor is the site within an Alquist-Priolo Earthquake Fault Zone. Nevertheless, the site is within a seismically active region prone to occasional damaging earthquakes.

Structures within the site may be designed using procedures for seismic design presented in ASCE/SEI 7-16. Mapped acceleration parameters are initially determined for sites having a shear wave velocity of 2,500 feet per second (Section C11.4.4). The S_s and S_1 values are adjusted to obtain the maximum considered earthquake (MCE) spectral acceleration values for the site based on its site class of D. The seismic design parameters for the site's coordinates (latitude 33.8040 N and longitude 117.9453 W) were obtained from the web based ASCE 7 Hazard Tool https://asce7hazardtool.online/ The parameters are presented on the following page (the full report is presented in Appendix C).

SEISMIC PARAMETER	VALUE PER CBC
Short Period Mapped Acceleration (S_s)	1.416g
Long Period Mapped Acceleration (S_1)	0.5g
Site Class Definition	D
Site Coefficient (F_a)	1.0
Site Coefficient (F_v)	1.7*
$S_{MS} = F_aS_s$	1.416g
$S_{M1} = F_vS_1$	0.85g*
$S_{DS} = 2/3S_{MS}$	0.944g
$S_{D1} = 2/3S_{M1}$	0.0.567g*
PGA_M	0.662

**Based on proposed development meeting requirements of the exemption for Site Class D sites in Section 11.4.8 of ASCE 7-16. Further analysis may be required once the Response Modification Factor and Period of the proposed development are known.*

The purpose of the building code earthquake provisions is primarily to safeguard against major structural failures and loss of life, not to limit damage nor maintain function. Therefore, values provided in the building code should be considered minimum design values and should be used with the understanding site acceleration could be higher than addressed by code-based parameters. Cracking of walls and possible structural damage should be anticipated in a significant seismic event.

5.3. STORMWATER INFILTRATION TESTING

The test zone was pre-soaked by filling to the top of the casing with water. At the conclusion of the pre-soak, the pipe will be refilled with water to the top of the slotted pipe. After the pre-soak, a falling head test was performed for the infiltration well. The measurements were taken at 20-minute intervals.

Based on our test results and field exploration observations, the soils were found to be suitable for construction of a stormwater infiltration system (greater than 0.3 inches/hour). A design infiltration rate of 0.52 inches/hour may be used for design purposes using a reduction factor of 2. The test results can be found in Appendix D.

Sizing of the infiltration system or field construction should be specified by the project design civil engineer. Input should be solicited from and data provided to the civil engineer, structural engineer and geotechnical engineer to optimize the design while minimizing the potential detrimental effects the addition of water could have on the site or adjacent sites. Plans and specifications should be provided to our office for review. Depending on actual design depth(s) and location(s) additional infiltration rate testing may be warranted.

5.4. BASEMENT EXCAVATION

5.4.1. General

The 4 level building proposed in the northern portion of the site will have two levels of subterranean storage, which could have a lower finished floor 24± feet below the ground level floor. Therefore, the bottom of the basement footings could be 26± feet below the ground surface. For this depth of excavation, shoring is anticipated to consist of soldier beams and lagging supported by either tiebacks or rakers. In addition, soils nails could be used when the nails would not project offsite.

The project civil engineer should prepare an excavation plan detailing the excavation and relationship to existing utilities and structures. This office should review the excavation plan prior to starting construction. In addition, this office should evaluate possible loads (such as crane loading) than may surcharge the excavation.

5.4.2. Soil Conditions

The soil conditions anticipated to be encountered in the basement excavation are summarized previously in this report. One caution is a zone of pea gravel was encountered in boring B-2 to a depth of 17 feet. However, the limits of the gravel was not explored.

5.4.3. Shored Excavation

Shoring for the basement excavation may consist of soldier beams and lagging supported by either tiebacks or rakers along with the possibly of soil nails. Lagging should be used to support the cut between the piles. Grouting is the preferred method to fill the voids between the cut and lagging. The shoring should be designed to include the lowest construction elevation (bottom of footing). Care will be required to avoid damaging buried utilities or possible foundations of adjacent structures. The excavation and shoring will encounter sandy alluvial deposits is as described previously herein and in detail in the attached Logs of Subsurface Data (Appendix A).

5.4.4. Surcharge Loading

An area surcharge of 300 psf should be included in the shoring design where the shoring is near street or interior traffic. The lateral pressure on the shoring due to a uniform area surcharge of intensity q (force/area) is equal to a uniform pressure of 0.4q over the entire height of the wall. Surcharge on the shoring from construction equipment (e.g., crane or concrete pump) or adjacent structures directly adjacent the top of a shored cut should be evaluated on an individual basis. The structural engineer should evaluate the surcharge loading from the adjacent building, etc.

5.4.5. Soil Pressure

Shoring should be designed for temporary lateral earth pressure plus lateral pressure imposed by existing adjacent foundations or surcharges. For supported shoring systems (i.e., systems supported with tiebacks or rakers) the lateral earth pressure will be initiated below the ground surface, therefore, at the ground surface the pressure may be taken as zero. The pressure will increase with depth to 24H at a depth of 0.2H below the ground surface. The shoring pressure would then extend uniformly at 24H to a depth of 0.8H and then decrease uniformly to zero at the base of the excavation. H is the supported total height of the cut and the resultant of 24H is in units of psf. The pressure diagram is shown in Figure 4 under Lateral Earth Pressures in Basement Retaining Walls. As an alternate to a trapezoidal pressure distribution, basement walls may be design using an equivalent at rest pressure of 50 pounds per cubic foot. The resultant may be applied at one third the wall height measured from the bottom of the wall. The width of active pressure acting on the pile below the bottom of the excavation may be taken as the pile diameter.

Cantilevered shoring systems should be designed for an active earth pressure distribution of 30 pounds per cubic foot (pcf) with level ground behind the shoring based on a triangular pressure distribution starting at zero at the ground surface. The value of 30 pcf is an ultimate value without a factor of safety. The width of active pressure acting on the pile below the bottom of the excavation should be two pile diameters for a cantilevered soldier pile.

The shoring pressures do not include lateral loads from surcharges (such as crane loading) near the top of the excavation.

5.4.6. Cantilever Shoring Tilt

Similar to a cantilever retaining wall, cantilever shoring designed for an active pressure can yield at the top to develop full active pressure. Generally, tilt is a function of the wall height and it this case is estimated at .001 to .002 of the wall height.

5.4.7. Passive Pressure with Soldier Piles

The lower ends of the soldier piles will be seated in alluvial deposits is as described previously herein and in the attached Logs of Subsurface Data (Appendix A). For isolated piles (spaced at least 3 diameters center to center) the passive earth pressure should start at zero at the excavated grade. This value may be increased at a rate of 300 pounds per cubic foot for each foot of depth below the proposed base of excavation to a maximum of 2500 pounds per square foot. The surface area (pile diameter) that the allowable passive pressure may induce passive resistance may be doubled for soldier beams that are a minimum of 3 diameters apart center to center.

For vertical support, a unit friction value of 350 pounds per square foot may be used for that portion of the soldier pile encased in structural concrete or drilled and cast concrete pile extending below the lowest depth of excavation. The unit of friction is independent of the pile diameter; however, the piles should be at least 24 inch diameter with a minimum embedment depth of 15 feet below the lowest excavation depth. Fixity may be assumed at 5 feet below the lowest unsupported grade (such as the basement excavation).

5.4.8. Friction Values for Anchors

Tieback anchors should extend into the embankment at an angle of about 20° to 35° below horizontal. The bond or anchor length of the tieback is the portion located behind the active wedge, a 60° plane from horizontal generated upwards from the toe of the proposed excavation. The portion of the tieback that extends from the soldier pile to the 60° plane is identified as the free anchor length. Tieback anchors are to be designed to obtain their capacities from within the bond length. Capacity should not be assumed from within the free anchor length, nor should the free anchor length be grouted prior to tensioning of the tendon.

Tiebacks will be founded predominately in sandy silts to silty sand alluvial deposits is as described previously herein and in detail in the attached Logs of Subsurface Data (Appendix A). The frictional resistance of 16 inch diameter anchors a minimum of 15 feet below the ground surface may be designed within an allowable frictional resistance of 600 psf. The resistance may be increased to 1,000 psf below a depth of 30 feet.

The capacities presented above are based on minimum spacing requirements. The minimum spacing between adjacent tiebacks should be 3 diameters center to center where the diameter is the largest diameter of the tiebacks.

Installation of the tiebacks should be observed by this office. Installation should also be in accordance with the Post Tensioning Institute *Recommendations for Prestressed Rock and Soil Anchors*.

5.4.9. Testing of Tiebacks

Failure or yielding of anchors generally occurs within the soil surrounding the anchor, not at the soil/concrete interface. The capacity of any anchor is calculated based on soil strength. Each tieback should be tested to verify the design strengths in accordance with the Post Tensioning Institute (PTI) Recommendations for Prestressed Rock and Soil Anchors (or per the shoring engineer’s recommendations). Testing requirements should be clearly indicated on the shoring plans.

5.4.10. Soil Nail Walls

Soil nail retaining walls consisting of grouted anchors extending into the excavation backcut with a reinforced shotcrete face may be used for permanent and temporary support of vertical excavations. The walls should be design by a structural engineer familiar with shoring design and construction. Shear strengths of a frictional strength of 32 degrees and cohesion of 0.0 pounds per square foot (psf) along with a unit weight of 118 pounds per cubic foot (pcf) may be used for cuts into the underlying alluvial soils. A bond strength of 20 pounds per square inch (psi) may be assumed for nail design. Vertical lifts and nail spacing should not exceed five feet with a minimum number of sacrificial nails for test purposes equal to 5 percent of the total wall nails. Vertical drains should be installed between the columns of soil nails. The final wall design should be reviewed by this office.

5.4.11. Lagging

Lagging consisting of treated timber will be required the entire depth of the shored excavation. Wood lagging should be new 3 inch No. 2 or better rough timber (full dimension) Douglas Fir, straight, free of bends, and free from defects that might impair structural strength. Lagging to be left in-place shall be pressure treated for contact with soil. The upper two feet of the shoring and lagging measured from the adjacent grade should be removed when the shoring is no longer needed for support of the excavation. The resulting cavity should be backfilled with grout/slurry or clean soils compacted to a minimum of 90% relative compaction. The cavity may be filled with concrete when the area is below a slab or walkway.

Lagging should be designed to resist an equivalent fluid pressure equal to 30 pcf measured below the ground surface. A maximum lagging pressure of 400 psf may be assumed where the maximum spacing of soldier piles does not exceed 8 feet center to center. An alternate to installing lagging would be to construct the shoring as a continuous gunite/shotcrete wall descending as the excavation proceeds. Cavities behind the lagging and retained soils should be filled with minimum 1-1/2 sack sand/cement slurry (preferred).

5.4.12. General Considerations

The basement excavation can be made with ordinary excavating equipment. Soils between existing foundations and shoring system should be maintained in an undisturbed and intact condition. Caving of soldier beam excavations should be anticipated since sandy materials will be encountered in the excavations. The shoring contractor should be prepared to provide methods to prevent caving such as the use of hollow stem augers, casing, or drilling mud.

Caving of the tieback excavations should be anticipated since sandy materials may be encountered within the alluvial soils. The shoring contractor should be prepared to provide methods to prevent caving such as the use of hollow stem augers or casing. Where caving soils are encountered within the free length of the tieback excavation, that portion of the excavation may be backfilled with sand or low strength sand/cement slurry before testing the anchor. The sand backfill should be placed by pumping. In no cases should the free length portion of the friction tieback be grouted (with high strength grout that would restrict movement of the bond length) prior to testing.

5.4.13. Barricades

Appropriate barricades should be placed at the top of all temporary excavations that are approached by pedestrians or public vehicle traffic (such as in streets or parking areas).

5.4.14. Shoring System Monitoring

The shoring system should be monitored for vertical and horizontal movements at the top of each soldier beam. Reference points for horizontal movement should also be selectively placed at various tieback levels as the excavation progresses. A licensed surveyor should perform the surveying.

The reference points and pile tops should be read prior to commencing the excavation. To create a baseline, all soldier piles should be surveyed twice (approximately one day apart) before beginning excavation. Additional readings should be performed roughly biweekly throughout construction until the shoring and excavation is complete. More frequent reads may be required at critical times of construction or if significant movement is indicated. After completion of the shoring construction and excavation, readings may be taken biweekly until the shoring is no longer needed for support of the excavation.

The survey data should be submitted to the shoring engineer and Gorian and Associates, Inc. within 24 hours of the measurements. The tolerable movement for any location within the structure will be evaluated with the data and is dependent on the soil conditions at that location, the stage of construction, and adjacent structures or loading. Some movement of the shoring can be expected and is considered tolerable. In general, movement in excess of 2 inches horizontally or vertically will require supplemental shoring before excavation continues.

5.4.15. Temporary Slopes

Temporary slopes, if required, should be at a gradient of 1(horizontal):1(vertical) or flatter to a maximum depth of 3 feet. Due to the sandy nature of the soils, vertical cuts should not be made within the site. Sloped excavations can be evaluated when the proposed location and depth are known. One caution is a zone of pea gravel was encountered in boring B-2 to a depth of 17 feet. However, the limits of the gravel was not explored.

During construction, the contractor is responsible for the excavation and maintenance of safe and stable slope angles considering the subsurface conditions and the methods of operations. Temporary excavations should be made per the applicable requirements of the current Cal/OSHA excavation regulations. Surcharge loads should be setback from the top of temporary excavations a minimum horizontal distance of 10 feet.

5.4.16. Shoring Plan Review and Construction Inspection

A structural engineer with shoring experience should prepare the shoring plans. Our office should be provided with a copy of the proposed shoring plans and calculations for review. Variations in subgrade conditions or construction techniques exercised by the shoring contractor may require this office to provide specific modifications to the shoring system or installation. Therefore, this office should perform all geotechnical observations to confirm the subsurface conditions.

5.5. SITE PREPARATION AND GRADING

5.5.1. General

Geotechnical recommendations are presented in the following sections for grading of the building pad within the basement and above the basement. Site preparation and fill placement should be performed per the City of Anaheim standards. Undisturbed in-placed alluvial floodplain deposits are suitable for the support of the proposed construction project.

5.5.2. Demolition

Presently, the area is covered by paving and facilities related to the current use of the property that are planned for demolition. Utilities to remain should be protected in place.

5.5.3. Site Clearing

Prior to starting earthwork, trash, debris, and remnants of demolition within all areas of construction should be stripped and removed from the site. Utilities within the area of proposed grading should be identified and removed or protected prior to grading.

5.5.4. Soil Removals within Basement Excavations

The basement may be cut to the proposed grade and no additional soil removal should be necessary. However, soils disturbed during the basement excavation should be removed to firm in-place soils at the bottom of the excavation and replaced as compacted fill.

5.5.5. Soil Removals Outside Basement Excavations

Soil removals, as a minimum, should extend below soils disturbed during the site clearing. For areas supporting shallow continuous and isolated footings outside of the basement areas, as well as those supporting structural fill or lightly loaded footings, the soil removals should extend to firm native soil, anticipated to be directly below the disturbed zone.

Soil removal and recompaction should be performed as necessary within all areas of at-grade construction requiring compacted engineered fill. The removals should extend a minimum of five feet outside the footings or area of fill placement. Removal may be stopped at property lines and adjacent buildings. This office should evaluate removals adjacent existing structures prior to excavation. When the removals are completed and prior to in-place processing, this office should observe the bottom of the removal areas.

5.5.6. Soil Compaction

Fill soil or in-place compaction should be completed to a minimum 90 percent relative compaction. Relative compaction is the ratio of the in-place dry soil density to the maximum dry soil density as determined in general accordance with ASTM laboratory standard D-1557.

5.5.7. In-Place Soil Processing

Once the soil removals are complete and prior to placing fill, the bottom of the removal area should be processed. Processing consists of scarifying the exposed surface to a depth of roughly 6 to 8 inches, conditioning the scarified soil to above the optimum moisture content, and compacting the scarified soil. Processed soil should be compacted to 90 percent relative compaction.

5.5.8. Fill Placement

Soils generated from the removal areas should be suitable for reuse as fill. Fill soils should be free of significant vegetation, rocks greater than 6 inches in maximum linear dimension, and other deleterious materials. In addition, fill soils should be mixed and blended. Fill soils should be placed in lifts not exceeding 8 inches in maximum loose thickness, moisture conditioned to slightly over optimum moisture content, and compacted to at least 90 percent relative compaction.

5.6. SOIL EXPANSIVENESS

An expansion test conducted on the upper soils within the site are non-expansive. Expansive soils contain clay particles that change in volume (shrink or swell) due to a change in the soil moisture content. The amount of volume change depends upon the soil swell potential (amount of expansive clay in the soil), availability of water to the soil, and the soil confining pressure. Swelling occurs when soils containing clay become wet due to excessive water from poor surface drainage, over-irrigation of lawns and planters, and sprinkler or plumbing leaks. Swelling clay soils can cause distress to structures, walks, drains, and patio slabs.

5.7. FOUNDATION DESIGN

5.7.1. Design Data

The structure may be supported on continuous or isolated footings underlain by engineered compacted soil or firm native soils as addressed above and may be designed for an allowable bearing pressure of 2,500 pounds per square foot (psf). In the basement area, continuous and isolated footings with the minimum width and depth, may be designed using an allowable bearing pressure of 4000 psf. Shallow footings adjacent basement walls, should be included in the design of the wall or stepped down below a 2(horizontal):1(vertical) plane projecting upward from the bottom of basement footings. The allowable net bearing pressure may be increased by one-third when considering wind or seismic loads. The weight of concrete below grade may be excluded from the footing load.

Continuous and isolated footings should have minimum widths of 18 inches and 24 inches, respectively. The footings should be embedded a minimum of 24 inches for interior and exterior footings. The embedment should be measured from the lowest adjacent grade (lowest grade at the time of excavation or after). Interior and basement footings may be embedded a minimum of 24 inches below the interior slab. The above embedments are for footings embedded into soils having a medium expansion index. Steel reinforcement should be per the structural engineers' recommendations. However, minimum continuous footing reinforcement should consist of two number five bars in the top and bottom (total of 4 bars). In addition, interior slabs should be tied to the footings with number 4 bars at 24-inch centers bent 3-feet into the slab and extended to within 3 inches of the bottom of the footing. Perimeter isolated footings should be tied together with a grade beam extending 30 inches deep below the lowest adjacent grade.

5.7.2. Mat Slab Design Data

Mat slabs may be designed using an allowable soil bearing pressure of 1,500 pounds per square foot (at the basement grade) or a modulus of subgrade reaction "K" of 150 pounds per cubic inch (pci) at the basement excavated surface. The project structural engineer should determine the steel reinforcement and concrete compressive strength. The slabs supporting interior steel stud walls should be a minimum of 8 inches thick. A mat slab should be underlain by a minimum 6-inch-thick layer of ½ inch or larger clean aggregate or per applicable building codes, whichever is the more restrictive. In addition, interior mat slab design should include a moisture retarder as indicated under *Slabs on Grade* below.

5.7.3. Lateral Resistance

Lateral forces on foundations may be resisted by passive earth pressure and base friction. Lateral passive earth pressure may be considered equal to a fluid weighing 250 pcf. The lateral passive pressure may be increased to a maximum of 1500 psf. Base friction may be computed at 0.3 times the normal load. Base friction and passive earth pressure may be combined without reduction.

5.7.4. Estimated Settlements

Static settlement of footings should be evaluated once building footing locations and structural loads are known. However, footing settlement for static loading is anticipated on the order of 1/2 inch or less, with a maximum differential settlement of 1/4± inch over a span of approximately 30 feet or between adjacent individual footings. This is provided building construction is started directly after footing excavation, footings are cast soon after the footing excavation, and construction is completed in a timely manner. Settlements due to static loading are expected to occur rapidly as the loads are applied.

All structures settle during construction and some minor settlement of structures can occur after construction during the life of the project. Minor wall cracking could occur within the structure associated with expansion and contraction of the structural members. In addition, wall or slab cracking may be associated with settlement or expansive soil movement. Additional settlement/soil movement could occur if the soils dry or become saturated due to excessive water infiltration generally caused by excessive irrigation, poor drainage, etc.

5.7.5. Footing Excavations

Excavation of the footings should be started directly after the excavation of the basement and the footings should be poured soon after footing excavation is complete. This office should observe the footing excavations prior to placing reinforcing steel. Footings should be cut square and level and cleaned of loose soils. Soil excavated from the footing and utility trenches should not be spread over any areas of construction unless properly compacted. Soils silted into the footing excavations should be removed to the required depth prior to casting the concrete. The footings should be cast as soon as possible to avoid deep desiccation of the footing subsoils.

5.7.6. Premoistening

Footing subsoils should be kept in a moist condition preferably at 3% over the optimum moisture content for a depth of 18 inches below the bottom of the footing. Saturated soils or soils silted into the footing excavations should be removed prior to concrete placement.

5.8. SLABS-ON-GRADE

5.8.1. Site Preparation

The slab-on-grade subgrade, if disturbed during foundation and utility construction, should be conditioned prior to placement of aggregate materials. Loose soils should be removed to firm in-place material, the exposed subgrade processed, and the material replaced as engineered compacted fill or aggregate material.

5.8.2. Slab-on-Grade Design Data

Interior concrete slabs on-grade not used for structural support should be 5 inches thick and underlain by 6-inch-thick layer of ½ inch or larger clean aggregate or per applicable building codes, whichever is the more restrictive. The slab should be reinforced with a minimum of number 4 bars at 18-inch centers in each direction. The reinforcement should be placed and kept at slab mid-depth.

Exterior concrete slabs-on-grade (non-auto traffic) and walkways should be a minimum of 4 inches thick and underlain by a minimum of 4 inches of sand. In areas of heavy loading for truck traffic (including trash pickup areas and loading docks) the slab thickness should be increased to a minimum of 7 inches thick. Exterior slabs should be reinforced with a minimum of No. 3 bars on 24-inch centers in each direction. The reinforcement should be placed at mid-depth of the slab. Sidewalks may be constructed of non-reinforced concrete provided the sidewalks are cut into square panels (i.e., 4-foot wide walks should be cut into 4 foot by 4 foot squares). A deepened edge should be considered on all exterior slabs (non-auto traffic) to prevent water from entering the sand base. The edge should extend a minimum of 2 inches into the subgrade soils.

5.8.3. Premoistening

Soils under lightly loaded slabs on-grade should be kept in a moist condition preferably at 3% over the optimum moisture content for a depth of 18 inches.

5.8.4. Concrete Placement and Cracking

Minor cracking of concrete slabs is common and is generally the result of concrete shrinkage continuing after construction. Concrete shrinks as it cures resulting in shrinkage tension within the concrete mass. Since concrete is weak in tension, development of tension results in cracks within the concrete. Therefore, the concrete should be placed using procedures to minimize the cracking within the slab. Shrinkage cracks can become excessive if water is added to the concrete above the allowable limit and proper finishing and curing practices are not followed. Concrete mixing, placement, finishing, and curing should be performed per the American Concrete Institute Guide for Concrete Floor and Slab Construction (ACI 302.1R). Concrete slump during concrete placement should not exceed the design slump specified by the structural engineer. Concrete slabs on grade should be provided with tooled crack control joints at 10-15 foot centers or as specified by the structural engineer.

5.8.5. Moisture Vapor Barrier

Moisture migration occurs when there is a differential potential in the relative moisture below and above the concrete slab on grade. Therefore, concrete slabs on grade within the building interior should be considered sensitive to moisture and an appropriate moisture vapor retarder layer should be installed and maintained below concrete slabs-on-grade. The water vapor retarder should be one that is specifically designed as a vapor retarder and consist of a minimum 15 mil extruded polyolefin plastic and complying with Class A requirements under ASTM E1745 (*Standard Specification for Plastic Water Vapor Retarders Used in Contact with Soil or Granular Fill under Concrete Slabs*). The vapor retarder should be installed in accordance with ASTM E1643. The water vapor retarder should be installed in direct contact with the concrete slab along with a concrete mix design to control bleeding, shrinkage, and curling (ACI 302.2R). The vapor retarder shall be installed over a minimum 6-inch-thick layer of ½ inch or larger clean aggregate or per applicable building codes, whichever is the more restrictive. The vapor retarder should be placed per ASTM E1643-98(2005) *Standard Practice for Installation of Water Vapor Retarders Used in Contact with Earth or Granular Fill Under Concrete Slabs*. In addition, various trades and the concrete contractor should be required to protect the moisture retarder during construction.

Panel joints should be lapped and sealed. Perforations through the moisture vapor retarder such as at pipes, conduits, columns, grade beams, and wall footing penetrations should be sealed per the manufacture's specifications or ASTM E1643. Proper construction practices should be followed during construction of slabs on-grade. Repair and seal tears or punctures in the moisture barrier that may result from the construction process prior to concrete placement.

Minimizing shrinkage cracks in the slab on-grade can further minimize moisture vapor emissions. A properly cured slab utilizing low-slump concrete will reduce the risk of shrinkage cracks in the slab as described herein.

The concrete contractor should make the necessary changes in the concrete placement and curing for concrete placed directly over the retarder. Placing the concrete directly on top of the moisture vapor retarder layer allows the layer to be observed for damage directly prior to concrete placement.

The slabs should be tested for moisture content prior to the selection of the flooring and adhesives. Moisture in the slabs should not exceed the flooring manufacture's specifications. The concrete surface should be sealed per the manufacture's specifications if the moisture readings are excessive. It may be necessary to select floor coverings that are applicable to high moisture conditions.

5.9. BASEMENT RETAINING WALLS

5.9.1. Foundations

Allowable bearing capacities and lateral resistance provided herein for conventional footings may be used for retaining/subterranean wall design.

5.9.2. Lateral Earth Pressures

Retaining walls restrained at the top should be designed for a minimum lateral earth pressure equal to 30H with a trapezoidal distribution. The lateral earth pressure at the ground surface may be taken as zero. The pressure will increase with depth to 30H at a depth of .2H below the ground surface. The pressure would then extend uniformly at 30H to a depth of 0.8H and decrease uniformly to zero at the base of the excavation. (see Figure 4 on the following page). H is the supported height of wall. The resultant of 30H is in units of psf. Surcharges from adjacent loading should be added to the wall pressure.

As an alternate to a trapezoidal pressure distribution, basement walls may be design using an equivalent at rest pressure of 60 pounds per cubic foot. The resultant may be applied at one third the wall height measured from the bottom of the wall.

5.9.3. Lateral Seismic Pressure

The restrained basement wall lateral seismic soil pressure for walls over 6 feet high is 26 pcf and is calculated as $\Delta\text{Pae} = ½\ \gamma H^2\ (0.68\ \text{PGA}_\text{M}/\text{g})$ (where PGA_M = .66 and γ in-situ = 118 pcf). Walls should be designed for a total seismic load of the static and dynamic load increments. The seismic pressure of 26 pcf is a triangular pressure with the base of the triangle at the base of the wall. For the onsite soils, the point of application may be 1/3H from the base of the wall.

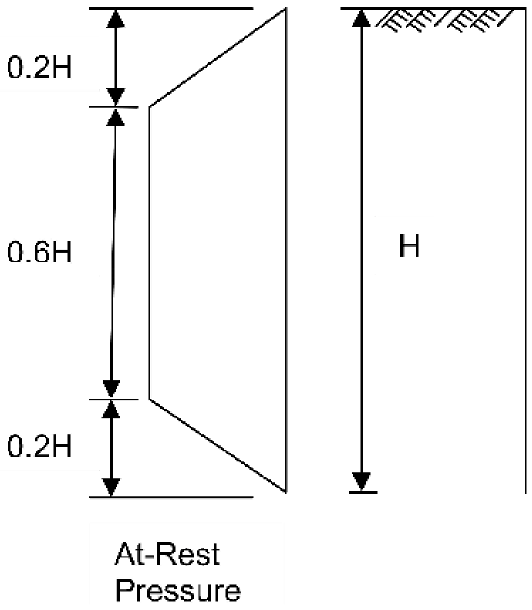


Figure 4

5.9.4. Waterproofing

Basement walls should be waterproofed on the exterior in addition to installing the drainage system and wall backfill. The waterproofing and backdrain system should be designed by a waterproofing consultant experienced with this type of structure.

5.9.5. Drainage

A drainage system should be constructed behind the basement and site retaining walls to relieve buildup of hydrostatic pressures. The drainage system for basement walls may consist of a prefabricated drainage composite consisting of a filter fabric bonded to a corrugated panel. The drainage system should extend to within 2 feet of finish grade with the upper 2 feet backfilled with native material. The drainage system should be hydraulically connected to a perimeter pipe drain consisting of a minimum 4-inch diameter perforated PVC (Schedule 40) pipe or equivalent. The pipe may be laid horizontally on the footing; however, the pipe invert should be at least 6 inches below the top of slab-on-grade. The outlet pipe from the perimeter drain should be a non-perforated 4-inch diameter PVC (Schedule 40) pipe that is sloped to and connected to a storm drain system or sump. An as-built plan should be prepared detailing the location of the wall drainage system.

5.9.6. Backfilling

Basement walls should be backfilled where necessary with granular material or soils having a low expansion potential. The backfill should be placed in 6-inch lifts at slightly over optimum moisture content and compacted to at least 90% relative compaction. If the backcut is flatter than ½(h):1(v), the backfill should be benched into the backcut slope. Light equipment should be used immediately behind the walls to prevent possible over-stressing. Bracing needed to resist basement wall movement should be in-place prior to placing the backfill.

5.10. SOIL CORROSIVITY

The results of the analytical laboratory testing to evaluate the potential for corrosion of materials in contact with the onsite soils are presented in Appendix B. The testing was performed on a soil sample considered to represent the onsite soils. From ACI Table 19.3.1.1 the evaluated soil is categorized as Class S0. The required concrete design requirements for this exposure class can be obtained from ACI Table 19.3.2.1. The potential for corrosion of metals in contact with the site soils is mildly corrosive as determined from Table 1 in Appendix B. For specific recommendations, a corrosion engineer should be consulted.

5.11. SITE DRAINAGE

Positive drainage should be continuously provided and maintained away from the structure during and after construction in accordance with applicable building codes and/or the approved grading plan. Regarding landscaping, planters adjacent a structure should be constructed so that irrigation water will not saturate the soils behind the basement walls or underlying the building footings and slabs. Trees should not be planted adjacent a structure where roots could grow under the foundations or slabs.

5.12. GUTTERS AND DOWNSPOUTS

Gutters and downspouts should be installed on the building to collect roof water and direct the water away from the structure. Downspouts should drain into PVC collector pipes that will carry the water away from the building.

5.13. PAVEMENT DESIGN

The anticipated structural section would be 3 inches of asphaltic concrete over 6 inches of aggregate base for parking areas. The structural section should be increased to be 3 inches of asphaltic concrete over 8 inches of aggregate base for drive areas. The final structural sections should be confirmed at the conclusion of grading. The upper 6 inches of subgrade and the base materials should be compacted to at least 90% and 95% of the maximum dry density, respectively.

Planter areas should be graded so excess water drains away from adjacent AC pavement and curbs. Also, adjacent the planters, consideration should be given to deepening the curbs so that water is not allowed to saturate the pavement subgrade.

5.14. PLAN REVIEW(S)

As the development process continues and final detailed grading and site/foundation plans and specifications are developed, they should be reviewed by Gorian and Associates, Inc. Additional geotechnical recommendations may be warranted at that time.

6. CLOSURE

This report was prepared under the direction of State registered geotechnical engineer for the addressee and design consultants solely for design and construction of the project as described herein. No warranty, express or implied, is made as to conclusions and professional advice included in this report. Gorian and Associates, Inc. disclaim any and all responsibility and liability for problems that may occur if the recommendations presented in this report are not followed.

This report may not contain sufficient information for other uses or the purposes of other parties. Recommendations should not be extrapolated to other areas or used for other facilities without consulting Gorian and Associates, Inc. Services of this office should not be construed to relieve the owner or contractors of their responsibilities or liabilities.

The scope of the services provided by Gorian and Associates, Inc. and its staff, excludes responsibility and/or liability for work conducted by others. Such work includes, but is not limited to, means and methods of work performance, quality control of the work, superintendence, sequencing of construction and safety in, on, or about the jobsite.

The recommendations are based on interpretations of the subsurface conditions concluded from information gained from subsurface explorations and a surficial site reconnaissance. The interpretations may differ from actual subsurface conditions, which can vary horizontally and vertically across the site. Due to possible subsurface variations, this office should observe all aspects of field construction addressed in this report. Individuals using this report for bidding or construction purposes should perform such independent investigations as they deem necessary.

oOo

Please contact our office if you have questions regarding the information and recommendations contained in this report, or require additional consultation.

Respectfully submitted,

Gorian and Associates, Inc.

Jerome Blunck

By: Jerome J. Blunck, GE 151
Principal Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
JEROME JON BLUNCK
No. 151
GEOTECHNICAL
STATE OF CALIFORNIA

REFERENCES

Agusti, G.C., and Sitar, N., 2013. *"Seismic Earth Pressures on Retaining Structures in Cohesive Soils," Report submitted to the California Department of Transportation (Caltrans) under Contract No. 65A0367 and NSF-NEES-CR Grant No. CMMI-0936376: Seismic earth pressures on retaining structures*. Report No. UCB GT 13-02.

American Concrete Institute, ACI 318: *Building Code Requirements for Structural Concrete*.

ASCE/SEI 7-16 *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, Published by the American Society of Civil Engineers. 2017.

California Geological Survey (CGS), 2018, *Earthquake Fault Zones, A Guide for Government Agencies, Property Owners / Developers, and Geoscience Practitioners for Assessing Fault Rupture Hazards in California*. California Geological Survey Special Publication 42, Revised 2018. ftp://ftp.consrv.gov/pub/dmg/pubs/sp/Sp42.pdf.

California Geological Survey (CGS), 2008, *Guidelines for Evaluating and Mitigating Seismic Hazards in California. California Geologic Survey Special Publication 117A*. (revised March 2009).

California Geological Survey (CGS), *Earthquake Zones of Required Investigation*, CGS Homepage, Viewed online: https://maps.conservation.ca.gov/cgs/EQZApp/app/

California Division of Mines and Geology (CDMG) [now California Geological Survey (CGS)], 1975, *Guidelines for Evaluating the Hazard of Surface Rupture*. California Division of Mines and Geology Note Number 49.

California Division of Mines and Geology (CDMG) [now California Geological Survey (CGS)], 1995, *Supplement No. 1 to Special Publication 42* (1994 edition).

California Department of Water Resources, *California Groundwater Live*, Viewed online: (https://storymaps.arcgis.com/stories/b3886b33b49c4fa8adf2ae8bdd8f16c3).

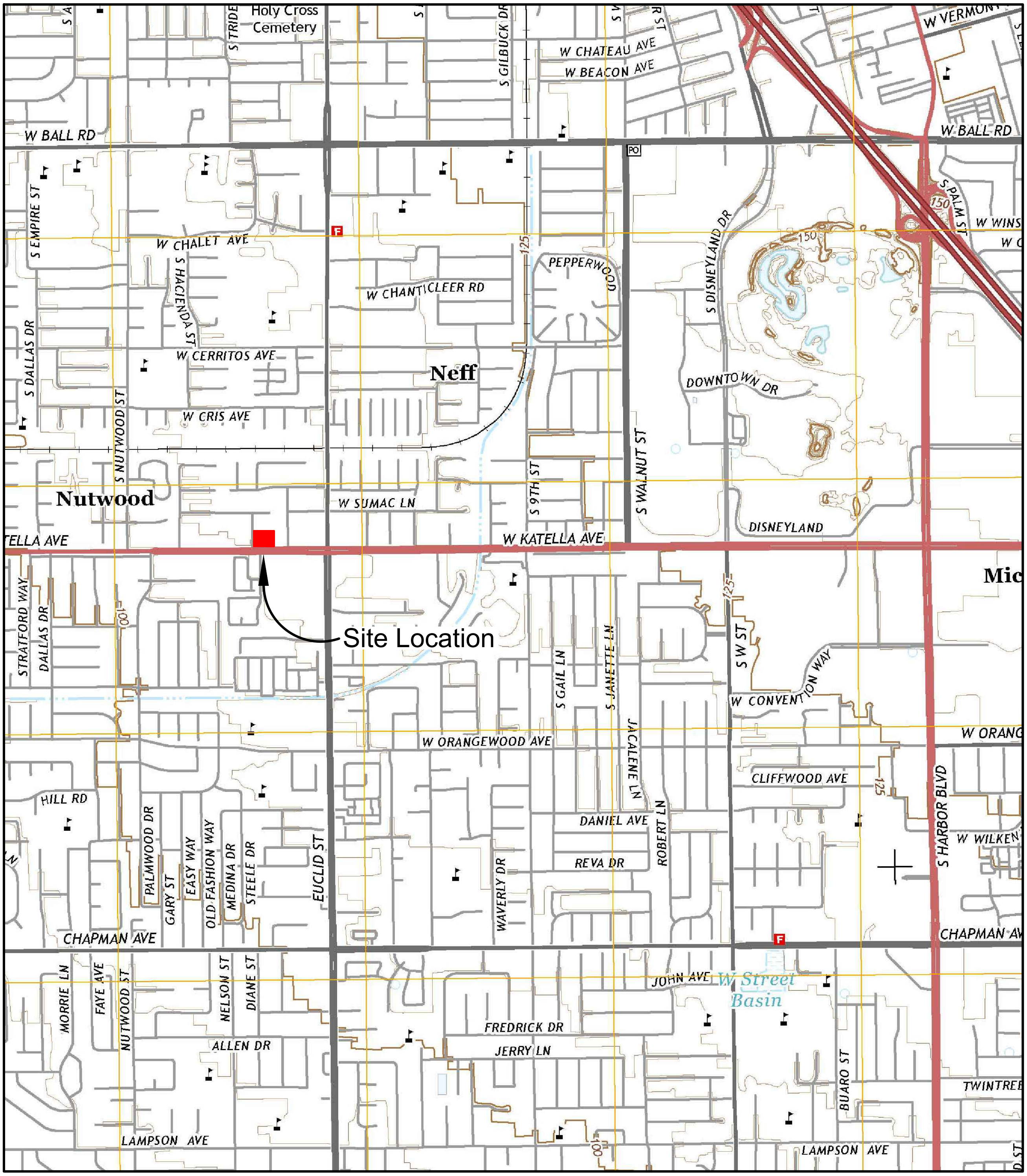
Federal Emergency Management Agency (FEMA), 2008, *FIRM Flood Inundation Zone Map*, Panel 06059C0137J (effective on12/03/2009). https://msc.fema.gov/portal/search?AddressQuery=1761%20W.%20Katella%20Ave.%2C%20Anaheim%2C%20CA#searchresultsanchor

Federal Highway Administration, *Soil Nail Walls, Reference Manual*, FHWA-NHI-14-007, FHWA GEC 007, February 2015.

Jennings, C.W. and Bryant, W.A., Compilers, 2010, *Fault Activity Map of California.* California Geological Survey Data Map Series, Map No. 6. Viewed online: < http://www.quake.ca.gov/gmaps/FAM/faultactivitymap.html >

Post Tensioning Institute *Recommendations for Prestressed Rock and Soil Anchors*, DC35.1-14

United States Geological Survey (USGS) interactive web application, *Unified Hazard Tool*. https://earthquake.usgs.gov/hazards/interactive/.



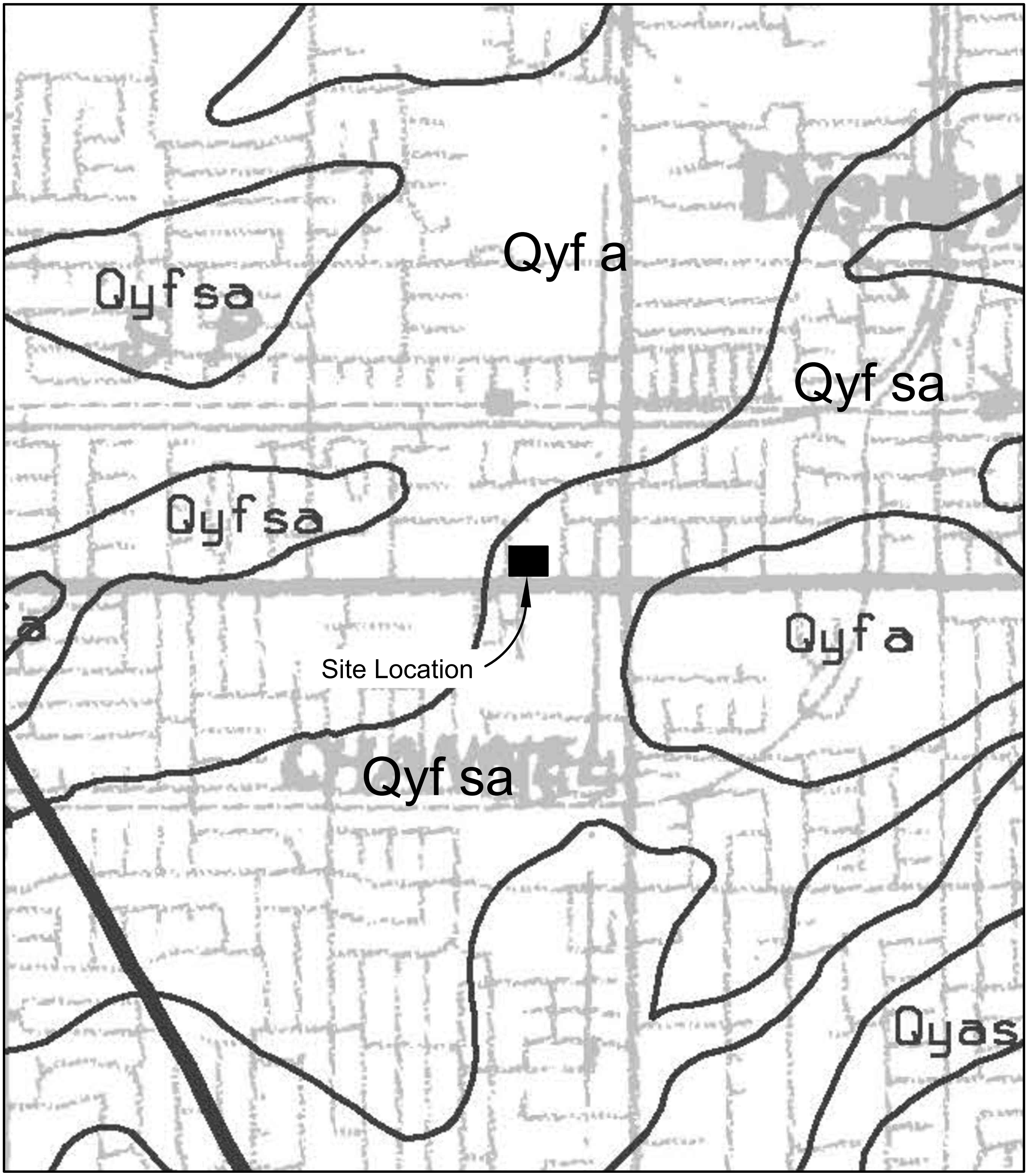
SITE VICINITY MAP

1761 West Katella Avenue
(APN 128-542-011)
Anaheim, California

Gorian & Associates, Inc.
Applied Earth Sciences

Job No: 3239-0-0-100
Date: March 2023
Scale: 1" = 2000'
Drawn by:
Approved by:
Figure 1

Source
USGS, Anaheim Quadrangle 7.5 minute series



Source: Map enlarged from U.S.G.S. 30X60-minute series

Explanation

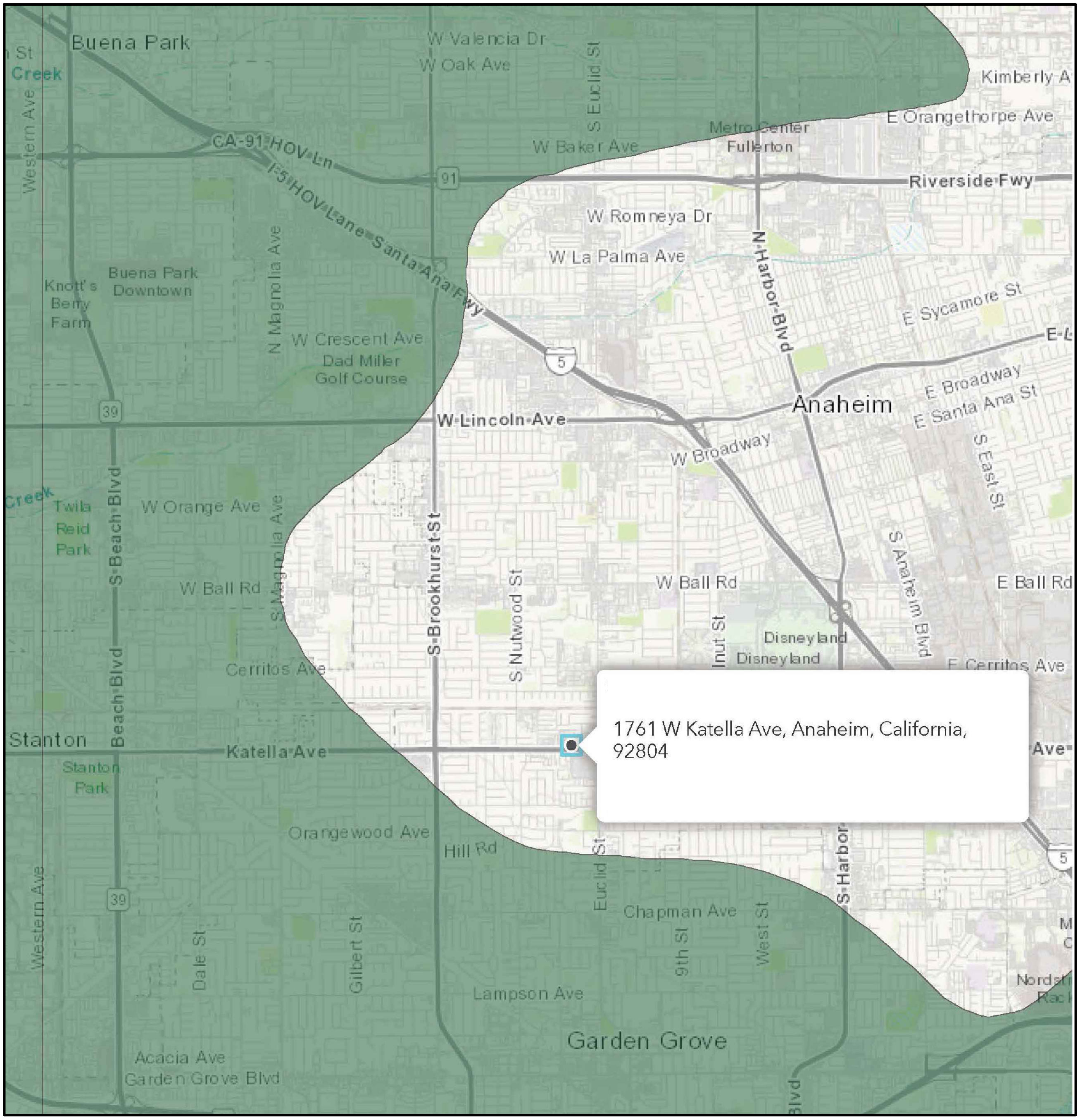
Qyf Young deposits of alluvial fans (Holocene and late Pleistocene)—Slightly consolidated to cemented, undissected to slightly dissected deposits of unsorted boulders, cobbles, gravel, and sand that form inactive parts of alluvial fans.

REGIONAL GEOLOGIC MAP

1761 West Katella Avenue
(APN 128-542-011)
Anaheim, California

Gorian & Associates, Inc.
Applied Earth Sciences

Job No: 3202-0-0-100		Date: March 2023
Scale: NTS	Drawn by: Approved by:	Figure 2



Explanation

🟩 Seismic Hazard Zone - Liquefaction

⬜ Outside of Seismic Hazard Zone

Source

CGS Homepage - Earthquake Zones of Required Investigation
https://maps.conservation.ca.gov/cgs/EQZApp/app/

SEISMIC HAZARD ZONE MAP

1761 West Katella Avenue
(APN 128-542-011)
Anaheim, California

Gorian & Associates, Inc.
Applied Earth Sciences

Job No: 3239-0-0-100		Date: March 2023
Scale: NTS	Drawn by: Approved by:	Figure 3

APPENDIX A

LOGS OF SUBSURFACE DATA

Project: Extra Space Storage
1761 W. Katella, Anaheim

Work Order: 3239-0-0-100

SUBSURFACE LOG

Excavation Number: B-1

Page Number: 1

Date(s) Excavated 03/16/2023	Logged By CHD	Excavation Location See Location Map	Approximate Surface Elevation
Excavation Dimension 8" Diameter	Equipment Contractor 2R Drilling	Equipment Type CME 75	Hammer Data 140# Auto

Elevation / Depth (ft.)	Bulk	Sample Type	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	USCS	Soil / Lithology	Description	Remarks
0								Asphaltic Concrete (2") on aggregate base (1") (damp, dense).	
	■					SM	▒	Alluvium: Brown very silty fine SAND (moist, medium dense)	
		■	11						
				12.9	106.4	ML	▥	Grayish brown SILT (moist, stiff).	
						CL	▨	Grayish brown silty CLAY (moist, very stiff).	
5		■	19	17.6	105.7				
	■					SM	▒	Yellowish brown to light gray silty fine SAND (moist, medium dense). Some thin silty interstratifications.	
		■	18	6.6	93.3				
10		■	19	12.7	88.9				
						SM	▒	Light yellowish brown silty fine SAND (damp, medium dense). Friable.	
15		■	18	2.1	115.6				
						CL	▨	Brown sandy CLAY (moist, very stiff).	
						SM	▒	Light yellowish brown silty fine SAND (damp, medium dense). Friable.	
20		■	21	3.9	112.9				
						CL	▨	Grayish brown silty CLAY (moist, very stiff).	
						ML	▥	Grayish brown SILT (moist, hard).	
25		■	35	14.4	110.2				
						SM	▒	Yellowish brown silty fine SAND (moist, medium dense).	
30		■	26	11.7	110.6				
35		■	21						
				19.6	102.6	ML	▥	Yellowish clayey SILT (moist, very stiff).	
						SM	▒	Light yellowish brown silty fine to medium SAND (damp, dense). Friable.	

Project: Extra Space Storage
1761 W. Katella, Anaheim

Work Order: 3239-0-0-100

SUBSURFACE LOG

Excavation
Number: B-1

Page Number: 2

Elevation / Depth (ft.)	Bulk	Sample Type	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	USCS	Soil / Lithology	Description	Remarks
40		■	32	1.6	105.1		▨		
								TOTAL DEPTH 41' No Caving Observed No Groundwater Encountered Backfilled with cuttings and tamped. AC Cold Patch on top	
45									
50									
55									
60									
65									
70									
75									
80									

Project: Extra Space Storage
1761 W. Katella, Anaheim

Work Order: 3239-0-0-100

SUBSURFACE LOG

Excavation Number: B-2

Page Number: 1

Date(s) Excavated 03/16/2023	Logged By CHD	Excavation Location See Location Map	Approximate Surface Elevation
Excavation Dimension 8" Diameter	Equipment Contractor 2R Drilling	Equipment Type CME 75	Hammer Data 140# Auto

Elevation / Depth (ft.)	Bulk	Sample Type	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	USCS	Soil / Lithology	Description	Remarks
0								Asphaltic Concrete (7.5")	
						GM		Artificial fill: Yellowish brown pea gravel (moist, loose).	
5	■	◺	5						
10		◺	6						
15		◺	6						
						ML		Alluvium: Grayish brown SILT (very moist to wet, stiff). Change at 17' per driller.	
20		■	12	26.9	89.7				
						SM		Light yellowish brown silty fine SAND (damp, dense). Friable.	
25		■	31	3.7	98.9				
30		■	30	7.4	98.4				
35		■	32	12.1	96.1			@35', becoming moist.	

Project: Extra Space Storage
1761 W. Katella, Anaheim

Work Order: 3239-0-0-100

SUBSURFACE LOG

Excavation Number: B-2

Page Number: 2

Elevation / Depth (ft.)	Bulk	Sample Type	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	USCS	Soil / Lithology	Description	Remarks
40		■	45	3.6	100.2		▨	@40', becoming damp.	
								TOTAL DEPT 41' Caving from -1' to -17' No Groundwater Encountered Backfilled with cuttings and tamped. AC Cold Patch on top.	
45									
50									
55									
60									
65									
70									
75									
80									

Project: Extra Space Storage
1761 W. Katella, Anaheim

Work Order: 3239-0-0-100

SUBSURFACE LOG

Excavation
Number: B-3

Page Number: 1

Date(s) Excavated 03/16/2023	Logged By CHD	Excavation Location See Location Map	Approximate Surface Elevation
Excavation Dimension 8" Diameter	Equipment Contractor 2R Drilling	Equipment Type CME 75	Hammer Data 140# Auto

Elevation / Depth (ft.)	Bulk	Sample Type	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	USCS	Soil / Lithology	Description	Remarks
0	■					SM		Asphaltic Concrete (3") on Aggregate Base (4") (damp, dense). Alluvium: Brown to yellowish brown silty fine SAND (moist, medium dense).	
		■	19	11.4	109.4				
	■								
5		■	16						
				7.7	114.1	SM		Grayish brown silty fine SAND (moist, medium dense).	
				5.3	98.7	SM		Light yellowish brown silty fine SAND (damp, medium dense).	
		■	15						
						ML		Light yellowish brown SILT (moist, very stiff).	
						SM		Light gray silty fine SAND (damp, medium dense).	
10		■	15	5.5	90.9				
						ML		Yellowish brown SILT (moist, very stiff) .	
15		■	17	10.6	104.4				
						SM		Yellowish brown silty fine SAND (damp, medium dense). Friable.	
						ML		Yellowish brown clayey SILT (very moist, very stiff).	
20		■	24	34.0	78.7				
						SM		Light gray very silty very fine SAND (damp, medium dense).	
						SM		Pale yellow silty fine to coarse SAND (damp, medium dense). Friable. Few fine gravels.	
25		■	25	0.9	107.1				
						SM		Light yellowish brown silty fine SAND (damp, dense).	
30		■	36	3.1	101.6				
35		■	32	3.5	101.2			Below 35', some silty fine to coarse SAND, friable.	

Project: Extra Space Storage
1761 W. Katella, Anaheim

Work Order: 3239-0-0-100

SUBSURFACE LOG

Excavation
Number: B-3

Page Number: 2

Elevation / Depth (ft.)	Bulk	Sample Type	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	USCS	Soil / Lithology	Description	Remarks
40		■	33	1.6	99.5		▨		
								TOTAL DEPTH 41' No Caving Observed No Groundwater Encountered Backfilled with cuttings, tamped. AC Cold Patch on top.	
45									
50									
55									
60									
65									
70									
75									
80									

Project: Extra Space Storage
1761 W. Katella, Anaheim

Work Order: 3239-0-0-100

SUBSURFACE LOG

Excavation Number: IB-1

Page Number: 1

Date(s) Excavated 03/16/2023	Logged By CHD	Excavation Location See Location Map	Approximate Surface Elevation
Excavation Dimension 8" Diameter	Equipment Contractor 2R Drilling	Equipment Type CME 75	Hammer Data 140# Auto

Elevation / Depth (ft.)	Bulk	Sample Type	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	USCS	Soil / Lithology	Description	Remarks
0								Asphaltic Concrete (2') on Aggregate Base (6") (damp, dense).	
						SM	░	Alluvium: Brown grading to yellowish brown silty fine SAND (moist). Locally interstratified with silt to clayey silt.	
5							░		
10						ML	▥	Light yellowish brown clayey SILT (moist, very stiff).	
15		◩	6/8/ 8				▥		
						SM	░	Light yellowish brown silty fine SAND (damp, medium dense). Friable.	
		◩	5/5/ 7			ML	▥	Yellowish brown sandy clayey SILT (moist, stiff).	
20							▥		
								TOTAL DEPTH 21' No Caving Observed No Groundwater Encountered Boring converted into infiltration test well. -21' to -20', medium bentonite chips. -20' to -15', 2" slotted (0.02) pipe. -15' to ground surface, 2" solid pipe -20' to -13', #3 sand. -13' to -12', medium bentonite chips. -12' to ground surface, cuttings. Presoaked.	
25									
30									
35									

Project: Extra Space Storage
1761 W. Katella, Anaheim

Work Order: 3239-0-0-100

SUBSURFACE LOG

Excavation Number: IB-2

Page Number: 1

Date(s) Excavated 03/16/2023	Logged By CHD	Excavation Location See Location Map	Approximate Surface Elevation
Excavation Dimension 8" Diameter	Equipment Contractor 2R Drilling	Equipment Type CME 75	Hammer Data 140# Auto

Elevation / Depth (ft.)	Bulk	Sample Type	Blow Counts	Moisture Content (% dry weight)	Dry Density (pcf)	USCS	Soil / Lithology	Description	Remarks
0								Asphaltic Concrete (2") on Aggregate Base (4") (damp, dense).	
						ML		Alluvium: Brown clsyey SILT (very moist to wet).	
5						SM		Yellowish brown silty fine SAND (damp, loose).	
		◣	5/5/4						
						ML		Light gray clayey SILT (moist, stiff).	
10									
		◣	3/6/6			CL		Gray sandy silty CLAY (moist, stiff).	
						SM		Yellowish brown silty fine SAND (moist, medium dense).	
15								TOTAL DEPTH 14' No Caving Observed No Groundwater Encountered Boring converted into infiltration test well. -14' to -13', medium bentonite chips. -13' to -8', 2" slotted (0.02) pipe. -8' to ground surface, 2" solid pipe. -13' to -6', #3 sand. -6' to -5', medium bentonite chips.-5' to ground surface, cuttings. Presoaked.	
20									
25									
30									
35									

APPENDIX B
LABORATORY TESTING

General

Laboratory test results on selected samples are presented below. Test were performed to evaluate the physical and engineering properties of the encountered earth materials, including in-situ moisture content and dry density, optimum moisture-maximum dry density relationships, expansion potential, consolidation characteristics, and shear strength parameters. Soil corrosivity testing was performed under subcontract by a corrosion engineer.

Density and Moisture Tests

In situ dry density and moisture content were determined for each undisturbed soil sample. The results are presented on the Logs of Subsurface Data (Appendix A).

Maximum Density-Optimum Moisture

A maximum density/optimum moisture test (compaction characteristics) was performed on a selected bulk sample of the soils encountered. The test was performed in general accordance with ASTM D 1557. The results are as follows:

Boring Number	**Depth** (feet)	**Visual Classification**	**Maximum Dry Density – pcf**	**Optimum Moisture Content - %**
B-1	6.0	Yellowish brown silty fine sand	125.6	10.1

Soil Expansiveness

An expansion index test was performed on a soil sample obtained from the borings to evaluate expansion potential of the subgrade soils in general accordance with the Expansion Index Test method (ASTM test method D4829-08a). The results are as follows:

Boring Number	**Depth** (feet)	**Expansion Index**	**Expansion Range**
B-1	6.0	0	0-20

Direct Shear Test

Direct shear tests were performed on four relatively undisturbed samples to evaluate soil shear strength parameters. The sample set was sheared under normal pressures ranging from 1000 to 4000 pounds per square foot. The results are graphically presented herein.

Consolidation Test

Three consolidation tests were performed on selected samples of the soils below anticipated foundation depths to evaluate compressibility characteristics. The sample was loaded in increments to a maximum of 8,000 pounds per square foot and then rebounded. The sample was inundated at the indicated overburden pressure to evaluate the effect of moisture infiltration on compression behavior. The load-consolidation curve is presented herein as a graphic summaries.

Corrosion Testing

The results of the analytical laboratory testing to evaluate the potential for corrosion of materials in contact with the onsite soils are presented in this appendix. The testing was performed on a soil sample considered to represent the onsite soils. From ACI Table 19.3.1.1 the evaluated soil is categorized as Class S0. The required concrete design requirements for this exposure class can be obtained from ACI Table 19.3.2.1. The potential for corrosion of metals in contact with the site soils is mildly corrosive as determined from Table 1 below. For specific recommendations, a corrosion engineer should be consulted.

ACI Table 19.3.1.1 – Exposure Categories and Classes

Category	Class	Water-soluble sulfate (SO_4^{2-}) in soil, percent by mass	Dissolved sulfate (SO_4^{2-}) in water, ppm[1]
Sulfate (S)	S0	$SO_4^{2-} < 0.10$	$SO_4^{2-} < 150$
	S1	$0.10 \le SO_4^{2-} < 0.20$	$150 \le SO_4^{2-} < 1500$ or seawater
	S2	$0.20 \le SO_4^{2-} < 2.00$	$1500 \le SO_4^{2-} < 10{,}000$
	S3	$SO_4^{2-} > 2.00$	$SO_4^{2-} > 10{,}000$

1 ppm (parts per million) = milligrams per kilogram mg/kg of dry soil weight

ACI Table 19.3.2.1 – Requirements for Concrete by Exposure Class

Exposure Class	Maximum *w/cm*	Minimum f_c', psi	Cementitious materials - Types			Calcium chloride admixture
			ASTM C150	ASTM C595	ASTM C1157	
S0	N/A	2500	No type restriction	No type restriction	No type restriction	No restriction
S1	0.50	4000	II	Types IP, IS, or IT with (MS) designation	MS	No restriction
S2	0.45	4500	V	Types IP, IS, or IT with (MS) designation	HS	Not permitted
S3	0.45	4500	V plus pozzolan or slag cement	Types IP, IS, or IT with (MS) designation plus pozzolan or slag cement	HS plus pozzolan or slab cement	Not permitted

ACI Tables 19.3.1.1 and 19.3.2.1 - ACI 318-14 Building Code Requirements for Structural Concrete

Table 1. Relationship Between Soil Resistivity and Soil Corrosivity

Soil Resistivity, ohm-cm	Classification of Soil Corrosiveness
0 to 900	Very severe corrosion
900 to 2,300	Severely corrosive
2,300 to 5,000	Moderately corrosive
5,000 to 10,000	Mildly corrosive
10,000 to >10,000	Very mildly corrosive

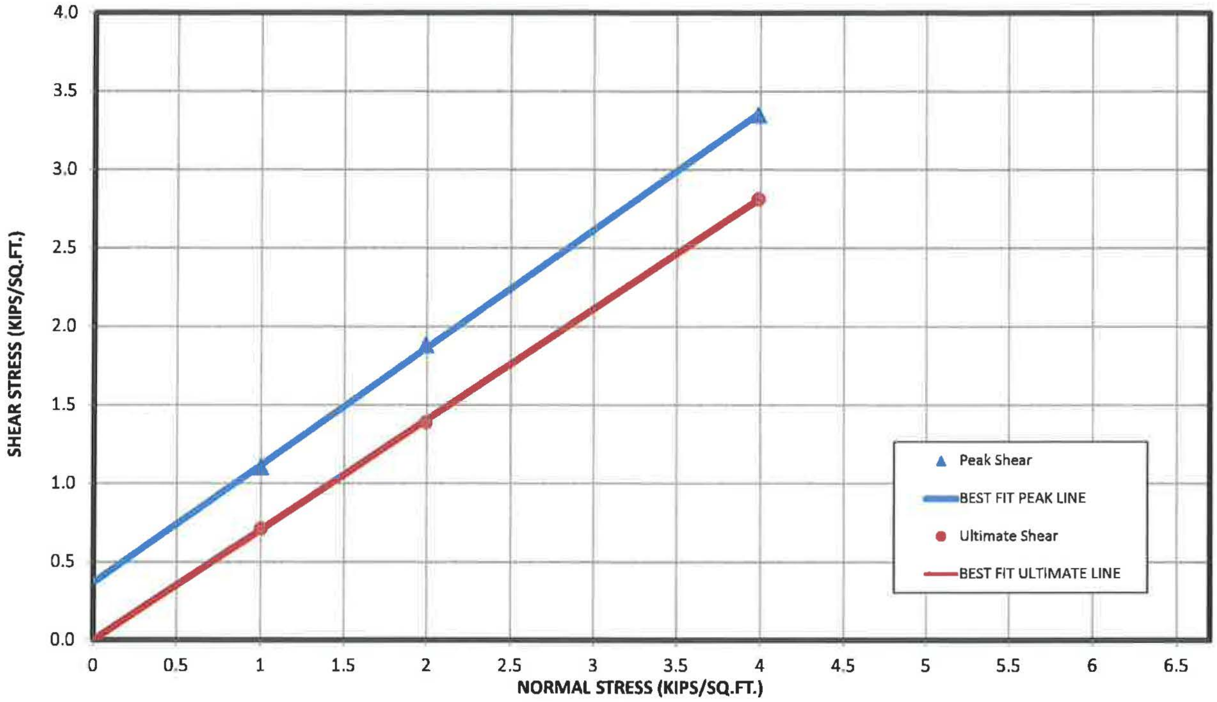
F. O. Waters, Soil Resistivity Measurements for Corrosion Control, Corrosion. 1952, Vol, No. 12, 1952, p. 407.

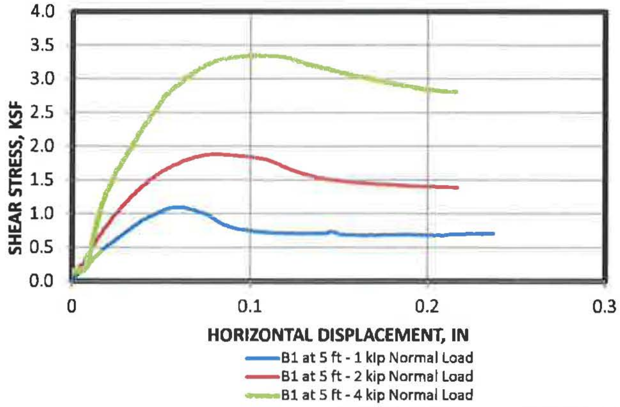
Grain Size Distribution

Grain size distribution was determined for two soil samples below the depth of the intended stormwater infiltration. The graphed results are attached hereto.

DIRECT SHEAR TEST RESULTS

Undisturbed Sample





TEST DATA:	#1	#2	#3
NORM. PRES. (KSF)	1.0	2.0	4.0
ULTIMATE			
SHEAR STRESS (KSF):	0.71	1.39	2.81
H.DISPL. (IN)	0.26	0.25	0.24
DISP. RATE (IN/MIN)	0.01	0.01	0.01
PEAK			
SHEAR STRESS (KSF):	1.10	1.88	3.35
H.DISPL. (IN)	0.08	0.11	0.13

PROJECT: Extra Space 1761 Katella
W.O: 3239.-0-0-100
EXCAVATION: B1
DEPTH: 5 ft

	PEAK	ULT.	RES.
COHESION (KSF):	0.375	0.000	
PHI (DEG):	37	35	

PRESHEAR DRY DENSITY (PCF):	105.8	106.1	107.6
PRESHEAR MOISTURE (% OF DD):		23.0	
EST.VOID RATIO, e (preshear):	0.53	0.53	0.51

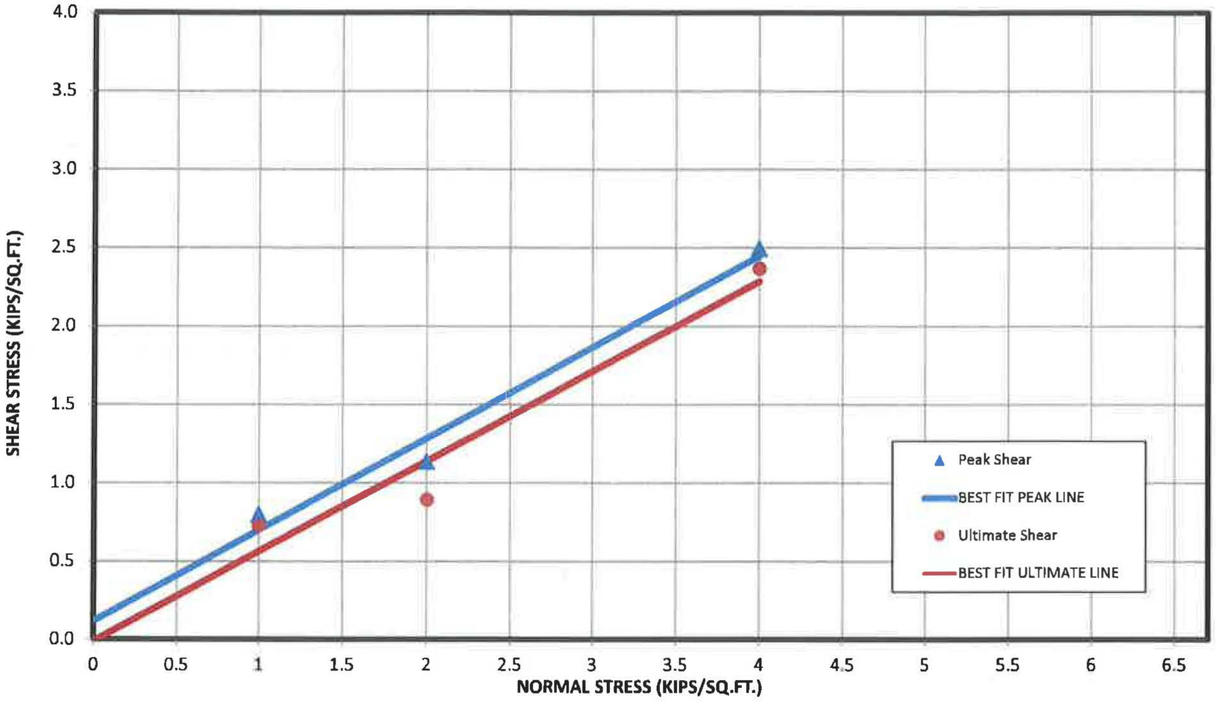
TEST FILES:

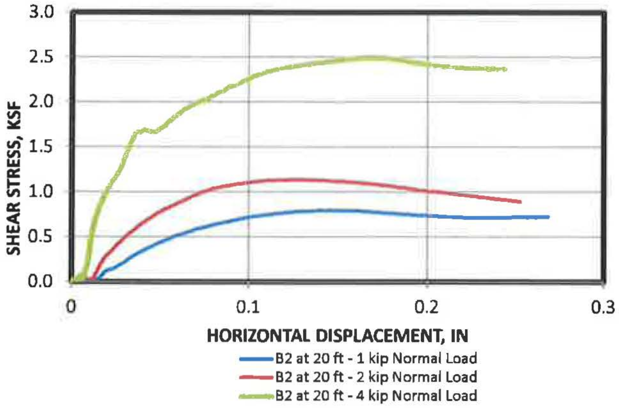
S:\GEOTEST\shears\GORIAN\TEST807.DAT
S:\GEOTEST\shears\GORIAN\TEST808.DAT
S:\GEOTEST\shears\GORIAN\TEST809.DAT

GORIAN
& ASSOCIATES, INC.

DIRECT SHEAR TEST RESULTS

Undisturbed Sample





PROJECT:	**Extra Space 1761 Katella**
W.O:	**3239.-0-0-100**
EXCAVATION:	**B2**
DEPTH:	**20 ft**

TEST DATA:	#1	#2	#3
NORM. PRES. (KSF)	1.0	2.0	4.0
ULTIMATE			
SHEAR STRESS (KSF):	0.73	0.89	2.37
H.DISPL. (IN)	0.27	0.27	0.25
DISP. RATE (IN/MIN)	0.01	0.01	0.01
PEAK			
SHEAR STRESS (KSF):	0.80	1.14	2.50
H.DISPL. (IN)	0.14	0.13	0.17

	PEAK	ULT.	RES.
COHESION (KSF):	0.125	0.000	
PHI (DEG):	30	30	

PRESHEAR DRY DENSITY (PCF):	92.8	93.5	110.9
PRESHEAR MOISTURE (% OF DD):		40.0	
EST.VOID RATIO, e (preshear):	0.75	0.74	0.46

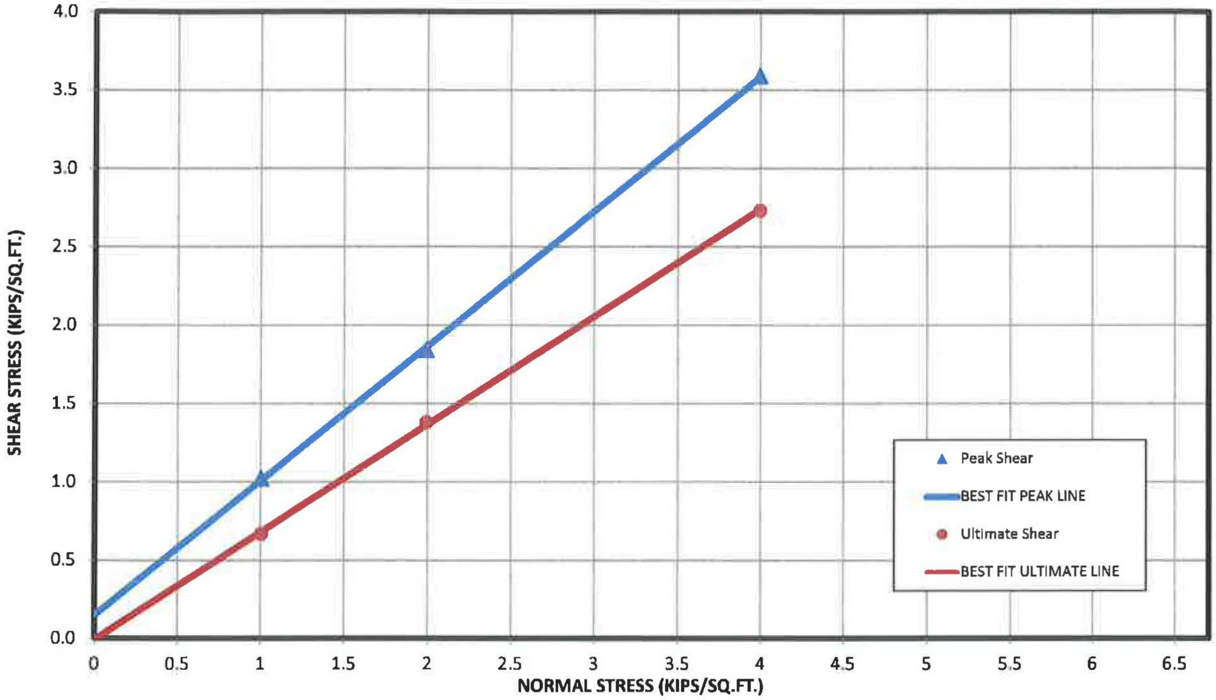
TEST FILES:

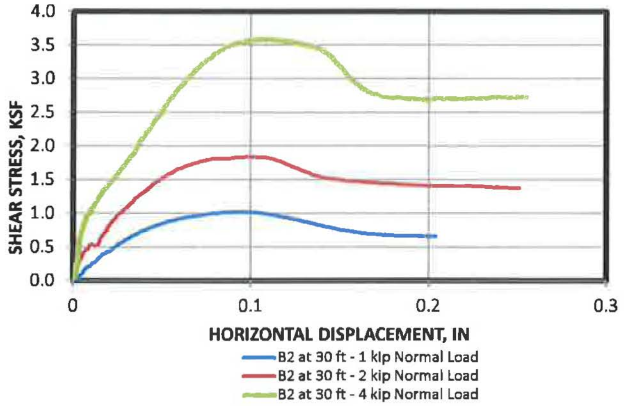
S:\GEOTEST\shears\GORIAN\TEST813.DAT
S:\GEOTEST\shears\GORIAN\TEST814.DAT
S:\GEOTEST\shears\GORIAN\TEST815.DAT

GORIAN
& ASSOCIATES, INC.

DIRECT SHEAR TEST RESULTS

Undisturbed Sample





TEST DATA:	#1	#2	#3
NORM. PRES. (KSF)	1.0	2.0	4.0
ULTIMATE			
SHEAR STRESS (KSF):	0.67	1.38	2.73
H.DISPL. (IN)	0.21	0.26	0.26
DISP. RATE (IN/MIN)	0.01	0.01	0.01
PEAK			
SHEAR STRESS (KSF):	1.02	1.84	3.59
H.DISPL. (IN)	0.10	0.10	0.11

PROJECT:	**Extra Space 1761 Katella**
W.O:	**3239.-0-0-100**
EXCAVATION:	**B2**
DEPTH:	**30 ft**

	PEAK	ULT.	RES.
COHESION (KSF):	0.150	0.000	
PHI (DEG):	41	34	

PRESHEAR DRY DENSITY (PCF):	116.9	110.0	115.5
PRESHEAR MOISTURE (% OF DD):		27.0	
EST.VOID RATIO, e (preshear):	0.39	0.48	0.41

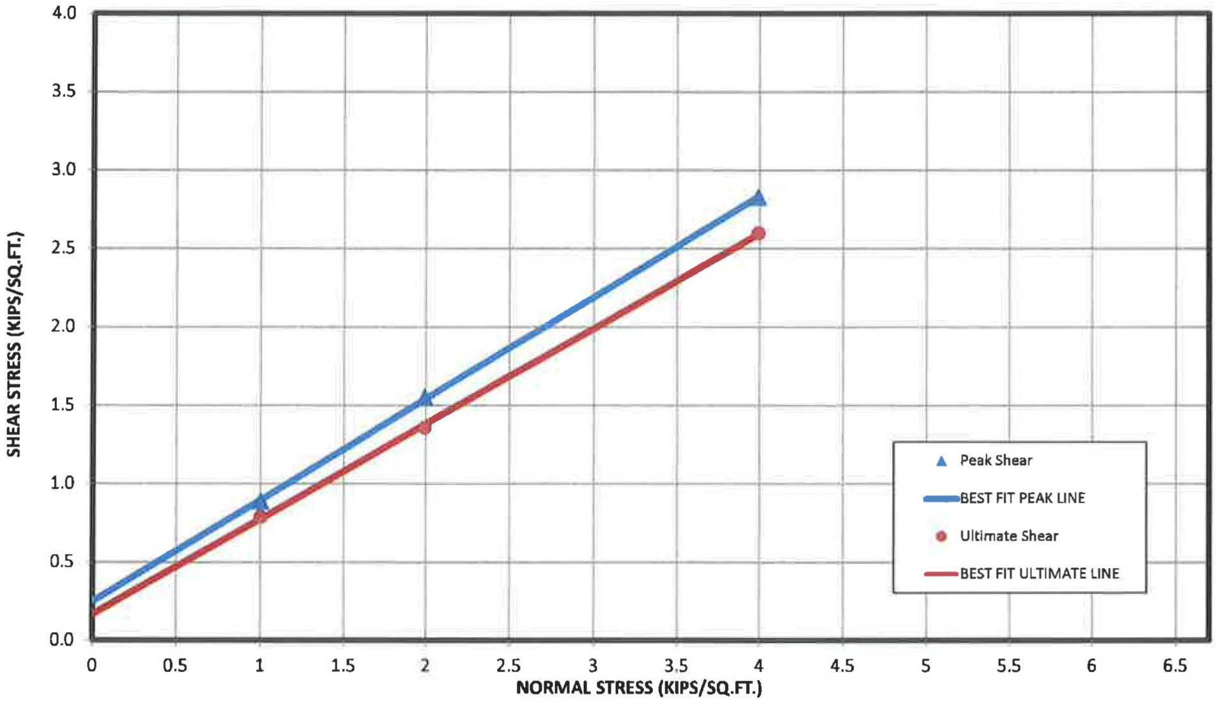
TEST FILES:

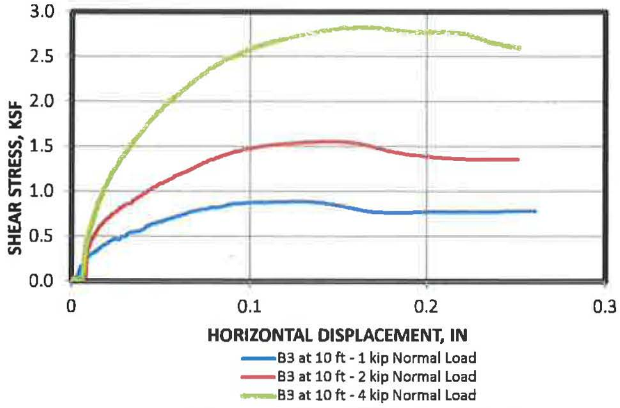
S:\GEOTEST\shears\GORIAN\TEST816.DAT
S:\GEOTEST\shears\GORIAN\TEST817.DAT
S:\GEOTEST\shears\GORIAN\TEST818.DAT

GORIAN
& ASSOCIATES, INC.

DIRECT SHEAR TEST RESULTS

Undisturbed Sample





TEST DATA:	#1	#2	#3
NORM. PRES. (KSF)	1.0	2.0	4.0
ULTIMATE			
SHEAR STRESS (KSF):	0.79	1.36	2.60
H.DISPL. (IN)	0.26	0.26	0.26
DISP. RATE (IN/MIN)	0.01	0.01	0.01
PEAK			
SHEAR STRESS (KSF):	0.89	1.55	2.83
H.DISPL. (IN)	0.12	0.15	0.17

PROJECT:	Extra Space 1761 Katella
W.O:	3239.-0-0-100
EXCAVATION:	B3
DEPTH:	10 ft

	PEAK	ULT.	RES.
COHESION (KSF):	0.250	0.175	
PHI (DEG):	33	31	

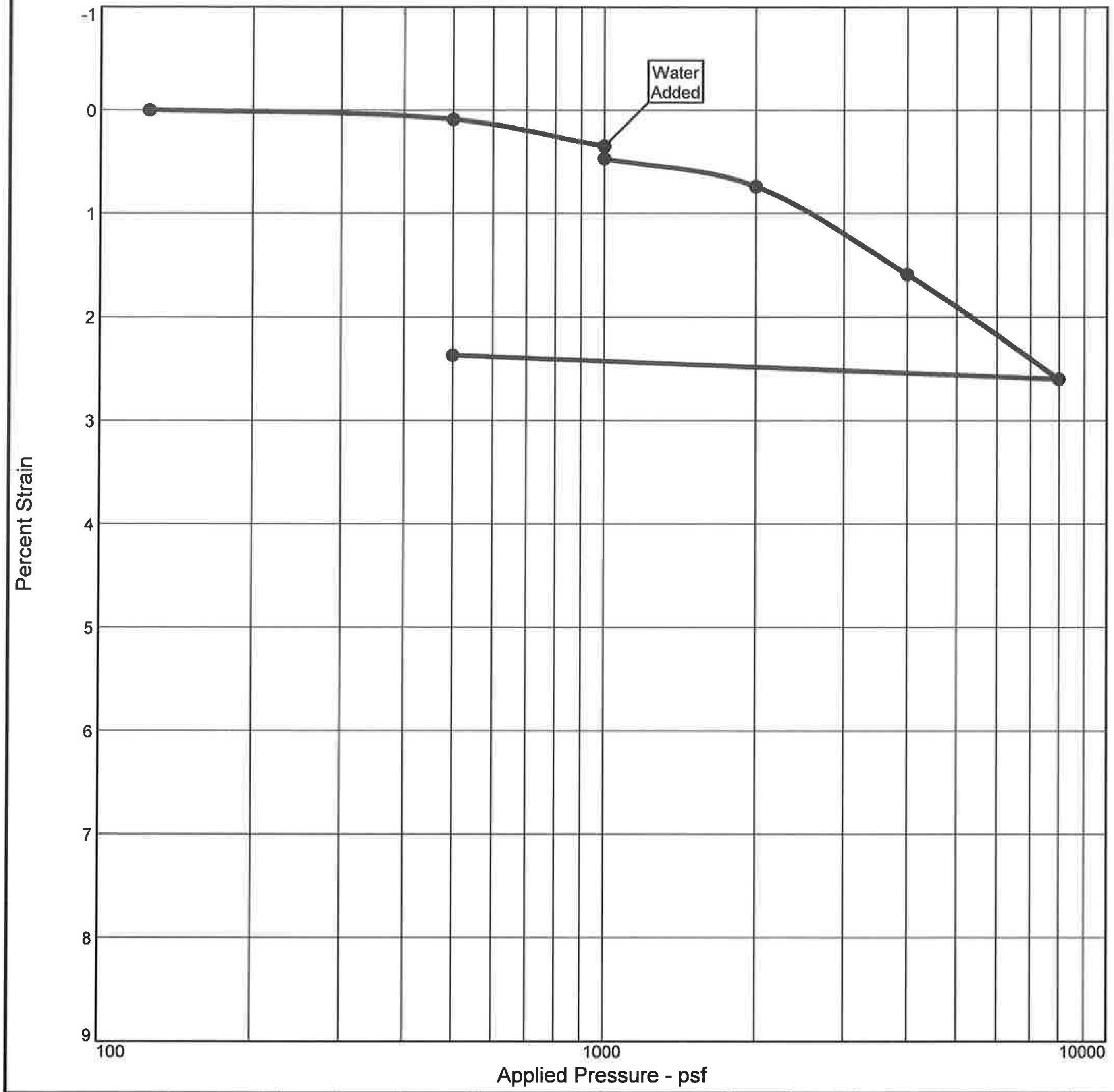
PRESHEAR DRY DENSITY (PCF):	90.9	90.9	91.1
PRESHEAR MOISTURE (% OF DD):		31.0	
EST.VOID RATIO, e (preshear):	0.78	0.78	0.78

TEST FILES:

S:\GEOTEST\shears\GORIAN\TEST819.DAT
S:\GEOTEST\shears\GORIAN\TEST820.DAT
S:\GEOTEST\shears\GORIAN\TEST821.DAT

GORIAN
& ASSOCIATES, INC.

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (psf)	P_c (psf)	C_c	C_r	Swell Press. (psf)	Clpse. %	e_o
Sat.	Moist.											
							2207				0.1	

MATERIAL DESCRIPTION	USCS	AASHTO

Project No. 3239-0-0- **Client:** Extra Space Storage

Project: 1761 Katella

Location: B-1 @ 15'

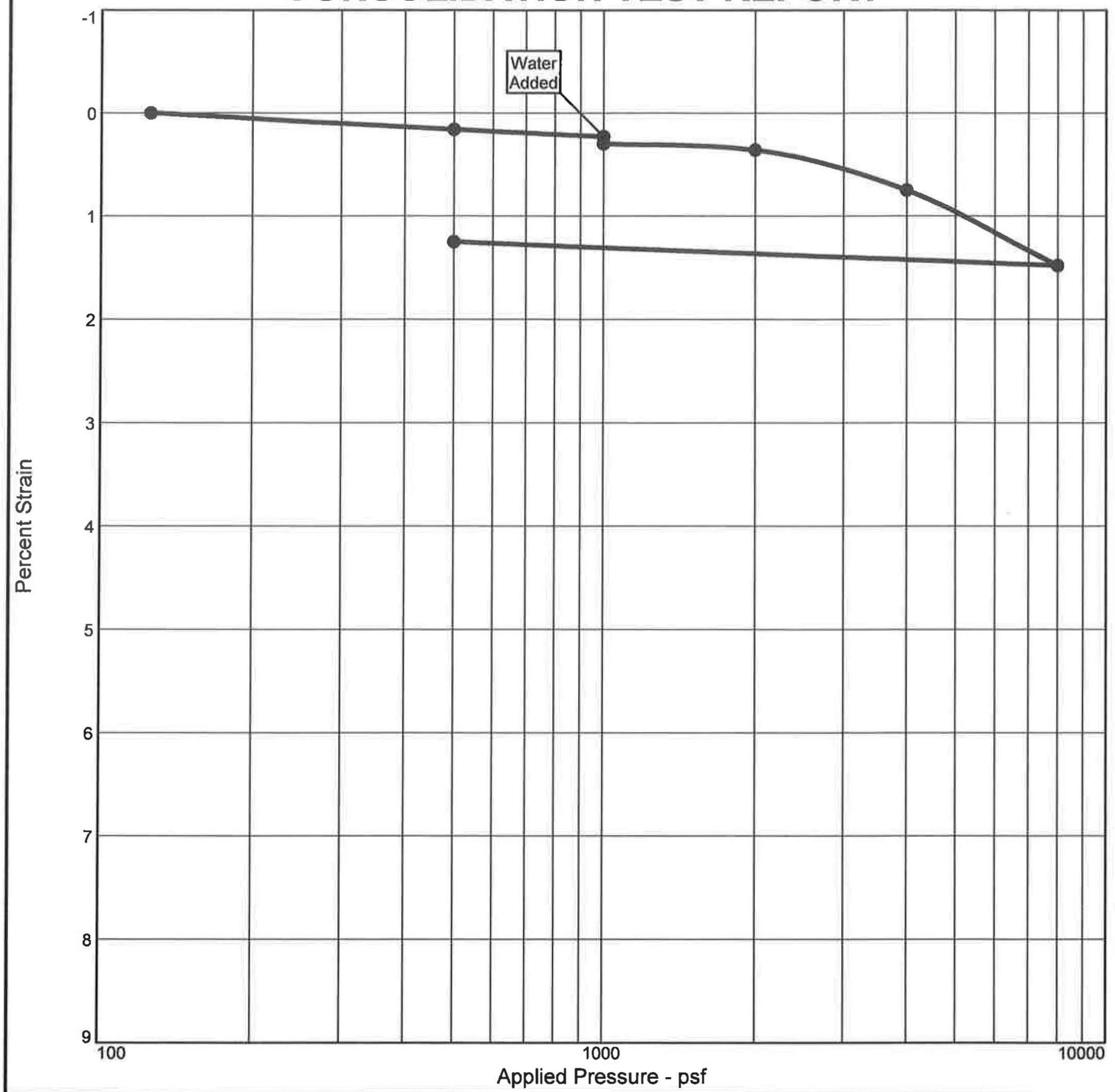
Gorian & Associates

Thousand Oaks, CA

Remarks:

Figure

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (psf)	P_c (psf)	C_c	C_r	Swell Press. (psf)	Clpse. %	e_o
Sat.	Moist.											
							2977				0.1	

MATERIAL DESCRIPTION	USCS	AASHTO

Project No. 3239-0-0- **Client:** Extra Space Storage

Project: 1761 Katella

Location: B-2 @ 25' **Sample Number:** B-2 @ 25'

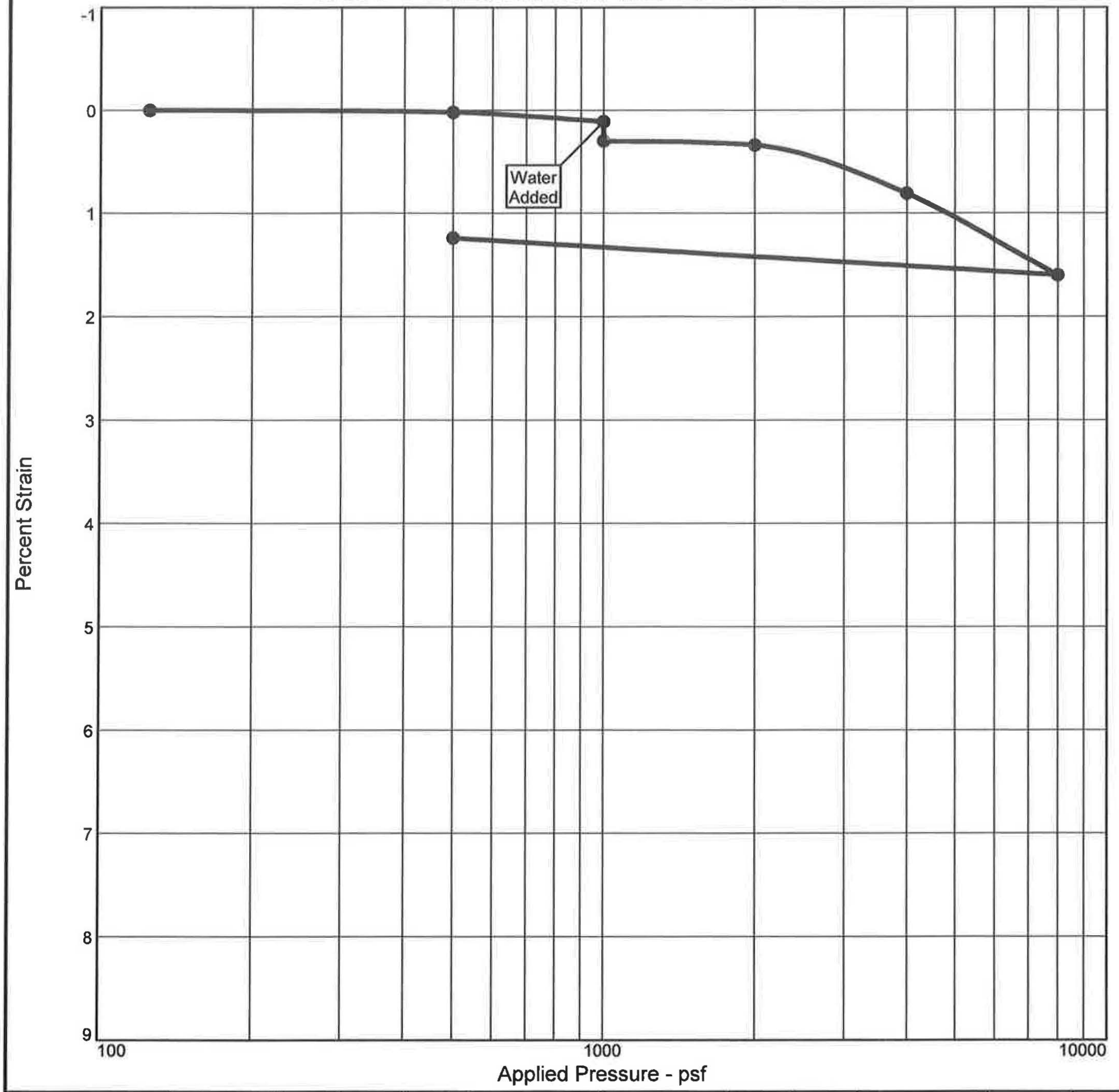
Gorian & Associates

Thousand Oaks, CA

Remarks:

Figure

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (psf)	P_c (psf)	C_c	C_r	Swell Press. (psf)	Clpse. %	e_o
Sat.	Moist.											
							2784				0.2	

MATERIAL DESCRIPTION	USCS	AASHTO

Project No. 3239-0-0- **Client:** Extra Space Storage

Project: 1761 Katella

Location: B-3 @ 30' **Sample Number:** B-3 @ 30'

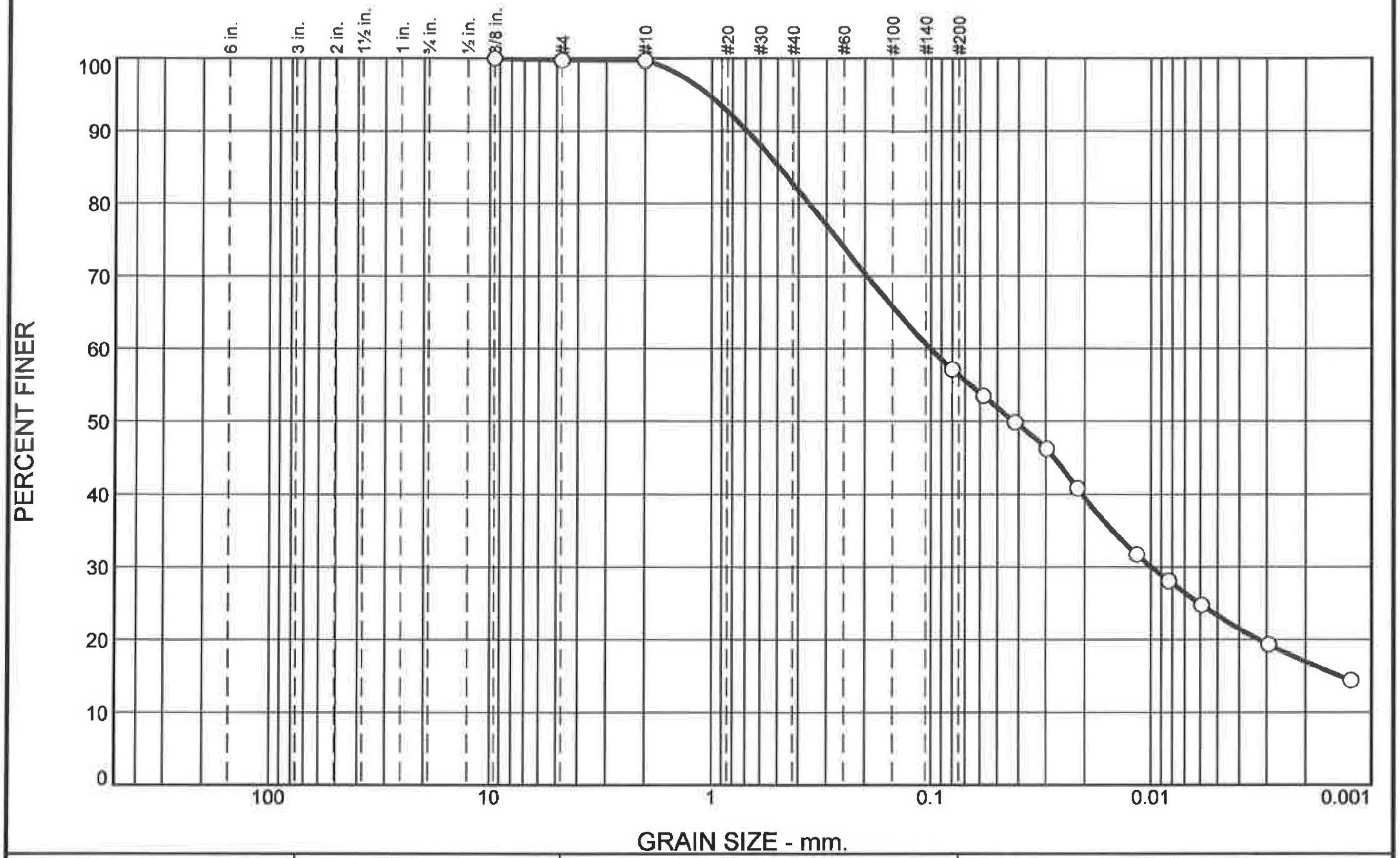
Gorian & Associates

Thousand Oaks, CA

Remarks:

Figure

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.2	0.1	16.9	26.4	33.1	23.3

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.375	100.0		
#4	99.8		
#10	99.7		
0.0800 mm.	57.1		
0.0576 mm.	53.5		
0.0414 mm.	49.9		
0.0298 mm.	46.2		
0.0216 mm.	40.8		
0.0116 mm.	31.7		
0.0083 mm.	28.1		
0.0059 mm.	24.8		
0.0029 mm.	19.3		
0.0012 mm.	14.4		

* (no specification provided)

Soil Description

Y.B Silty Clay

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.6890 D_{85}= 0.4895 D_{60}= 0.1005
D_{50}= 0.0420 D_{30}= 0.0100 D_{15}= 0.0014
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Sample Number: IB-2 **Depth:** 11'

Date: 3/24/23

Gorian & Associates

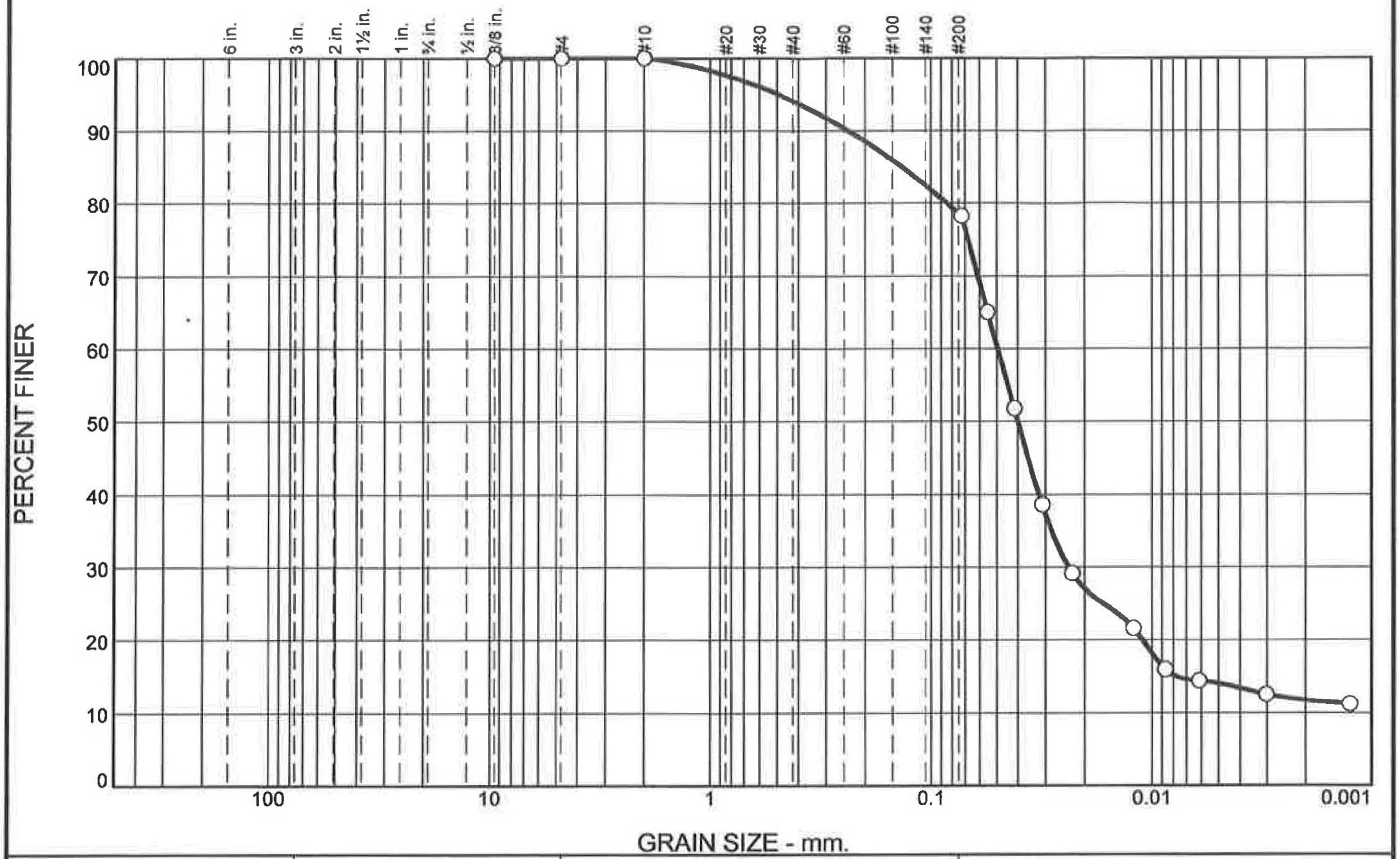
Thousand Oaks, CA

Client: Extra Space Storage
Project: 1761 Katella

Project No: 3239-0-0-100 **Figure**

Tested By: TT

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	5.9	15.5	64.6	14.0

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.375	100.0		
#4	100.0		
#10	100.0		
0.0724 mm.	78.2		
0.0550 mm.	65.0		
0.0414 mm.	51.8		
0.0310 mm.	38.6		
0.0227 mm.	29.2		
0.0121 mm.	21.6		
0.0087 mm.	16.0		
0.0061 mm.	14.4		
0.0030 mm.	12.5		
0.0013 mm.	11.2		

* (no specification provided)

Soil Description

Y.B Silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.2380 D_{85}= 0.1358 D_{60}= 0.0495
D_{50}= 0.0399 D_{30}= 0.0236 D_{15}= 0.0078
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Sample Number: IB-1 **Depth:** 18'

Date: 3/24/23

Gorian & Associates

Thousand Oaks, CA

Client: Extra Space Storage
Project: 1761 Katella

Project No: 3239-0-0-100 **Figure**

Tested By: TT

Soil Analysis Lab Results

Client: Gorian & Associates, Inc.
Job Name: Extra Space Storage - 1761 Katella
Client Job Number: 3239-0-0-100
Project X Job Number: S230324B
March 28, 2023

	Method	ASTM D4327		ASTM D4327		ASTM G187		ASTM G51	ASTM G200	SM 4500-D	ASTM D4327	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D4327	ASTM D4327
Bore# / Description	**Depth**	**Sulfates** SO_4^{2-}		**Chlorides** Cl^-		**Resistivity** As Rec'd	Minimum	**pH**	**Redox**	**Sulfide** S^{2-}	**Nitrate** NO_3^-	**Ammonium** NH_4^+	**Lithium** Li^+	**Sodium** Na^+	**Potassium** K^+	**Magnesium** Mg^{2+}	**Calcium** Ca^{2+}	**Fluoride** F_2^-	**Phosphate** PO_4^{3-}
	(ft)	(mg/kg)	(wt%)	(mg/kg)	(wt%)	(Ohm-cm)	(Ohm-cm)		(mV)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)
B - 1 Grayish brown silty clay	6.0	318.0	0.0318	39.1	0.0039	6,566	1,541	8.3	145	0.3	37.2	1.8	ND	276.3	5.3	17.8	175.5	18.2	1.1

Cations and Anions, except Sulfide and Bicarbonate, tested with Ion Chromatography
mg/kg = milligrams per kilogram (parts per million) of dry soil weight
ND = 0 = Not Detected | NT = Not Tested | Unk = Unknown
Chemical Analysis performed on 1:3 Soil-To-Water extract
PPM = mg/kg (soil) = mg/L (Liquid)

Note: Sometimes a bad sulfate hit is a contaminated spot. Typical fertilizers are Potassium chloride, ammonium sulfate or ammonium sulfate nitrate (ASN). So this is another reason why testing full corrosion series is good because we then have the data to see if those other ingredients are present meaning the soil sample is just fertilizer-contaminated soil. This can happen often when the soil samples collected are simply surface scoops which is why it's best to dig in a foot, throw away the top and test the deeper stuff. Dairy farms are also notorious for these items.

APPENDIX C

ASCE 7 HAZARDS REPORT

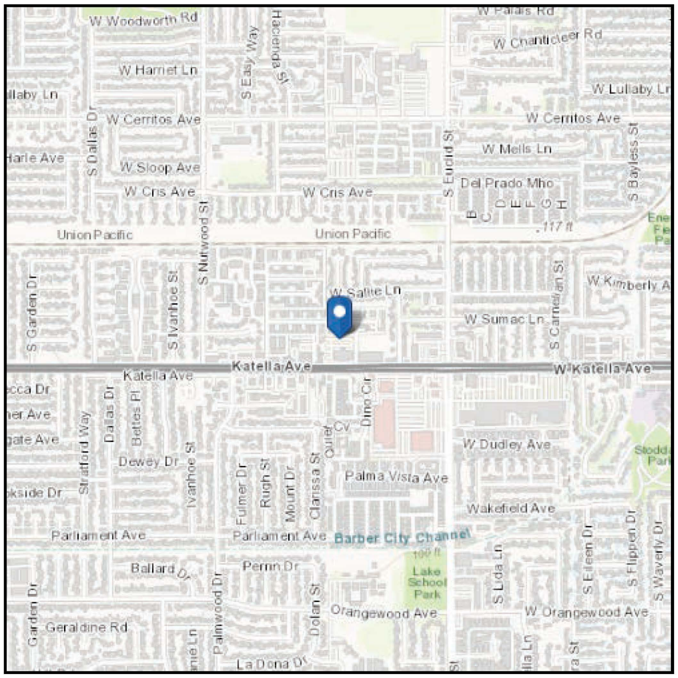
AMERICAN SOCIETY OF CIVIL ENGINEERS

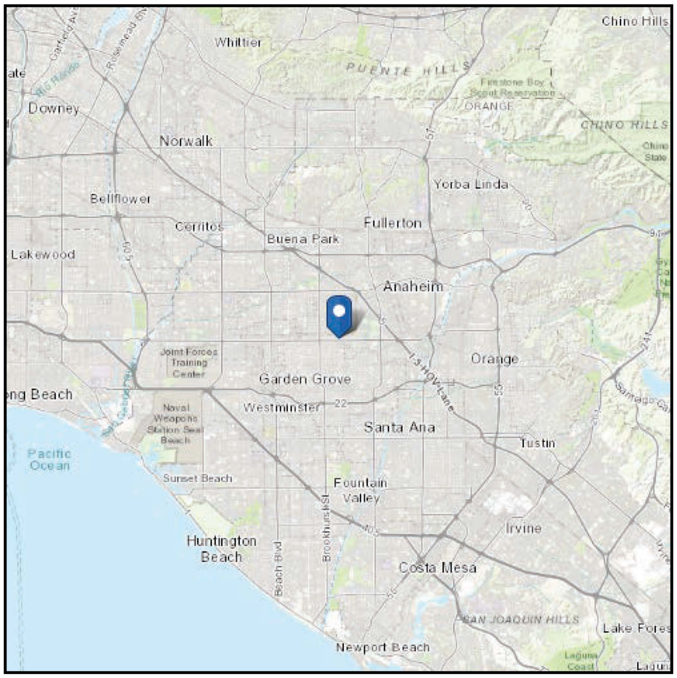
ASCE 7 Hazards Report

Address:
No Address at This Location

Standard: ASCE/SEI 7-16
Risk Category: II
Soil Class: D - Stiff Soil

Latitude: 33.804
Longitude: -117.9453
Elevation: 110.92126255210495 ft (NAVD 88)





Seismic

Site Soil Class: D - Stiff Soil

Results:

S_S :	1.416	S_{D1} :	N/A
S_1 :	0.5	T_L :	8
F_a :	1	PGA :	0.601
F_v :	N/A	PGA_M :	0.662
S_{MS} :	1.416	F_{PGA} :	1.1
S_{M1} :	N/A	I_e :	1
S_{DS} :	0.944	C_v :	1.383

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

Data Accessed: Tue Mar 21 2023

Date Source: USGS Seismic Design Maps

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.

APPENDIX D

STORMWATER INFILTRATION TESTING

Boring InfiltrationTesting Field Log

Work Order: 3239-0-0-100
Date 3/28/2023

Project Location 1761 West Katella Avenue
Earth Description
Tested By TT
Liquid Description Clear Water
Measurement Method Sounder
Depth to Invert of BMP

Start Time for Pre-Soak

Start Time for Test 9:50

Boring/Test Number IB-1
Diameter of Boring (DIA) 8 inches
radius (in feet) 0.3333333 feet
Depth of Boring 20 feet
Diameter of Casing 2 inches
Depth to Water Table >50 feet

Water Remaining in Boring (Y/N) N
Std Time Interval Btween Rdngs 30 minutes

Reading No.	Water Level start	Water Level end	Time start	time end	Δ Time (min)	H for surface area	h for volume	Surface Area (ft^2)	Volume (ft^3)	Raw Rate Volume / Surface Area (in/hr)
1	15	16.5	9:50	10:20	30	5	1.5	10.5	0.52	1.20
2	15	16.3	10:20	10:50	30	5	1.3	10.5	0.45	1.04
3	15	16.3	10:50	11:20	30	5	1.3	10.5	0.45	1.04
4	15	16.3	11:20	11:50	30	5	1.3	10.5	0.45	1.04
5	15	16.3	11:50	12:20	30	5	1.3	10.5	0.45	1.04
6	15	16.3	12:20	12:50	30	5	1.3	10.5	0.45	1.04
7	15	16.3	12:50	1:20	30	5	1.3	10.5	0.45	1.04
8	15	16.3	1:20	1:50	30	5	1.3	10.5	0.45	1.04

Measured Rate = ave of last 3 readings = 1.04

apply reduction factor: 2 **0.52** **in/hr**

Boring InfiltrationTesting Field Log

Work Order: 3239-0-0-100

Date 3/28/2023

Project Location 1761 West Katella Avenue

Earth Description

Tested By TT

Liquid Description Clear Water

Measurement Method Sounder

Depth to Invert of BMP

Start Time for Pre-Soak

Start Time for Test 9:37

Boring/Test Number IB-2

Diameter of Boring (DIA) 8 inches

radius (in feet) 0.3333333 feet

Depth of Boring 13 feet

Diameter of Casing 2 inches

Depth to Water Table >50 feet

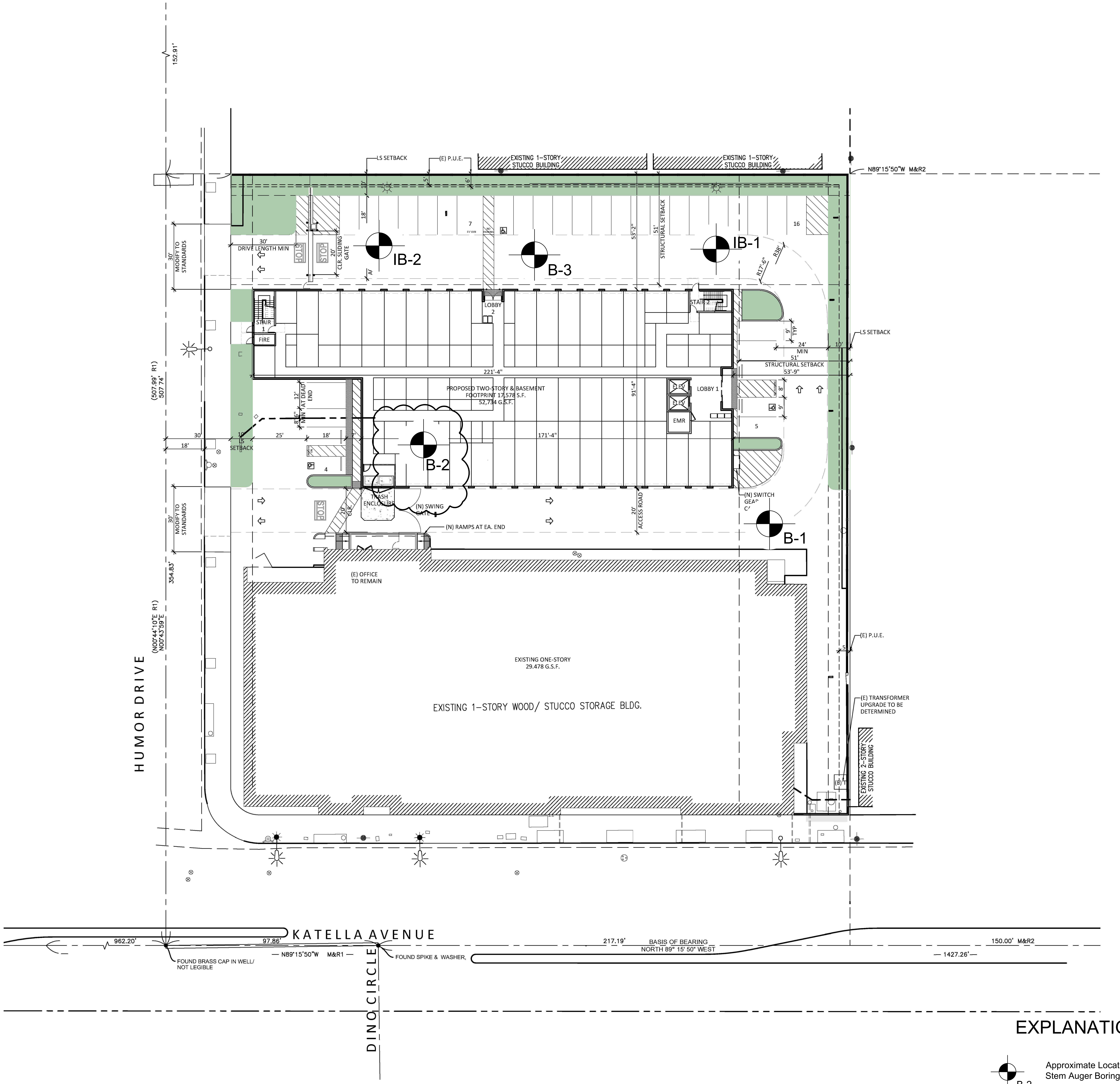
Water Remaining in Boring (Y/N) N

Std Time Interval Btween Rdngs 30 minutes

Reading No.	Water Level start	Water Level end	Time start	time end	Δ Time (min)	H for surface area	h for volume	Surface Area (ft^2)	Volume (ft^3)	Raw Rate Volume / Surface Area (in/hr)
1	8	10.1	9:37	10:07	30	5	2.1	10.5	0.73	1.68
2	8	10.2	10:07	10:37	30	5	2.2	10.5	0.77	1.76
3	8	10.3	1.:37	11:07	30	5	2.3	10.5	0.80	1.84
4	8	10.3	11:07	11:37	30	5	2.3	10.5	0.80	1.84
5	8	10.3	11:37	12:07	30	5	2.3	10.5	0.80	1.84
6	8	10.3	12:07	12:37	30	5	2.3	10.5	0.80	1.84
7	8	10.3	12:37	1:07	30	5	2.3	10.5	0.80	1.84
8	8	10.3	1:07	1:37	30	5	2.3	10.5	0.80	1.84

Measured Rate = ave of last 3 readings = 1.84

apply reduction factor: 2 **0.92** **in/hr**



PROJECT DESCRIPTION

REDEVELOPMENT OF SITE WITH EXISTING ONE-STORY 29,478 S.F. SELF-STORAGE BUILDING TO REMAIN AND PROPOSED CONSTRUCTION OF TWO-STORY WITH BASEMENT SELF-STORAGE BUILDING.

PROJECT INFORMATION

PROJECT ADDRESS:	1761 W. KATELLEA AVENUE, ANAHEIM, CA 92804	
SITE APN:	128-542-011	
SITE AREA GROSS:	84,010 S.F.	OR 1.93 ACRES
EXISTING ZONE:	C-G GENERAL COMMERCIAL	
PROPOSED ZONE:	NO CHANGE	
GENERAL PLAN:	RESIDENTIAL-LOW-MEDIUM	
EXISTING USE:	SELF-STORAGE W/ CUP GRANTED	
HISTORICAL LAND USE PERMITS:	CUP3779, SUBTTM3258, VAR1376, VAR2181	
USE (PROPOSED):	SELF-STORAGE CUP REQUIRED INCLUDES VARIANCES FOR PARKING AND F.A.R.	
LOT COVERAGE MAX ALLOWED:	NONE INDICATED	
LOT COVERAGE EXISTING:	40%	
LOT COVERAGE PROPOSED:	TBD	
F.A.R. ALLOWED:	0.50	
F.A.R. EXISTING:	0.77	62,239 S.F.
F.A.R. PROPOSED:	0.98	(82,212 S.F./ 84,010 S.F.)
SETBACKS REQUIRED:		
FRONT LANDSCAPE:	10' ABUTTING LOCAL STREET	
FRONT STRUCTURAL:	10' ABUTTING LOCAL STREET	
SIDE LANDSCAPE:	10' ABUTTING RESIDENTIAL & LOCAL STREET	
STRUCTURAL ALONG RESIDENTIAL ZONE:	51'	
REAR LANDSCAPE:	10' ABUTTING RESIDENTIAL	
REAR STRUCTURAL ALONG RESIDENTIAL :	51'	
SETBACKS PROVIDED:		
FRONT LANDSCAPE:	10'	
FRONT STRUCTURAL:	36'-5"	
SIDE LANDSCAPE:	10'	
STRUCTURAL ALONG RESIDENTIAL ZONE:	52'	
REAR LANDSCAPE:	10'	
REAR STRUCTURAL ALONG RESIDENTIAL :	53'-2"	
* **STRUCTURAL HEIGHT MAX::**	28' (2-STORIES)	
STRUCTURAL HEIGHT PROPOSED:	25'-4" (2-STORIES)	

LANDSCAPE REQUIREMENTS:

LANDSCAPING SHALL BE PERMITTED AND/OR REQUIRED IN INDUSTRIAL ZONES, SUBJECT TO CHAPTER 18.46.

FENCES, WALL, & HEDGES:

FENCES, WALLS, HEDGES, AND BERMS SHALL BE PERMITTED AND/ OR REQUIRED IN INDUSTRIAL ZONES, SUBJECT TO SECTION 18.46.110.

NOTES:

* 1. THE MAX STRUCTURAL HEIGHT IS BASED ON THE DISTANCE OF THE COMMERCIAL BUILDING FROM ANY SINGLE-FAMILY ZONE.

BUILDING AREA

TOTAL BUILDING AREA OF EXISTING BLDG(S):	32,757 S.F.	(2 EXISTING BUILDINGS)
EXISTING BLDG AREA TO BE DEMOLISHED:	-3,279 S.F.	
TOTAL:	29,478 S.F.	
PROPOSED BUILDING (G.S.F.):		
BASEMENT STORAGE:	17,578 S.F.	
1ST FLOOR STORAGE:	17,578 S.F.	
2ND FLOOR STORAGE:	17,578 S.F.	
TOTAL:	52,734 S.F.	
TOTAL GROSS SQUARE FOOTAGE:	82,212 S.F.	
NET INCREASE IN BUILDING FLOOR AREA:	52,734 S.F.	

CAR & BIKE PARKING ANALYSIS

PARKING PROVIDED FOR (E) SELF-STORAGE: PER SCTION 18.42	75 SP	
PARKING REQUIRED FOR NEW BUILDING: PER SECTION 18.42	REQUIRES PARKING DEMAND STUDY	
TOTAL PARK'G PROVIDED FOR SELF-STORAGE:	32 SP	
ACCESSIBLE PARKING REQUIRED:	1 SP	
ACCESSIBLE PARKING PROVIDED: (INCLUDED IN TOTAL PROVIDED)	3 SP	
EV / CAV SPACES REQUIRED:		
CLEAN AIR VEHICLES SPACES REQ'D:	1 SP	(5.106.5.2) TBD
EV CHARGING SPACES REQ'D:	0 SP	(5.106.5.3.3 AND TABLE 11B-228.3.2
EV CHARGING/ EV/CAV PROVIDED: (INCLUDED IN TOTAL PROVIDED)	0 SP	(INCLUDES --EVCS)**

* IF THE CALCULATION OF PARKING RESULTS IN THE REQUIREMENT FOR A FRACTION OF A PARKING SPACE, SUCH A PARKING SPACE NEED NOT BE PROVIDED UNLESS THE FRACTION EXCEEDS 50%

** FUTURE CHARGING SPACES QUALIFY AS DESIGNATED PARKING AS DESCRIBED IN 5.106.5.2 DESIGNATED PARKING FOR CLEAN AIR VEHICLES

BIKE PARKING REQUIRED: (FOR INDUSTRIAL)		
LONG TERM:	TBD	
SHORT TERM:	TBD	
	0 SP	TOTAL MINIMUM
BIKE PARKING PROVIDED:	TBD	LONG TERM
	TBD	SHORT TERM

Boring Location Map

EXPLANATION

B-2	Approximate Location of Hollow Stem Auger Boring
B-1	Approximate Location of Infiltration Test Boring
	Area of Encountered Pea Gravel Limits Unknown

Gorian & Associates, Inc.
Applied Earth Sciences

Job No: 3239-0-0-100		Date: March 2023
Scale: 1"=20'	Drawn by: Approved by:	PLATE 1

NORTH

0 5' 10' 20' 40'
scale: 1" = 20'-0"

ExtraSpace Storage®

1761 W. KATELLA AVENUE, ANAHEIM , CA 92804
APN 128-542-011

CONCEPTUAL SITE PLAN - STUDY
01.17.23

KSP
STUDIO
architecture + engineering
23 ORCHARD ROAD, SUITE 200
LAKE FOREST,CA 92630
T 949.380.3970 F 949.380.3771