Appendix G1 Geotechnical Evaluation Report

Appendices

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GEOTECHNICAL
PROFESSIONALS INC.

UPDATED GEOTECHNICAL EVALUATION REPORT FOR CEQA
PROPOSED LAGUNA NIGUEL TOWN CENTER
30102 PACIFIC ISLAND DRIVE
LAGUNA NIGUEL, CALIFORNIA

Prepared for:
LAGUNA NIGUEL TOWN CENTER PARTNERS
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Project No. 2952.I

October 11, 2019 (updated August 13, 2021)

October 11, 2019 (updated August 13, 2021)

Laguna Niguel Town Center Partners
1100 Newport Center Drive
Newport Beach, CA 92660

Attention: Mr. Christian Santos
Development Director

Subject: Updated Geotechnical Evaluation Report for CEQA
Proposed Laguna Niguel Town Center
30102 Pacific Island Drive (Crown Valley Parkway at Alicia Parkway)
Laguna Niguel, California
GPI Project No. 2952.I

Dear Christian:

Transmitted herewith is one electronic copy and three hard copies of our updated report of geotechnical investigation for the subject project. The report presents the results of our evaluation of the subsurface conditions at the site and recommendations for design and construction.

We appreciate the opportunity of offering our services on this project and look forward to seeing the project through its successful completion. Feel free to call us if you have questions regarding our report or need further assistance.

Very truly yours,
Geotechnical Professionals Inc.

Paul R. Schade, G.E. 2371
Principal

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TABLE OF CONTENTS

LIST OF TABLES AND FIGURES

1.0 INTRODUCTION

1.1 GENERAL

This updated report presents the results of a preliminary geotechnical and geologic/seismic hazards study performed by Geotechnical Professionals Inc. (GPI) for the proposed Laguna Niguel Town Center development in Laguna Niguel, California. The purpose of this study and report is to support due diligence efforts by providing geotechnical and geologic input for the Environmental Impact Report for the project. This includes addressing the geotechnical related issues listed in Appendix G, Environmental Checklist Form, of the State California Environmental Quality Act CEQA Guidelines (see Appendix D). The location of the site is shown on Figure 1, Site Location Map. The layout of the site with existing and proposed conditions is shown on Figures 2 and 3, Existing Site Plan and Proposed Site Plan, respectively. A Site Geologic Map is presented on Figure 4.

This study includes review of recent limited subsurface explorations, available subsurface and geologic information, limited engineering analyses, and a preliminary geotechnical field exploration and laboratory testing program. A comprehensive geotechnical investigation to support detailed design and construction should be conducted for final design.

1.2 PROJECT DESCRIPTION

It is our understanding that Laguna Niguel Town Center Partners (LNTCP) is planning to develop the subject property, which is currently owned by the County of Orange. The project site is approximately 23.5 acres and bounded by Crown Valley Parkway, Alicia Parkway, Pacific Island Drive, and residential developments.

In general, the proposed development of the site includes about 174,851 square feet of commercial and civic space (retail, office, library), 275 apartment units, a parking structure, and outdoor plaza. The buildings will consist of commercial and civic buildings (one, two, and three stories above grade), a parking garage (three to four levels, two to three stories partially subterranean), and two residential buildings, one with three to four story apartments with first floor tuckunder parking and one with a three- and four-story apartments wrapping a free standing, five-story parking garage. Preliminary structural loads are anticipated to be up to about 600 kips for the five-story parking structure, 375 kips for the apartment buildings, and up to 250 kips for the office/retail/library buildings. To achieve the final site grades, the existing grades will be predominantly cut, with only localized areas of fill.

2.0 SCOPE OF WORK

Our overall scope of work for the development included a review of readily available subsurface and geologic data from previous geotechnical reports by others, a limited field exploration program, limited laboratory testing, geologic and seismic hazard evaluation, preliminary foundation analyses, and preparation of this Geotechnical Evaluation Report for CEQA. This report presents the results of our study to address potential geotechnical and geologic/seismic hazards for the development as outline in Appendix G of the CEQA Guidelines previously described.

A future comprehensive Geotechnical Investigation Report will be prepared for the project to provide more detailed analysis and geotechnical recommendations for design and construction.

Our preliminary field exploration program for the overall development consisted of five Cone Penetration Tests (CPTs) and four exploratory borings. The CPTs were generally advanced to depths of approximately 30 to 50 feet below existing grades. The borings were drilled to depths of approximately 30 to 50 feet below existing grades. The locations of the subsurface explorations for the development are shown on Figures 2 through 4. Details and the results from the preliminary field exploration and laboratory testing programs are presented in Appendices A to C.

Our scope of work included review of in-house geotechnical reports and those made available to us, on-line open file geologic hazards reports, geology maps, vintage stereoscopic aerial photographs, and groundwater data. The data recently acquired from our limited exploration and laboratory testing were also reviewed and engineering analyses were performed to assess potential foundation systems and other geotechnical related constraints for the proposed buildings at the subject site. The results of our data review and conclusions regarding mitigation of potential geologic/seismic hazards are presented in this report.

3.0 SITE AND SUBSURFACE CONDITIONS

3.1 EXISTING SITE CONDITIONS

The site conditions at the time of our investigation are shown on Figure 2. Currently, about half the site is undeveloped and the remaining portions contain buildings that will be demolished or relocated. These buildings include a former fire station, a former County courthouse and District Attorney building, and a County vehicle maintenance facility. A City library along Crown Valley Parkway will be demolished and rebuilt as part of the project. An existing fire station and the Laguna Niguel City Hall are adjacent to the project site, not a part of the subject project, and will remain.

The ground surface elevation across the site varies from about Elevation 305 to 370 feet. A 40- to 50-foot-high ascending slope extends along the western and southwestern property lines, with apartments and homes in-place at the top of the slope. Based on a geotechnical investigation report by others (DHLA, 2009), the slope below the homes (southern portion of the west property limits) was evaluated in 2009 for on-going lateral deflection and creep.

3.2 SITE HISTORY

Our understanding of the development history of the site is based on information provided by our client, a review of historical aerial photographs (Historical Aerials and Google Earth), vintage stereoscopic aerial photographs, and additional online sources. A brief site history is presented below. In general, the site has been in its approximate current configuration since 1994.

The earliest aerial photograph reviewed for this study was taken in 1938. At that time, the site was undeveloped and included a tributary canyon on the west side of the property that drained into Sulphur Creek from the northwest. Sulphur Creek was the primary canyon to the south-southeast of the site near the current alignment of Crown Valley Parkway.

Between 1938 and 1963, the site appears to be relatively unchanged with some minor unpaved roads being developed. Vintage USDA stereoscopic air photographs dated December of 1952 similarly show the site area in natural condition. In 1967, a paved roadway along the approximate alignment of Crown Valley Parkway along the southern property limit appears.

In stereoscopic air photographs taken in 1970, Crown Valley Parkway and Alicia Parkway were constructed and the east portion of the site was graded for construction of the courthouse and adjacent district attorney building, with a surface parking lot in-place near the corner of Crown Valley Parkway and Alicia Parkway (newly constructed) where the current City Hall development is located. Vintage stereoscopic aerial photographs dated 1970 through 1978 show that the west side of the site was not graded, including the tributary canyon from the northwest, which remained as a natural canyon area.

By 1980, the tributary canyon had been filled in, the remainder of the site outside of the development shown in the 1970 aerial had been cleared, and the housing development to the southwest was in place. In 1981, one of the buildings currently occupied by the County fleet vehicle service facility had been constructed, the ascending slope to the west was constructed, and Pacific Island Drive was in-place. By 1994, the library, fire station, and apartments to the west were built and the parking around the courthouse building had been extended to the north.

3.3 SUBSURFACE SOILS

Our preliminary limited field investigation and prior explorations by others disclosed a subsurface profile consisting of shallow to relatively deep fill soils overlying natural materials. Detailed descriptions of the conditions encountered in our subsurface explorations are presented in the logs of CPT's and borings in Appendices A and B, respectively.

Based on a review of the data from previous geotechnical investigations, as well as our recent limited subsurface investigation, near-surface fill soil conditions at the site can generally be characterized as follows:

- Eastern Portion of the Site: The eastern portion of the site is generally a cut into bedrock of the Capistrano Formation (Tc) with relatively shallow fills overlying the bedrock. We encountered fill soils to a depth of about 2 feet in our recent Boring B-2. Prior borings by others (Group Delta, 2005) performed in this area encountered fill soils to depths of about 0 to 5 feet below the ground surface. The fill soils predominantly consist of clay and appear to be derived from the Capistrano Formation bedrock.

- Western Portion of the Site: The western portion of the site comprises the area where a former tributary canyon, approximately 30 to 40 feet deep, was filled to current site grades with clay, sandy clay, and silty clay fill soils generated from on-site cuts in the Capistrano Formation (Tc) and possibly other sources. Within the former tributary canyon area, we encountered fills to depths of 20 to 29 feet below the ground surface in our recent explorations. Fills up to 34 feet were encountered in prior explorations by others (Group Delta, 2005). Although not encountered in our limited field exploration program, we anticipate deeper fills are present within the former canyon area. In general, the fills were moist to wet. Based on laboratory test results, the clays have a very high Expansion Index (EI) resulting in a high potential to shrink and swell with changes in moisture content. We were unable to obtain documentation on the placement and compaction of the fill in the former canyon area.

The natural materials encountered throughout the site consist predominantly of Capistrano Formation (Tc) siltstone and claystone bedrock. Within the former canyon area (western portion of the site), it appears that a thin layer of colluvial or alluvial soils are in-place between the fill and underlying bedrock. The thin layer of colluvial/alluvial soils were not encountered outside of former canyon area. The bedrock is predominantly very stiff to hard (soil rather than rock consistency terminology) and very moist to wet. The natural materials exhibited moderate to high strength and low to moderate compressibility characteristics. Although not tested, the upper natural materials are anticipated to have a high to very high potential for expansion (the fill soils that were tested as having a very high expansion potential appear to be derived from grading of the on-site bedrock).

4.0 REGULATORY FRAMEWORK

4.1 GENERAL

This section provides an introduction to applicable state and local laws, regulations, and codes that will govern the project.

4.2 STATE LEVEL

The State of California adopted the 2019 California Building Code (CBC), Volumes 1 and 2 on July 1, 2019, effective on January 1, 2020. The 2019 CBC makes up Part 2 of Title 24 of the California Code of Regulations. Chapter 16 of Volume 2 contains provisions for structural design. Provisions for soils and foundation studies and design are presented in Chapter 18. Appendix J of the code applies to grading.

The Alquist-Priolo Geologic Hazard Zones Act was passed by the State of California in 1972 to address the hazard and damage caused by surface fault rupture during an earthquake. The Act was renamed the Alquist-Priolo Earthquake Fault Zoning Act in January 1994. The Act has been updated since then and requires the State Geologist to establish "earthquake fault zones" along known active faults in the state. Wherever an active fault exists, if it has the potential for surface rupture, a structure for human occupancy cannot be placed over the fault and must be a minimum distance from the fault. Cities and counties that contain earthquake fault zones are required to regulate development projects within these zones.

The California Seismic Safety Commission was established by the Seismic Safety Act in 1975 with the intent of providing oversight, review, and recommendations to the Governor, State Legislature, as well as state and local governments regarding seismic issues. The commission was renamed the Alfred E. Alquist Seismic Safety Commission in 2006.

The Seismic Hazard Mapping Act of 1990 was enacted, in part, to address seismic hazards not included in the Alquist-Priolo Act, including strong ground shaking, liquefaction, landslides, and or other seismic related ground failures. Under this Act, the State Geologist is assigned the responsibility of identifying and mapping seismic hazard zones. The recommended guidelines and criteria for the preparation of seismic hazard zones are presented in Special Publication 118, Recommended Criteria for Delineating Seismic Hazard Zones in California. The California Geological Survey (CGS), formerly the State of California, Division of Mines and Geology (CDMG), adopted seismic design provisions in Special Publication 117A, Guidelines for Evaluating and Mitigating Seismic Hazards in California (revised and readopted on September 11, 2008).

4.3 CITY LEVEL

The City of Laguna Niguel adopted the 2019 CBC, which became effective January 1, 2020. The CBC includes provisions to address issues related to site grading, cut and fill slope design, soil expansion, geotechnical studies before and during construction, slope stability, allowable bearing pressures and settlement below footings, effects of adjacent slopes on foundations, retaining walls, basement walls, shoring of adjacent properties, and potential primary and secondary seismic effects.

5.0 GEOLOGIC CONDITIONS

5.1 REGIONAL GEOLOGY

The project site is located in one of 11 geomorphic provinces in California termed the Peninsular Ranges Geomorphic Province. This province is characterized by northwest trending mountain ranges and active faults. The faults are related and of a similar trend to the San Andreas fault, a major active fault that marks the plate boundary between the North American Plate to the east and the Pacific Plate to the west. Several active faults are located in relatively close proximity to the site, most notably the Newport-Inglewood, located approximately 4.3 kilometers from the site.

The project is not in an Alquist-Priolo (AP) Earthquake Fault Zone so the potential for surface fault rupture is very low. Potential fault or earthquake related hazards at this site are due to the secondary effects of earthquakes, generally due to the hazards caused by strong ground motions induced by earthquakes on faults at distance from the site.

The San Joaquin blind thrust fault, approximately 5.6 kilometers from the site, is a recently discovered fault that is buried several kilometers below the ground surface. A blind thrust fault is a deeply buried, shallow dipping thrust fault that does not project to the ground surface. Blind thrusts are capable of generating a major earthquake that may cause uplift, but because they do not intersect the ground surface, primary surface fault rupture is considered unlikely as a potential hazard. The San Joaquin fault has an estimated potential earthquake magnitude of 7.3 (Grant, L.B., et.al., 1999), that could produce ground motions estimated at 0.6g (Silverman, D., 2012). A recent earthquake in the Orange County coastal area with an epicenter of approximately 2.5 kilometers south of the subject site and a magnitude of 3.9 has been attributed to this fault. Based on these studies, an earthquake on this fault would not produce a ground rupture hazard but would induce ground motions similar to that of the Newport-Inglewood fault.

More locally, the site is within the San Joaquin Hills, consisting of moderate to steep relief hillside terrain underlain by sedimentary bedrock. The San Joaquin Hills are dissected by streams and drainage divides sloping south and southwest towards the coastline. The drainages are typically partially filled by poorly consolidated colluvial and alluvial deposits overlying the deeper formational bedrock materials. The bedrock in the area is characterized as being tilted, with bedding inclinations generally dipping to the southeast at shallow to moderate angles.

5.2 SITE GEOLOGY

Prior to grading, the site consisted of low relief hillsides sloping southeasterly to Sulphur Creek, a southerly flowing drainage that now contains Crown Valley Parkway. Much of the eastern portion of the site area adjacent to Alicia Parkway consisted of a low relief, southeasterly trending ridgeline underlain by siltstone bedrock of the Miocene age Capistrano Formation. The western portion of the site consisted largely of a southeasterly trending tributary canyon to Sulphur Creek. The tributary has been filled by previous grading with fill deposits on the order of 30 to 40 feet thick, overlying the Capistrano Formation at depth. Prior to placement of fill in the canyon, it was reported to contain surficial deposits of colluvium, alluvium, and possibly terrace deposits. Documentation of the fill placement in the canyon is not readily available, and it is not clear whether the surficial deposits, which are potentially compressible, were entirely removed prior to placement of the canyon fill.

As encountered in our explorations to depths of 50 feet, and as characterized by others in prior investigations at the site, the subsurface profile primarily consists of fills overlying formational bedrock materials, with localized deposits of alluvial soils encountered between the fill soils and bedrock. The fills consisted primarily of medium to high plasticity clays with trace amounts of sand and gravel. Fill depths extended up to 29 feet below existing site grades in our recent explorations, with the deepest fills encountered within the western portions of the site, in the vicinity of the filled tributary drainage. Within the eastern portions of the site, the clay fills extended to depths of generally less than 5 feet (based on our review of prior explorations performed by others).

The underlying natural materials consisted of bedrock of the Capistrano Formation (Tc), comprised of siltstone and claystone. The Capistrano Formation is a marine deposit that is poor to moderately consolidated and prone to slope instability. Subsurface testing of the bedrock materials indicates they are generally stiff to very stiff, increasing in density with depth, when using soil consistency terminology. The geologic conditions in the site area are shown on Figure 5, Regional Geologic Map. Detailed logs of the subsurface conditions encountered in our explorations will be presented in our Preliminary Geotechnical Investigation Report.

5.3 GROUNDWATER

Data published by the State of California indicates a shallowest depth to groundwater of 5 to 20 feet at the subject site (CGS, 2001). The available data is isolated to former drainages (20 feet in the western and southwestern portions of the site; 5 feet along the southeastern portions of the site). Data doesn’t appear to be available outside of these drainages.

Groundwater was encountered at depths of approximately 14 to 24.5 feet below existing site grades in three of our borings, corresponding to approximate elevations of +329 to +336.5 feet. Groundwater was not encountered in our southernmost boring (B-4) within the 30-foot depth explored (tip elevation of +305 feet). Details of the groundwater depths in the vicinity of the site are shown on the Historical High Groundwater Map, Figure 6.

5.4 TECTONIC SETTING

5.4.1 Regional Fault Systems

The geologic structure of southern California is dominated by northwest trending faults associated with the San Andreas Fault System. Faults such as the Newport-Inglewood, Whittier, Palos Verdes Hills and San Jacinto are considered active and are associated with the San Andreas, collectively forming the boundary between the North American and Pacific tectonic plates. Most of these faults have ruptured the ground surface historically and/or produced significant earthquakes.

Anomalous to the general northwest structural fabric are a series of active east-west trending reverse or thrust faults. The majority of these occur as north dipping planes projecting along the southern base of the Santa Monica and San Gabriel Mountains in the greater Los Angeles area. The known active thrust faults in the region include the Cucamonga, Sierra Madre, San Fernando, Raymond, Santa Monica and Hollywood faults.

5.4.2 Concealed Faults

Another category of fault known as "blind thrusts" was recognized as a significant seismic hazard following the 1987 magnitude 6.0 Whittier Narrows Earthquake and then again by the 1994 San Fernando magnitude 6.7 Earthquake. A blind thrust is a deeply buried, shallow dipping thrust fault, which does not project to the ground surface. Blind thrusts are capable of generating a major earthquake that may cause uplift in the form of anticlinal hills.

At the present time, the potential magnitudes and recurrence intervals of blind thrust produced earthquakes cannot be quantified with confidence due to the fact that many characteristics of these features (including areal extent and Quaternary slip rates) are poorly understood. Nonetheless, the proximity to densely populated urban centers and their history of producing damaging earthquakes clearly demonstrate the risk that blind thrusts pose to large metropolitan areas and surrounding cities.

5.4.3 Nearby Seismogenic Sources

We reviewed the 2008 National Seismic Hazard Maps Source Parameters (USGS, 2014) to identify known active faults within a 100-mile radius of the project site. The names and distances of the faults lying with 25 miles of the project site are provided in the following table (Table 5.4.3-1). We present a map showing the significant regional faults in Figure 7, Regional Fault Map.

Table 5.4.3-1 – Significant Regional Faults

Fault Name	Approximate Distance* (km)
Newport Inglewood	4.3
San Joaquin Hills	5.6
Palos Verdes	19.3
Coronado Bank	20.4
Elsinore	21.1
Chino	22.5
Puente Hills (Coyote Hills)	26.7
Rose Canyon	32.4
Puente Hills (Santa Fe Springs)	33.0
San Jose	36.7
Puente Hills (Los Angeles)	38.7

* Defined as the closest distance to projection of rupture area along fault trace.

The site does not lie within an Alquist-Priolo Earthquake Fault Zone as designated by the California Geological Survey (CGS, 2001). In addition, named surface faults are not mapped projecting towards or through the site.

Brief details for some of the faults closest to the subject site are as follows:

<u>Newport-Inglewood Fault</u>

The Newport-Inglewood Fault is near the southwesterly side of the Los Angeles Basin, is a strike-slip fault, and is defined by a series of low disconnected hills and mesa surfaces. Strike slip faulting is associated with anticlinal folding, which can result in convex upward folds that accumulate petroleum in areas already host to oil reservoirs (e.g. Signal Hill, Dominguez Hills, and Baldwin Hills). The subject site is not in a geologic setting associated with these anticlinal folds. In 1933 the destructive Long Beach Earthquake occurred on the fault just offshore of Newport Beach. The event caused considerable damage and a high loss of life. Since then the various strands of the fault have produced many minor earthquakes that have been at a magnitude of 4.5 or less. The fault lies at a distance of approximately 6.8 kilometers to the east of the project site at its closest approach. A maximum earthquake magnitude of 6.9 and slip rate of 1.0 mm/yr has been assigned to the fault.

San Joaquin Blind Thrust

The Puente Hills Blind Thrust Fault does not reach the earth's surface, and no hazard of surface fault rupture is known to exist; however, movement on blind thrust faults can produce major earthquakes and resulting ground motions that may result in significant damage to structures. Based on the current lack of knowledge regarding the recurrence intervals and potential magnitudes of the earthquakes that could be produced by the San Joaquin Hills Blind Thrust Fault, as well as the overall extent and slip rates, the potential seismic risk posed by the fault is difficult to quantify. Based on the known earthquakes that have occurred on similar faults in the Los Angeles Basin, the San Joaquin Hills Blind Thrust Fault should be considered a similar seismic source as the Newport-Inglewood, without the potential for surface ground rupture.

Palos Verdes Fault

The Palos Verdes Fault is usually described as three individual segments, namely the San Pedro Bay, the onshore, and the Santa Monica Bay segments. All segments possess a reverse right oblique sense of displacement, trend to the northwest and dip to the southwest. There is evidence of Holocene fault movement on the San Pedro segment based on geophysical data. No confirming evidence has been established to indicate the other two segments have moved in the last 10,000 years. The fault lies approximately 8.8 kilometers south of the site at its closest approach. Seismicity associated with the fault is relatively low and most events recorded are micro-earthquakes. The fault has been assigned a 7.0 magnitude and a slip rate of 3.0 mm/yr (DMG, 1998).

5.4.4 Seismic Exposure

As is the case with most locations in Southern California, the subject site is located in a region that is characterized by moderate to high seismic activity. The project site and vicinity has historically experienced strong ground shaking due to earthquakes. The locations of earthquake epicenters with respect to the subject site are shown graphically on Figure 8, Regional Seismicity.

6.0 GEOLOGIC-SEISMIC HAZARDS

6.1 GENERAL

A summary of the requirements of Section VII. Geology and Soils of CEQA Appendix G: Environmental Checklist are presented below and followed by the results of our geologic and seismic hazards evaluation for the proposed development.

6.2 THRESHOLDS OF SIGNIFICANCE

In accordance with guidance provided in Section VII Geology and Soils of Appendix G of the State CEQA Guidelines, the project could have a potentially significant impact if it were to:

(a) Directly or indirectly cause potential substantial adverse effects, including the risk of loss, injury, or death involving:

  i. Rupture of a known earthquake fault, as delineated on the most recent Alquist-Priolo Earthquake Fault Zoning Map issued by the State Geologist for the area or based on other substantial evidence of a known fault.

  ii. Strong seismic ground-shaking.

  iii. Seismic-related ground failure, including liquefaction.

  iv. Landslides.

(b) Result in substantial soil erosion or the loss of topsoil.

(c) Be located on a geologic unit or soil that is unstable, or that would become unstable as a result of the project, and potentially result in on- or off-site landslide, lateral spreading, subsidence, liquefaction or collapse;

(d) Be located on expansive soil, as identified in Table 18-1-B of the Uniform Building Code (1994), creating substantial direct or indirect risks to life or property.

(e) Have soils incapable of adequately supporting the use of septic tanks or alternative wastewater disposal systems where sewers are not available for the disposal of wastewater.

(f) Directly or indirectly destroy a unique paleontological resource or site or unique geological feature.

The attachment to this report provides input for Section VII Geology and Soils of the CEQA Appendix G Environmental Checklist Form based on our evaluation of potential geologic and seismic hazards discussed herein. Note that septic will not be used on-site for the proposed project, and GPI's evaluation did not include assessment of paleontological resources.

6.3 SURFACE FAULT RUPTURE

The site does not lie within an Alquist-Priolo (AP) Earthquake Fault Zone as designated by the California Geological Survey (CGS, 2001) Surface faults have not been mapped projecting towards or through the site area. As such, shallow ground rupture is considered unlikely at this site.

6.4 SEISMIC GROUND SHAKING

As is the case with most locations in Southern California, the subject site is located in a seismically active area of southern California. The type and magnitude of seismic hazards that may affect the site are dependent on both the distance to causative faults and the intensity and duration of the seismic event. The subject site will likely experience strong ground shaking caused by earthquakes on active, regional faults in the future.

6.5 LIQUEFACTION AND SECONDARY EFFECTS

Loosely compacted/deposited granular soils located below the water table can fail through the process of liquefaction during strong earthquake-induced ground shaking. In this process, there is a rapid decrease in shearing resistance of cohesionless soils, caused by a temporary increase in the pore water pressure. Factors known to influence liquefaction potential include soil type and depth, grain size, relative density, ground-water level, degree of saturation, and both intensity and duration of ground shaking.

As a result of liquefaction, a typical building structure may be exposed to several hazards, including liquefaction-induced settlement, foundation bearing failure, and lateral displacement or lateral spreading. The surface manifestation of liquefaction in deeper soil deposits often takes place in the form of sand boils and ground subsidence. Such phenomena often lead to loss of adequate support for building foundations (bearing failures) and cause tilting, excessive movement, and cracking of superstructures. The severity of ground subsidence depends largely on the relative thickness of the surficial non-liquefiable layer compared to the thickness of layers undergoing liquefaction.

According to the published State Seismic Hazard Zones map for the San Juan Capistrano Quadrangle, the site is not located in an area designated by the State Geologist as a "zone of required investigation" due to the potential for earthquake-induced liquefaction. In addition, the soils underlying the proposed site are primarily high plasticity, cohesive fills and bedrock materials. As such, the potential for damage due to liquefaction, seismic-induced lateral spreading, and seismically induced settlement is low. Details of the liquefaction zones in the vicinity of the site are shown on Figure 9, Seismic Hazard Zones Map.

6.6 LANDSLIDES

The existing slopes ascending from the western and southwestern property boundaries are about 35 to 65 feet in height and mapped within an area designated by the State Geologist as a "zone of required investigation" due to the potential for earthquake-induced landsliding (CGS, 2001). Based on a review of the available topographic information, the slopes are generally inclined at 2:1 (horizontal:vertical) with localized portions as steep as nearly 1.5:1. A review of referenced documents indicates that the slopes were initially graded to buttress against slope instability, but that slope movement has been occurring since the initial construction of the residential developments above the slope. According to a geotechnical report evaluating the slopes (DHLA, 2009), the slope deformation is characteristic of lateral fill extension and slope creep, causing on-going distress to the structures supported at the top of the slope. There is no indication that the slope deformation to date has impacted the subject site at the toe of the slopes.

Because the majority of the western and southwestern slopes are off-site, mitigation measures for the potential slope instability will include construction of retaining structures to support the slopes where they extend onto the site and establishing adequate offsets between the base of the slopes and the proposed site structures. At this time, we anticipate retaining structures along the property line within the slope will consist of a soldier pile or equivalent retaining wall designed to resist lateral static and seismic earth pressures imposed by the adjacent slope. The design level geotechnical investigation will evaluate the suitability of a soldier pile retaining wall, providing geotechnical design parameters or recommendations for an equally or more effective mitigation measure.

In addition to constructing retaining walls as discussed above, current requirements of the California Building Code require a minimum lateral offset between the toe of a descending slope and the face of buildings at the base of the slope to be the smaller of 15 feet or one-half the height of the slope. The design level geotechnical investigation will evaluate the stability of the on-site and adjacent slopes, confirm the suitability of the offset, or provide an equally or more effective mitigation measure.

Based on a review of the preliminary grading plans, significant new permanent cut or fill slopes are not planned.

The potential for seismic-related ground failure due to landsliding for the project is considered less than likely with mitigation measures outlined herein and confirmed in the design level geotechnical investigation report.

6.7 TSUNAMIS AND SEICHES

Various types of seismically induced flooding, which may be considered as potential hazards to a particular site, include flooding due to a tsunami (seismic sea wave), a seiche, or failure of a major water retention structure upstream of the project. The site is located approximately 2 miles inland from the Pacific Ocean at elevations of approximately +330 feet to +360 feet above mean sea level. Due to the distance to the coast and elevation at the site, the probability of flooding due to a tsunami is considered to be nonexistent.

In addition, there are no dams or reservoirs located upstream at the site that may be susceptible to seiche. The Sulphur Creek Reservoir is located approximately 1.5 miles northeast of the subject site; however, the reservoir lies at an elevation of approximately +190 feet, over 100 feet below the predominant site grades. As such, the probability of site flooding due to seiche is also considered to be nonexistent.

6.8 EXPANSIVE AND COLLAPSE POTENTIAL

Expansive soils generally consist of clays that can shrink and swell with changes in moisture content. Movement of soils in response to shrinkage and swelling has the potential to impact near-surface improvements such as lightly loaded foundations, floor slabs, and flatwork. Based the data reviewed, near surface soils are anticipated to have high to very high expansion potential. Therefore, the potential for expansive soils to adversely affect the project if not mitigated is considered to be high. The project design should implement appropriate controls to minimize the impact of expansive soils on the proposed project, which will be provided in the design-level geotechnical report. Mitigation measures to reduce the adverse impact of expansive soils may include:

- In-place chemical treatment of the expansive soils (cement or lime treatment, or equivalent)

- Removal and replacement of the expansive soils with non-expansive import soils where the potential for shrink/swell is not tolerable

- A structural control method that could be utilized would include design of foundations, floor slabs, and hardscape to resist the potential swell pressures of the expansive soils by increasing concrete reinforcing or using post-tension methods as outlined in the California Building Code.

Collapsible soils generally consist of relatively dry, low-density materials that become weaker and more compressible with the addition of water or excessive loading. Due to the cohesive and very stiff to hard nature of the onsite soils, the potential for collapse of soils at this site to impact the project is considered very low.

6.9 SUBSIDENCE AND SETTLEMENT

The project site is not within an area of known subsidence associated with fluid withdrawal (groundwater or petroleum), peat oxidation (natural decay of organic peat materials), or hydrocompaction (compression of soils due to the introduction of water). Therefore, the potential for subsidence and associated settlement is considered to be low.

6.10 FLOODING AND INUNDATION

According to a flood map (Map Number 06059C0439J, dated December 3, 2009) prepared by the Federal Emergency Management Agency (FEMA), the project site is not located within a mapped flood zone (msc.fema.gov). Based on this information, the potential for flooding to negatively impact the project is considered to be very low.

6.11 SEDIMENTATION AND EROSION

The majority of the ground surface at the site is relatively level and is, or will be, covered with asphalt or concrete pavements. As such, erosion is not considered a hazard at the site. During construction, provisions should be in place to mitigate potential temporary erosion and sedimentation conditions. These provisions will be provided in the design-level geotechnical report and incorporated into the project civil and landscaping plans.

6.12 CORROSIVE SOILS

Limited on-site corrosivity laboratory test data (Group Delta, 2005) suggests that the site soils are severely corrosive to concrete and ferrous metals. If potentially corrosive soils are confirmed at the site during design level studies, the project design should implement appropriate controls to minimize the impact of corrosive soils on the proposed project. The design level geotechnical report will include recommendations by a Registered Corrosion Engineer after performing confirmation testing during the design phase of the project. These recommendations will include specific concrete mix designs, structural details for reinforced concrete foundations, and utility line protection that conforms to the California Building Code. As such, corrosive soils are not considered to be a hazard at the site after proper mitigation measures are implemented.

7.0 LIMITATIONS

The report and other materials resulting from GPI's efforts were prepared exclusively for use by Laguna Niguel Town Center Partners and their consultants in preparing the EIR for the proposed improvements. The report is not intended to be suitable for reuse on extensions or significant modifications of the project or for use on any project other than the currently proposed development as it may not contain sufficient or appropriate information for such uses. If this report or portions of this report are provided to contractors or included in specifications, it should be understood that they are provided for information only.

Soil deposits may vary in type, strength, and many other important properties between points of exploration due to non-uniformity of the geologic formations or to man-made cut and fill operations. While we cannot evaluate the consistency of the properties of materials in areas not explored, the conclusions drawn in this report are based on the assumption that the data obtained in the field and laboratory are reasonably representative of field conditions and are conducive to interpolation and extrapolation.

Our investigation and evaluations were performed using generally accepted engineering approaches and principles available at this time and the degree of care and skill ordinarily exercised under similar circumstances by reputable Geotechnical Engineers practicing in this area. No other representation, either express or implied, is included or intended in our report.

Respectfully submitted,
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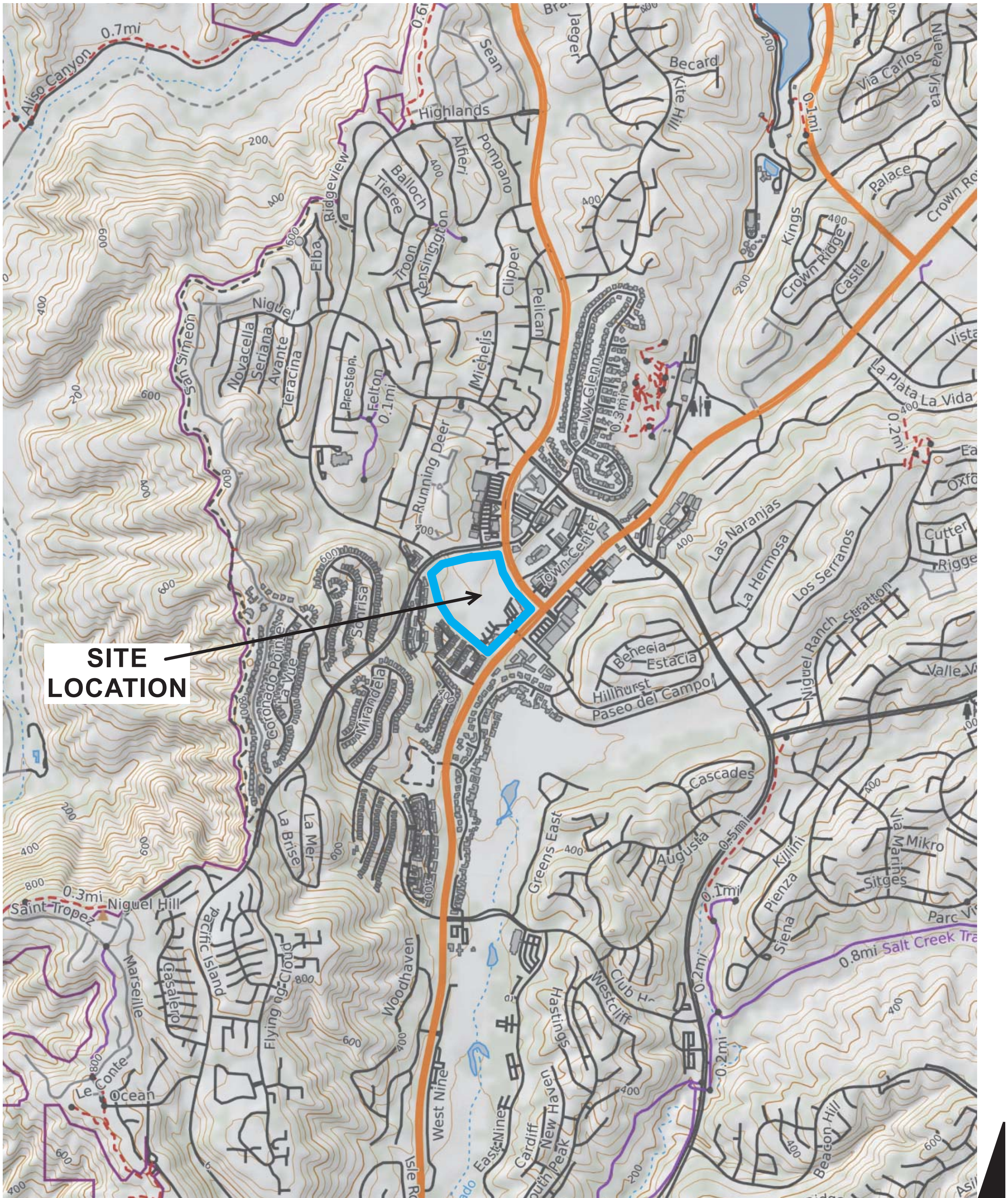

0 2000 4000 FEET

BASE MAP REPRODUCED FROM USGS 7.5' TOPO MAPS (WGS84 USNG ZONE 11SMT) © CALTOPO

GEOTECHNICAL PROFESSIONALS, INC.

LNTCP LAGUNA NIGUEL TOWN CENTER

GPI PROJECT NO.: 2952.I	SCALE: 1" = 2000'

SITE LOCATION MAP

FIGURE 1

EXPLANATION

B-5 APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING

C-5 APPROXIMATE LOCATION AND NUMBER OF CONE PENETRATION TEST

B-13 APPROXIMATE LOCATION AND NUMBER OF PRIOR EXPLORATION BY OTHERS (2005)

(5') APPROXIMATE DEPTH OF FILL (FEET)

SUBSURFACE CROSS SECTION

N

0 150 300 FEET

BASE MAP REPRODUCED FROM GOOGLE EARTH © 2018

GPI GEOTECHNICAL PROFESSIONALS, INC.

LNTCP LAGUNA NIGUEL TOWN CENTER

GPI PROJECT NO.: 2952.I

SCALE: 1" = 150'

EXISTING SITE PLAN

FIGURE 2

BASE MAP REPRODUCED FROM CONCEPTUAL GRADING & DRAINAGE PLAN (C1.00) PROVIDED BY BURNHAM WARD PROPERTIES: DATED 05/12/2021

GPI GEOTECHNICAL PROFESSIONALS, INC.

LNTCP LAGUNA NIGUEL TOWN CENTER

GPI PROJECT NO.: 2952.I | SCALE: 1" = 150'

PROPOSED SITE PLAN

FIGURE 3

EXPLANATION

Af_1	APPROXIMATE LIMITS OF FILL PLACED DURING PREVIOUS GRADING (CIRCA 1980-1981)
Af_2	APPROXIMATE LIMITS OF FILL PLACED DURING PREVIOUS GRADING OF EXISTING STRUCTURES AND PARKING AREAS
Tc	CAPISTRANO FORMATION
B-5	APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING
C-5	APPROXIMATE LOCATION AND NUMBER OF CONE PENETRATION TEST
B-13	APPROXIMATE LOCATION AND NUMBER OF PRIOR EXPLORATION BY OTHERS (2005)
(5')	APPROXIMATE DEPTH OF FILL (FEET)
-X- -X-	APPROXIMATE BOTTOM OF FILLED CANYON AND POSSIBLY OF CANYON SUBDRAINS
	GEOLOGIC CONTACT
	SUBSURFACE CROSS SECTION

N

0 150 300 FEET

BASE MAP REPRODUCED FROM TOPOGRAPHIC PLAN PROVIDED BY OWEN GROUP (UNDATED)

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LNTCP LAGUNA NIGUEL TOWN CENTER

GPI PROJECT NO.: 2952.I | SCALE: 1" = 150'

SITE GEOLOGIC MAP

FIGURE 4

BASE MAP REPRODUCED FROM THE GEOLOGIC MAP OF THE SAN JUAN CAPISTRANO QUADRANGLE, ORANGE COUNTY CALIFORNIA BY MORTON, EDGINGTON, AND FIFE: DATED 1974

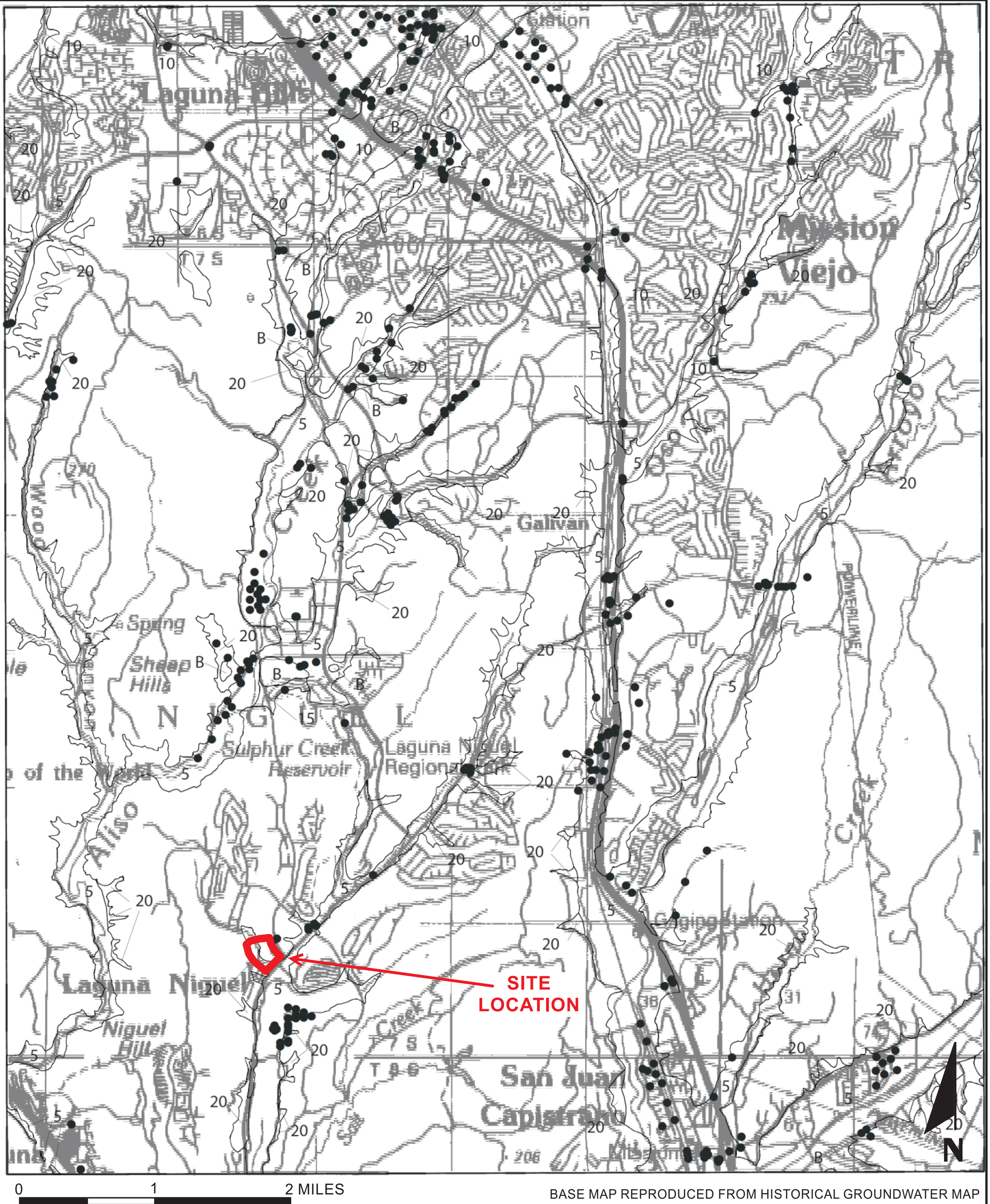
GEOTECHNICAL PROFESSIONALS, INC.

REGIONAL GEOLOGIC MAP

LNTCP LAGUNA NIGUEL TOWN CENTER

GPI PROJECT NO.: 2952.I | SCALE: 1" = 2000'

FIGURE 5



BASE MAP REPRODUCED FROM HISTORICAL GROUNDWATER MAP (PLATE 1.2) FROM SEISMIC HAZARD ZONE REPORT 053 (2001)

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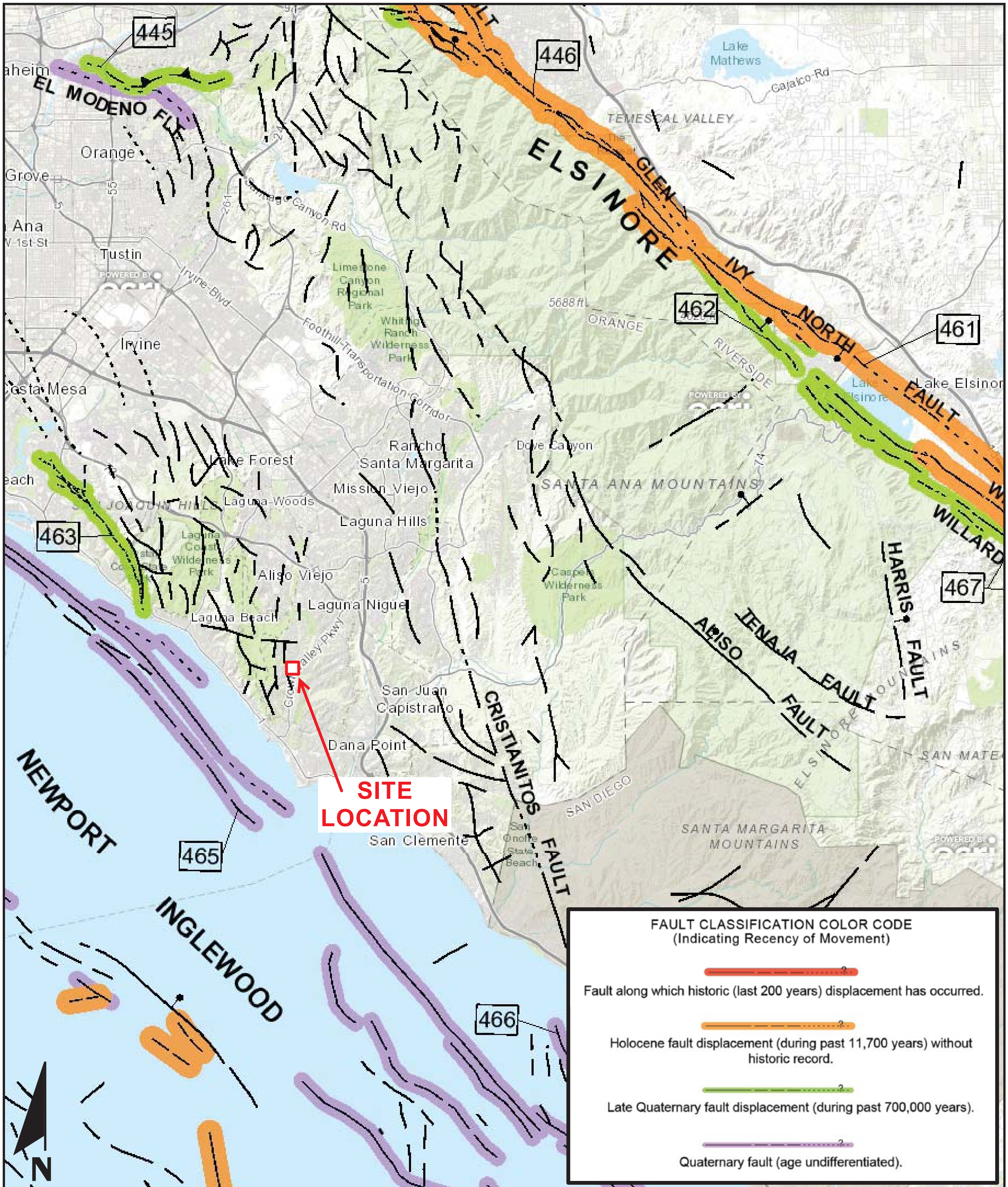
LNTCP LAGUNA NIGUEL TOWN CENTER

GPI PROJECT NO. 2952.I

SCALE: 1" = 1 MILE

HISTORICAL HIGH GROUNDWATER MAP

FIGURE 6



0 5 10 MILES

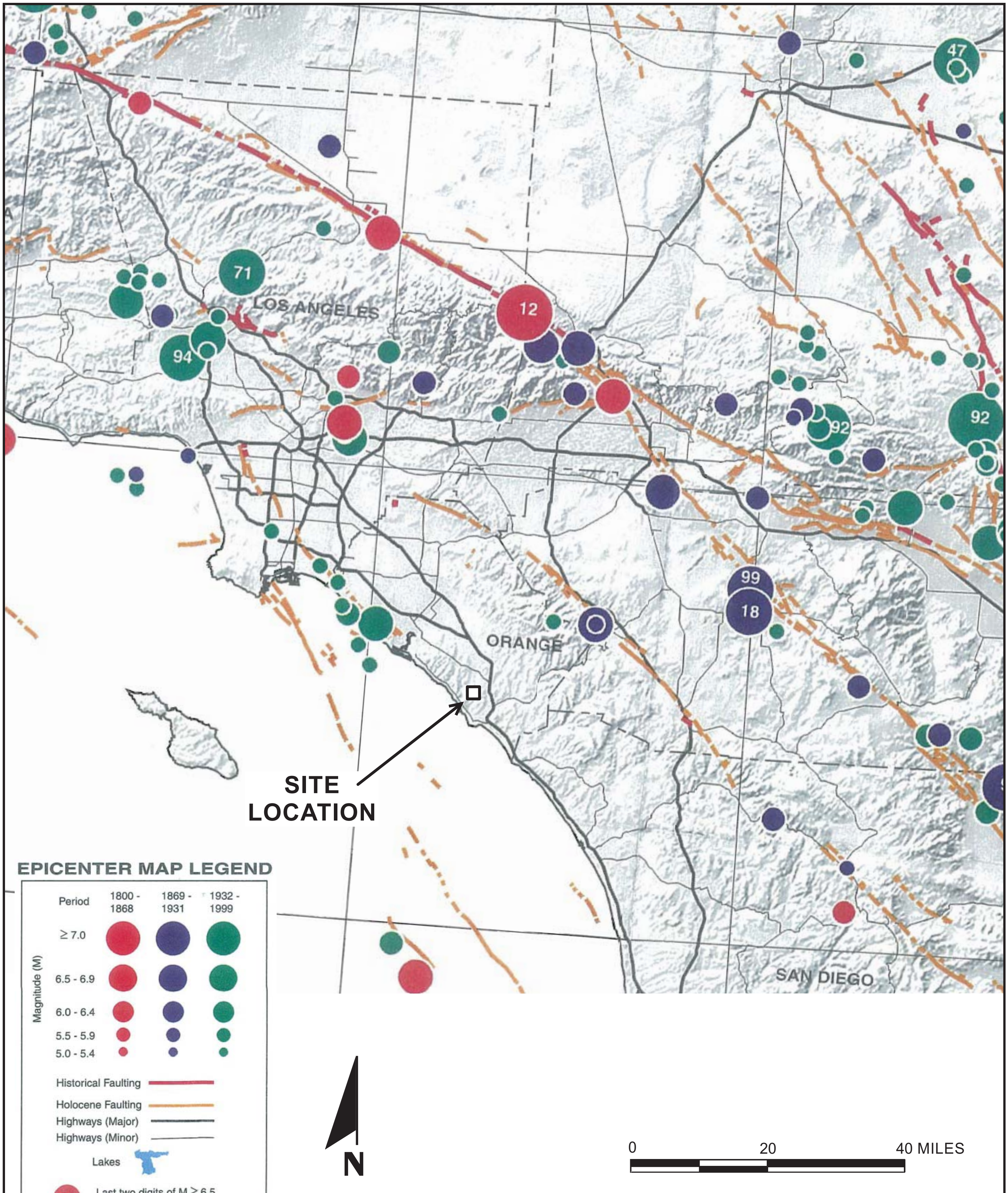
BASE MAP REPRODUCED FROM FAULT ACTIVITY MAP OF CALIFORNIA BY THE CALIFORNIA GEOLOGICAL SURVEY, C.W. JENNINGS, W.A. BRYANT: DATED 2010

GPI GEOTECHNICAL PROFESSIONALS, INC.

REGIONAL FAULT MAP

LNTCP LAGUNA NIGUEL TOWN CENTER

GPI PROJECT NO. 2952.I | SCALE: 1" = 5 MILES | FIGURE 7



REGIONAL FAULT MAP REPRODUCED FROM EARTHQUAKE FAULTS PLAN FROM MONTEREY PARK GENERAL PLAN BY BING YEN & ASSOCIATES, INC.: DATED JULY 2001

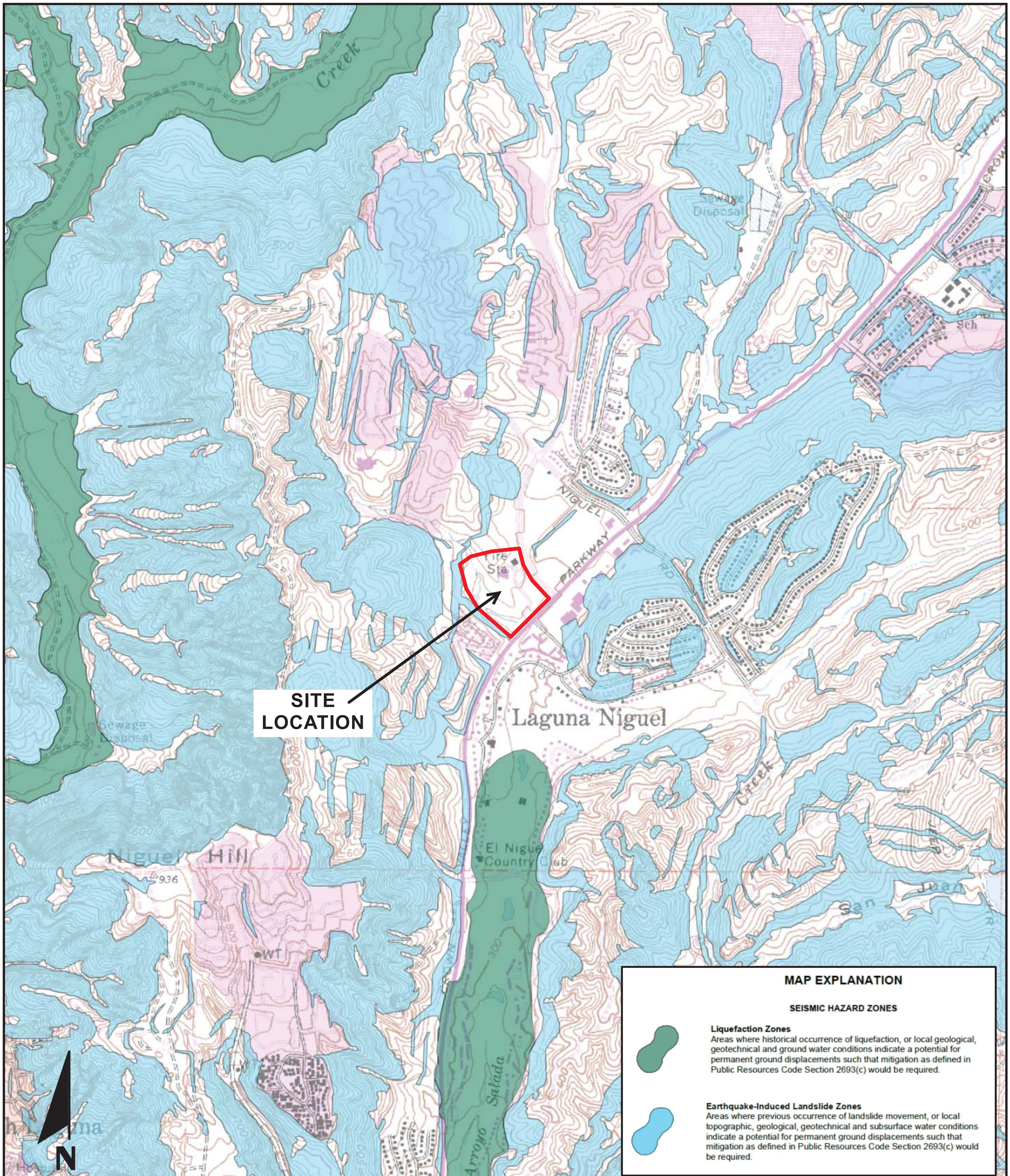
GPI GEOTECHNICAL PROFESSIONALS, INC.

LNTCP LAGUNA NIGUEL TOWN CENTER

GPI PROJECT NO.: 2952.I | SCALE: 1" = 20 MILES

REGIONAL SEISMICITY

FIGURE 8



0 2000 4000 FEET

BASE MAP REPRODUCED FROM EARTHQUAKE ZONES OF REQUIRED INVESTIGATION: SAN JUAN CAPISTRANO QUADRANGLE PREPARED BY THE CALIFORNIA GEOLOGIC SURVEY: RELEASED DECEMBER 21, 2001

GPI GEOTECHNICAL PROFESSIONALS, INC.

LNTCP LAGUNA NIGUEL TOWN CENTER

GPI PROJECT NO. 2952.I | SCALE: 1" = 2000'

SEISMIC HAZARDS ZONES MAP

FIGURE 9

APPENDIX A

APPENDIX A

CONE PENETRATION TESTS

The subsurface conditions were investigated by performing five Cone Penetration Tests (CPT's) at the site. The soundings were advanced to depths ranging from 30 to 50 feet below existing grades. The locations of the CPT's are shown on the Existing and Proposed Site Plans, Figures 2 and 3, respectively.

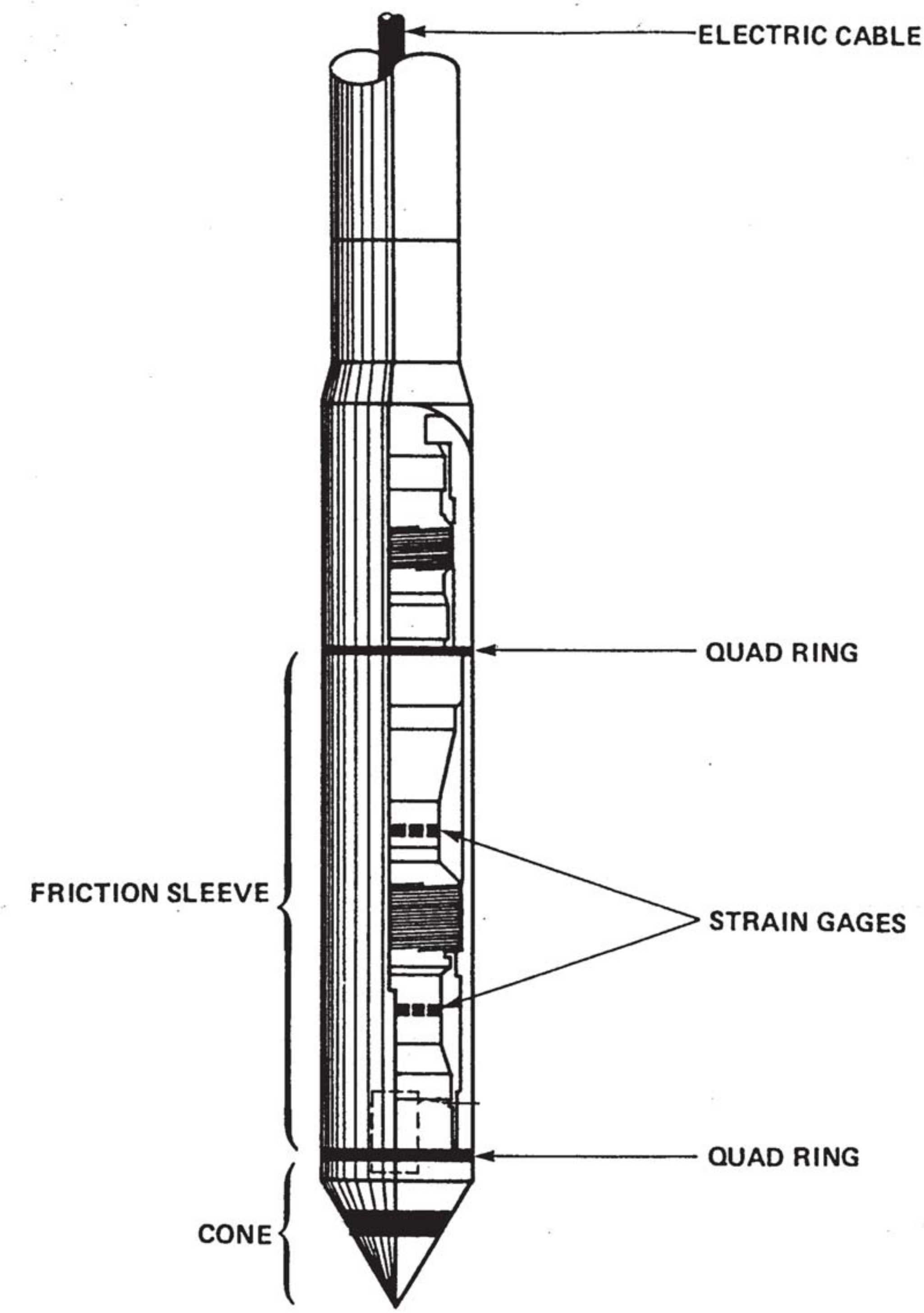
The Cone Penetration Test consists of pushing a cone-tipped probe into the soil deposit while simultaneously recording the cone tip resistance and side friction resistance of the soil to penetration (refer to Figure A-1). The CPT described in this report was conducted in general accordance with ASTM specifications (ASTM D 5778) using an electric cone penetrometer.

The CPT equipment consists of a cone assembly mounted at the end of a series of hollow sounding rods. A set of hydraulic rams is used to push the cone and rods into the soil while a continuous record of cone and friction resistance versus depth is obtained in both analog and digital form at the ground surface. A specially designed truck is used to transport and house the test equipment and to provide a 30-ton reaction to the thrust of the hydraulic rams.

Standard data obtained during a CPT consists of continuous stratigraphic information with close vertical resolution. Stratigraphic interpretation is based on relationships between cone tip resistance and friction resistance. The calculated friction ratio (CPT friction sleeve resistance divided by cone tip resistance) is used as an indicator of soil type. Granular soils typically have low friction ratios and high cone resistance, while cohesive or organic soils have high friction ratios and low cone resistance. These stratigraphic material categories form the basis for all subsequent calculations which utilize the CPT data.

Computer plots of the reduced CPT data acquired for this investigation are presented in Figures A-2 to A-6 of this appendix. The field testing and computer processing was performed by Kehoe Testing and Engineering under subcontract to Geotechnical Professionals Inc. (GPI). The interpreted soil descriptions were prepared by GPI.

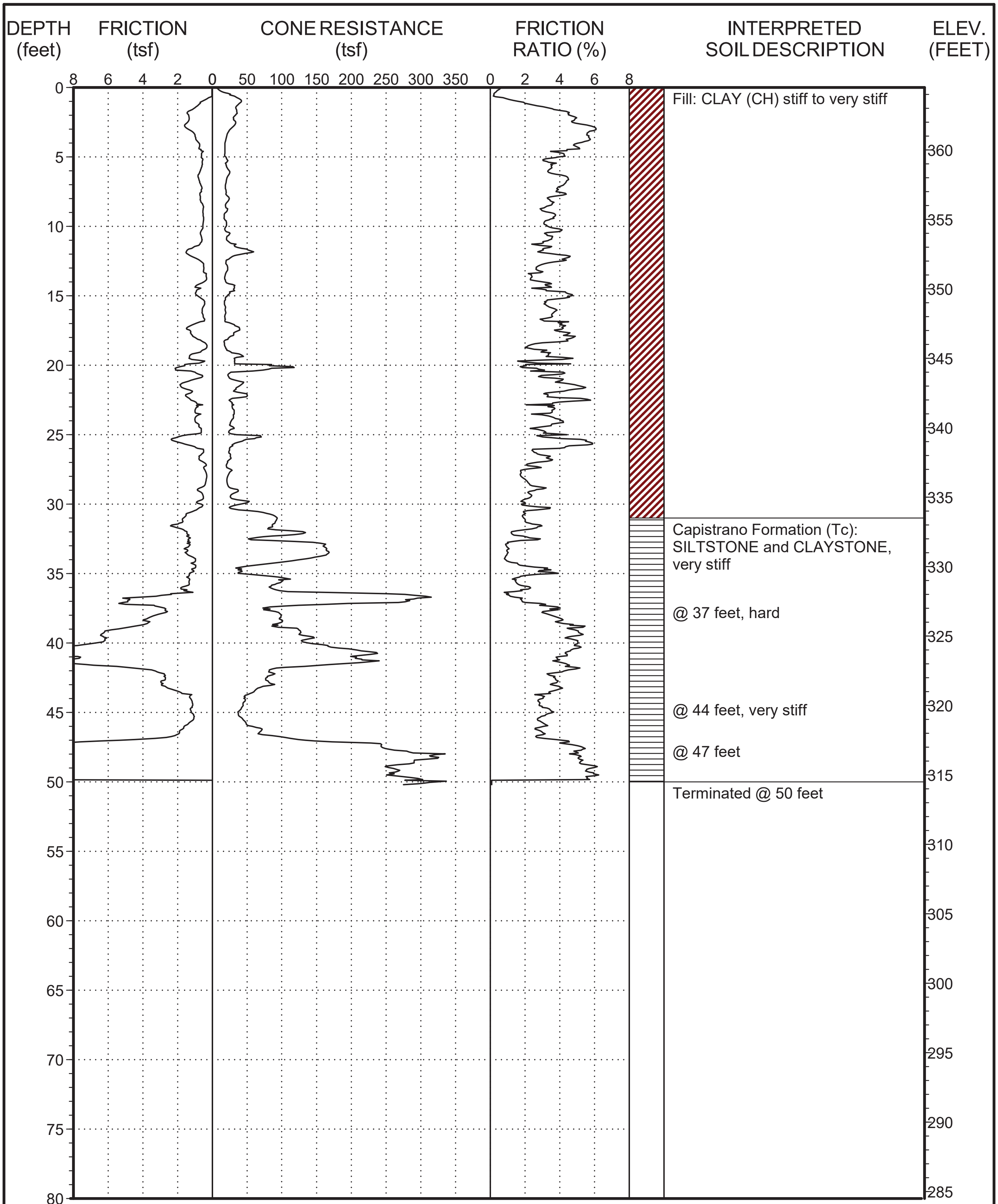
The CPT locations were laid out in the field by measuring from existing site features. Upon completion the uncaved portions of the CPT holes were backfilled with bentonite chips. Ground surface elevations at the CPT locations were estimated from topographic survey provided by Laguna Niguel Town Center Partners on August 7, 2019.



GEOTECHNICAL PROFESSIONALS, INC.

CONE PENETROMETER

FIGURE A-1



Date performed:7-3-19

This summary applies only at the location of this cone penetration test and at the time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The interpreted soil description is derived from the friction ratio and cone resistance and is a simplification of actual conditions encountered.

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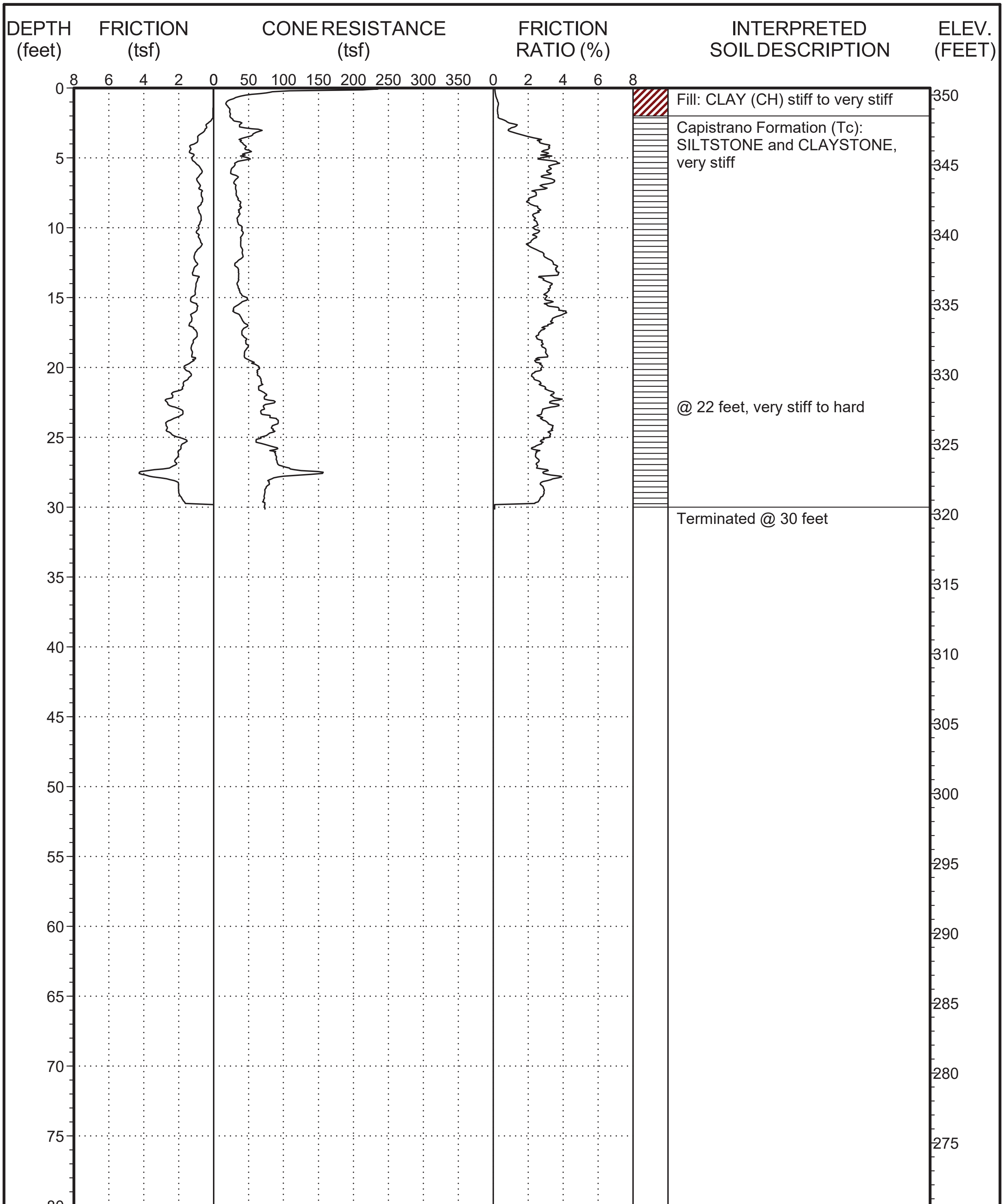
LOG OF CPT NO. C-1

FIGURE A-2



Date performed:7-3-19

This summary applies only at the location of this cone penetration test and at the time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The interpreted soil description is derived from the friction ratio and cone resistance and is a simplification of actual conditions encountered.

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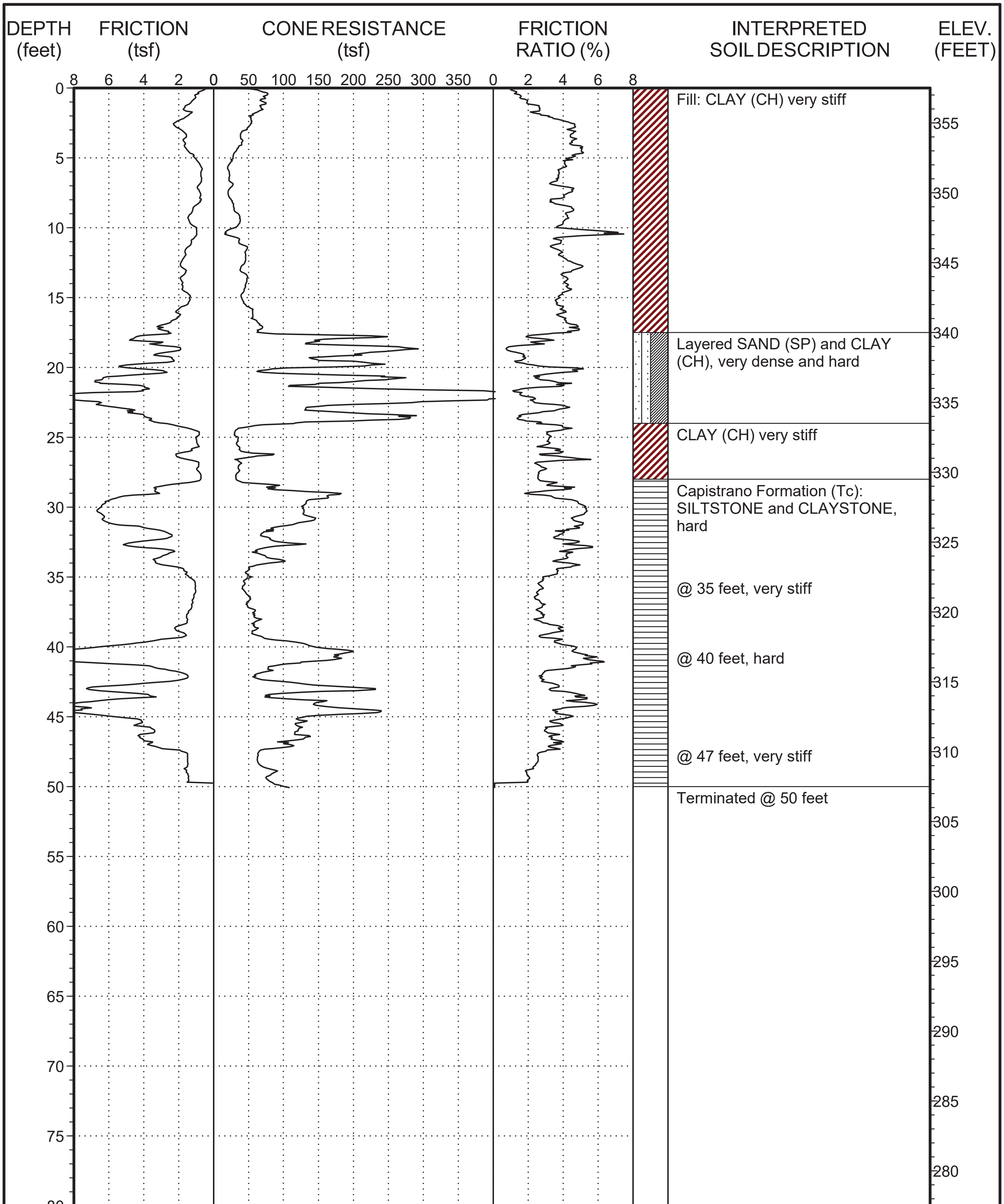
LOG OF CPT NO. C-2

FIGURE A-3



Date performed:7-3-19

This summary applies only at the location of this cone penetration test and at the time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The interpreted soil description is derived from the friction ratio and cone resistance and is a simplification of actual conditions encountered.

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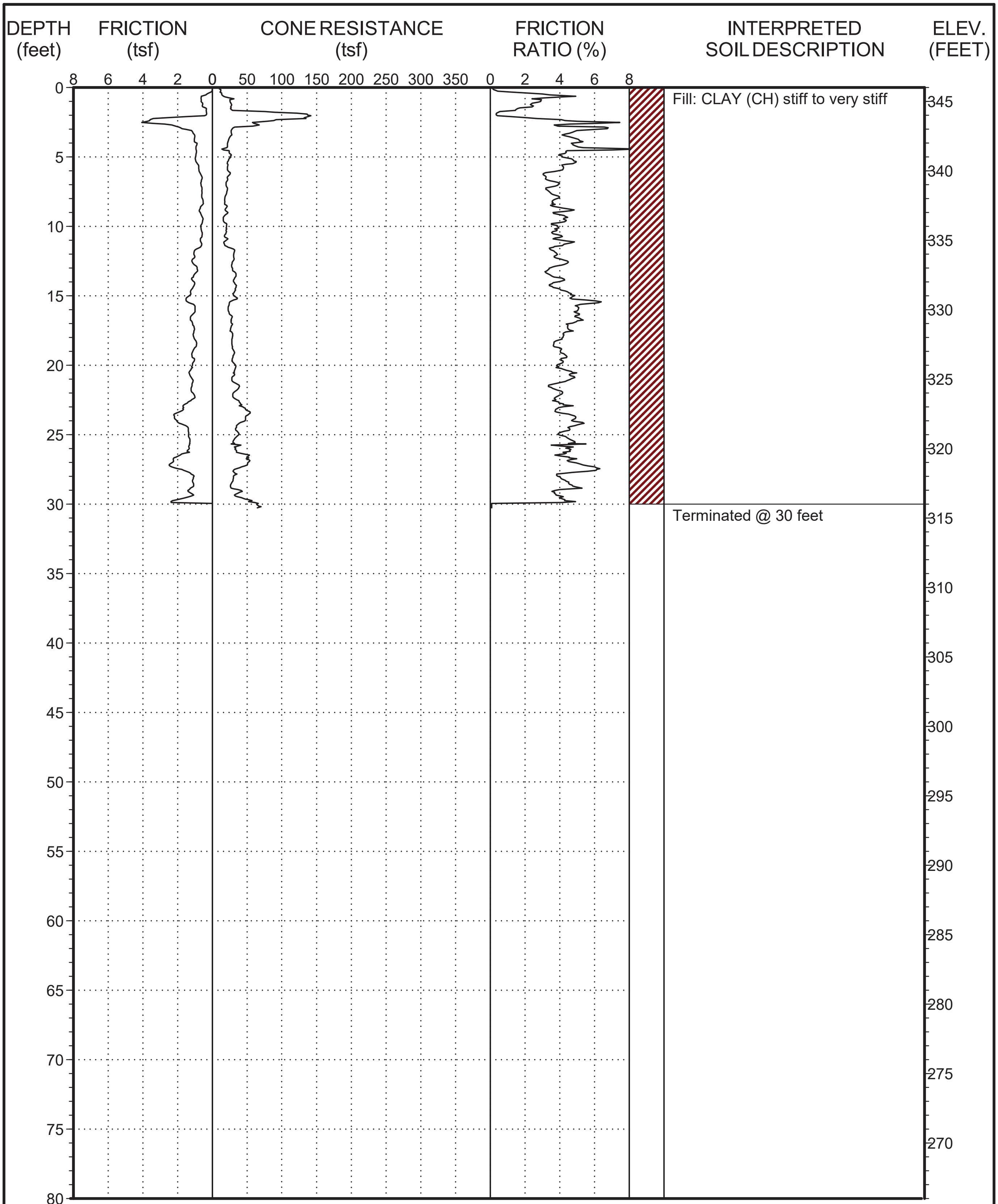
PROJECT NO.: 2952.I

LNTCP LAGUNA NIGUEL TOWN CENTER

LOG OF CPT NO. C-3

FIGURE A-4



Date performed:7-3-19

This summary applies only at the location of this cone penetration test and at the time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The interpreted soil description is derived from the friction ratio and cone resistance and is a simplification of actual conditions encountered.

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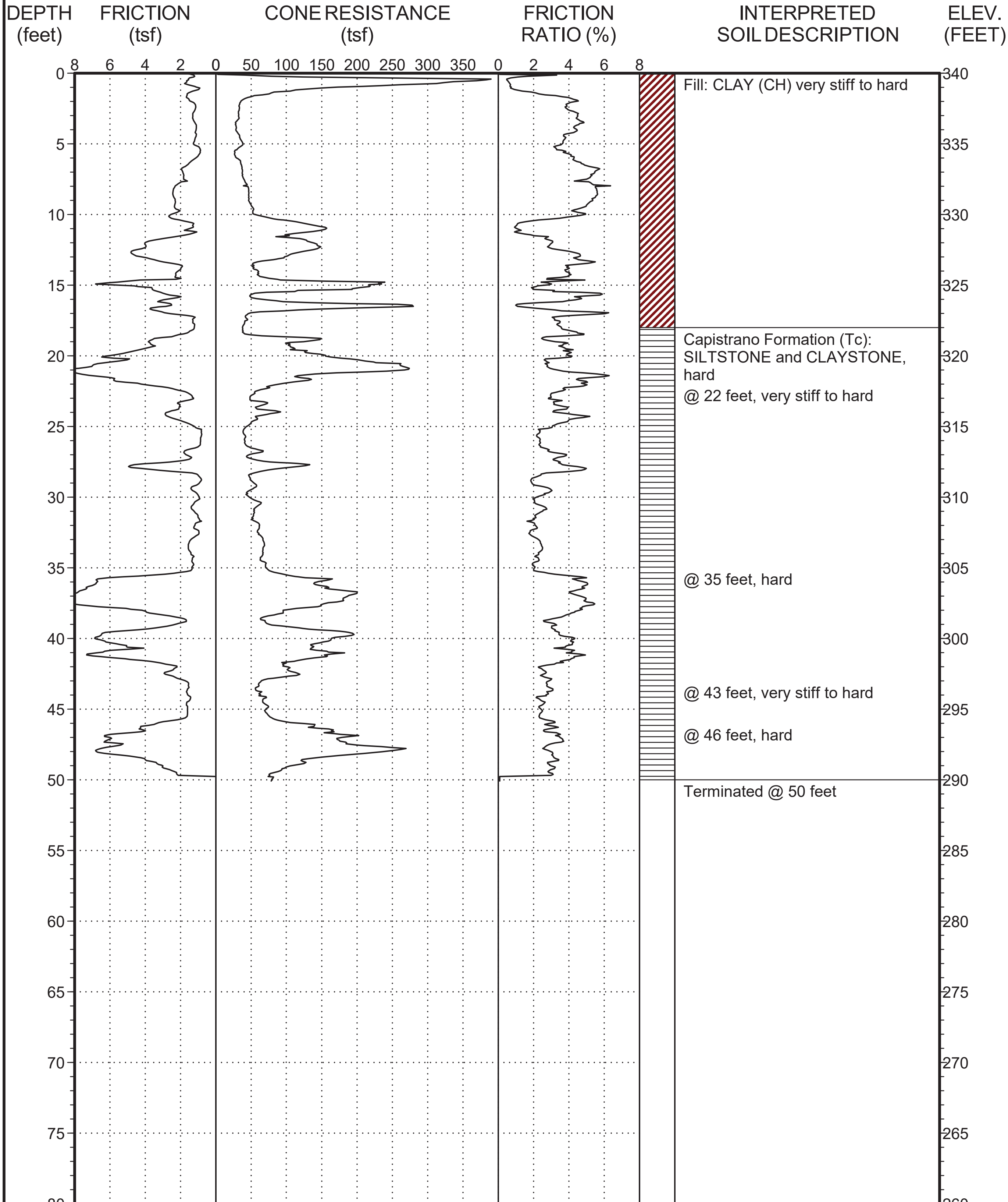
PROJECT NO.: 2952.I

LNTCP LAGUNA NIGUEL TOWN CENTER

LOG OF CPT NO. C-4

FIGURE A-5



Date performed: 7-3-19

This summary applies only at the location of this cone penetration test and at the time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The interpreted soil description is derived from the friction ratio and cone resistance and is a simplification of actual conditions encountered.

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LNTCP LAGUNA NIGUEL TOWN CENTER

LOG OF CPT NO. C-5

FIGURE A-6

APPENDIX B

APPENDIX B

EXPLORATORY BORINGS

The subsurface conditions at the site were investigated by drilling and sampling four exploratory borings. The borings were advanced to depths ranging from 31 to 48 feet below the existing ground surface. The locations of the explorations are shown on the Existing and Proposed Site Plans, Figures 2 and 3, respectively.

The exploratory borings were drilled using truck-mounted hollow-stem auger drill equipment. Relatively undisturbed samples were obtained using a brass-ring lined sampler (ASTM D 3550). The brass-rings have an inside diameter of 2.42 inches. The ring samples were driven into the soil by a 140-pound hammer dropping 30 inches. The number of blows needed to drive the sampler into the soil was recorded as the penetration resistance.

At selected locations, disturbed samples were obtained using a split-spoon sampler by means of the Standard Penetration Test (SPT, ASTM D 6066). The spoon sampler was driven into the soil by a 140-pound hammer dropping 30 inches, employing the “free-fall” hammer described above. After an initial seating drive of 6 inches, the number of blows needed to drive the sampler into the soil a depth of 12 inches was recorded as the penetration resistance. These values are the raw uncorrected blowcounts.

The field explorations for the investigation were performed under the continuous technical supervision of GPI's representative, who visually inspected the site, maintained detailed logs of the borings, classified the soils encountered, and obtained relatively undisturbed samples for examination and laboratory testing. The soils encountered in the borings were classified in the field and through further examination in the laboratory in accordance with the Unified Soils Classification System. Detailed logs of the borings are presented in Figures B-1 to B-4 in this appendix.

The boring locations were laid out in the field by measuring from existing site features. Ground surface elevations at the boring locations were estimated rom topographic survey provided by Laguna Niguel Town Center Partners on August 7, 2019.

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	*DESCRIPTION OF SUBSURFACE MATERIALS* This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
			B	0	Fill: **CLAY (CH)** mottled brown and grey, very moist, stiff, trace sand and silt	
21.9	97	17	D			
26.3	93	17	D		@ 4 feet, very moist to wet	360
				5		
29.6	90	15	D		@ 7 feet, wet	
						355
30.8	90	24	D	10	@ 10 feet, stiff to very stiff, trace gravel	
						350
31.4	94	17	D	15		
						345
28.1	101	21	D	20	@ 20 feet, dark grey to black, some sand	
						340
14.4	122	19	D	25	@ 25 feet, moist	
					Alluvium: **CLAYEY SAND (SC)** dark grey, very moist to wet, medium dense	335
13.3 13.0	123	26	D	30		
					Capistrano Formation (Tc): **SILTSTONE and CLAYSTONE** light brown and grey, very moist to wet, stiff to very stiff	
						330
32.4	88	20	D	35		
						325

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
7-10-19

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
25

GPI

PROJECT NO.: 2952.I

LNTCP LAGUNA NIGUEL TOWN CENTER

LOG OF BORING NO. B-1

FIGURE B-1

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
	34.5		16	S	40		
							320
	16.6	114	94/9"	D	45	@ 45 feet, moist, hard	
	19.9		50/2"	S		@ 47 feet, very moist	
						Refusal at 47.7 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
7-10-19
EQUIPMENT USED:
8 " Hollow Stem Auger
GROUNDWATER LEVEL (ft):
25

PROJECT NO.: 2952.I
LNTCP LAGUNA NIGUEL TOWN CENTER

LOG OF BORING NO. B-1

FIGURE B-1

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
			B	0	2" AC, 3" Base	350
					Fill: **CLAY (CH)** brown and orange, very moist, very stiff	
19.7	102	25	D		Capistrano Formation (Tc): **SILTSTONE and CLAYSTONE** brown and grey, very moist to wet, very stiff, trace sand and gypsum	
25.1	101	28	D	5		345
31.1	92	29	D		@ 6 feet, wet	
30.7	90	28	D			
31.8	89	38	D	10	@ 10 feet, with oxidized streaks	340
37.7	83	38	D	15		335
31.5	90	45	D	20	@ 20 feet, moist, very stiff to hard	330
26.1	97	53	D	25		325
31.9	90	38	D	30	@ 30 feet, very stiff	320
					Total Depth 31 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
7-10-19

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
14

GPI

PROJECT NO.: 2952.I

LNTCP LAGUNA NIGUEL TOWN CENTER

LOG OF BORING NO. B-2

FIGURE B-2

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
			B	0	Fill: **CLAY (CH)** brown, very moist, very stiff, trace sand	
18.3	101	28	D			
						350
20.1	66	24	D			
21.4	98	18	D	5	@ 5 feet, mottled dark brown	
						345
30.4	88	12	D		@ 8 feet, grey with oxidized streaks, wet, firm	
10.4	113	42	D	10	@ 10 feet, very stiff, trace gravel	
						340
31.5	82	13	D	15	@ 15 feet, stiff	
						335
16.2	115	22	D	20	@ 20 feet, dark grey to black, very moist, stiff to very stiff	
						330
19.1	112	33	D	25	@ 25 feet, moist to very moist	
						325
					Alluvium: **CLAYEY SAND (SC)** brown, very moist, dense to very dense	
13.0	118	75/10"	D	30		
					Capistrano Formation (Tc) **SILTSTONE and CLAYSTONE** brown and grey, very moist to wet, hard, trace gravel	
						320
25.5		47	S	35		
						315
19.1	106	82/11"	D		Total Depth 40 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
7-10-19

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
21

GPI

PROJECT NO.: 2952.I

LNTCP LAGUNA NIGUEL TOWN CENTER

LOG OF BORING NO. B-3

FIGURE B-3

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
			B	0	4" AC, 9.5" Base	340
					Fill: **CLAY (CH)** mottled brown, very moist, stiff	
21.9	98	20	D			
19.7	99	29	D		@ 4 feet, brown, very stiff, trace sand	
				5		335
15.0	112	40	D		@ 6 feet, dark brown to black, moist, trace gypsum	
15.9	113	54	D		**SANDY CLAY (CL)** brown with gypsum, slightly moist, hard	
8.0	93	84	D	10	Fill? **CLAYEY SAND (SC)** brown, moist, very dense, trace gravel	330
9.2	138	50/4"	D	15	@ 15 feet, orange and brown, with gravel fragments	325
17.5	106	78	D	20	Capistrano Formation (Tc): **SILTSTONE and CLAYSTONE** grey with oxidized streaks, moist, hard	320
30.2	90	28	D	25	@ 25 feet, wet, very stiff	315
31.5	86	50	D	30	@ 30 feet, dark grey to black (unoxidized)	310
					Total Depth 31 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
7-10-19

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

PROJECT NO.: 2952.I

LNTCP LAGUNA NIGUEL TOWN CENTER

LOG OF BORING NO. B-4

FIGURE B-4

APPENDIX C

APPENDIX C

LABORATORY TESTS

INTRODUCTION

Representative undisturbed soil samples and bulk samples were carefully packaged in the field and sealed to prevent moisture loss. The samples were then transported to our Cypress office for examination and testing assignments. Laboratory tests were performed on selected representative samples as an aid in classifying the soils and to evaluate the physical properties of the soils affecting foundation design and construction procedures. Detailed descriptions of the laboratory tests are presented below under the appropriate test headings. Test results are presented in the figures that follow.

MOISTURE CONTENT AND DRY DENSITY

Moisture content and dry density were determined from a number of the ring samples. The samples were first trimmed to obtain volume and wet weight and then were dried in accordance with ASTM D 2216. After drying, the weight of each sample was measured, and moisture content and dry density were calculated. Moisture content and dry density values are presented on the boring logs in Appendix B.

ATTERBERG LIMITS

Liquid and plastic limits were determined for selected samples in accordance with ASTM D 4318. The results of the Atterberg Limits tests are presented in Figure C-1.

GRAIN SIZE DISTRIBUTION

Selected soil samples were dried, weighed, soaked in water until individual soil particles were separated, and then washed on the No. 200 sieve. That portion of the material retained on the No. 200 sieve was oven-dried and weighed to determine the percentage of the material passing the No. 200 sieve. A summary of the percentages passing the No. 200 sieve is presented below.

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	PERCENT PASSING No. 200 SIEVE
B-4	10	Clayey Sand (SC)	38

DIRECT SHEAR

Direct shear tests were performed on undisturbed and remolded bulk samples in accordance with ASTM D 3080. The bulk samples were remolded to approximately 90 percent of maximum density (ASTM D 1557). The samples were placed in the shear machine, and a normal load comparable to the in-situ overburden stress was applied. The samples were inundated, allowed to consolidate, and then were sheared to failure. The tests were repeated on additional test specimens under increased normal loads. Shear stress and sample deformation were monitored throughout the test. The results of the direct shear tests are presented in Figures C-2 and C-3.

CONSOLIDATION

One-dimensional consolidation testing was performed on selected undisturbed samples in accordance with ASTM D 2435. After trimming the ends, the samples were placed in the consolidometer and loaded to 0.27 ksf. Thereafter, the samples were incrementally loaded to a maximum load of 34.1 ksf. The samples were inundated at 2.1 ksf. Sample deformation was measured to 0.0001 inch. Rebound behavior was investigated by unloading the samples back to 0.53 ksf. Results of the consolidation tests, in the form of percent consolidation versus log pressure, are presented in Figures C-4 and C-5.

COMPACTION TEST

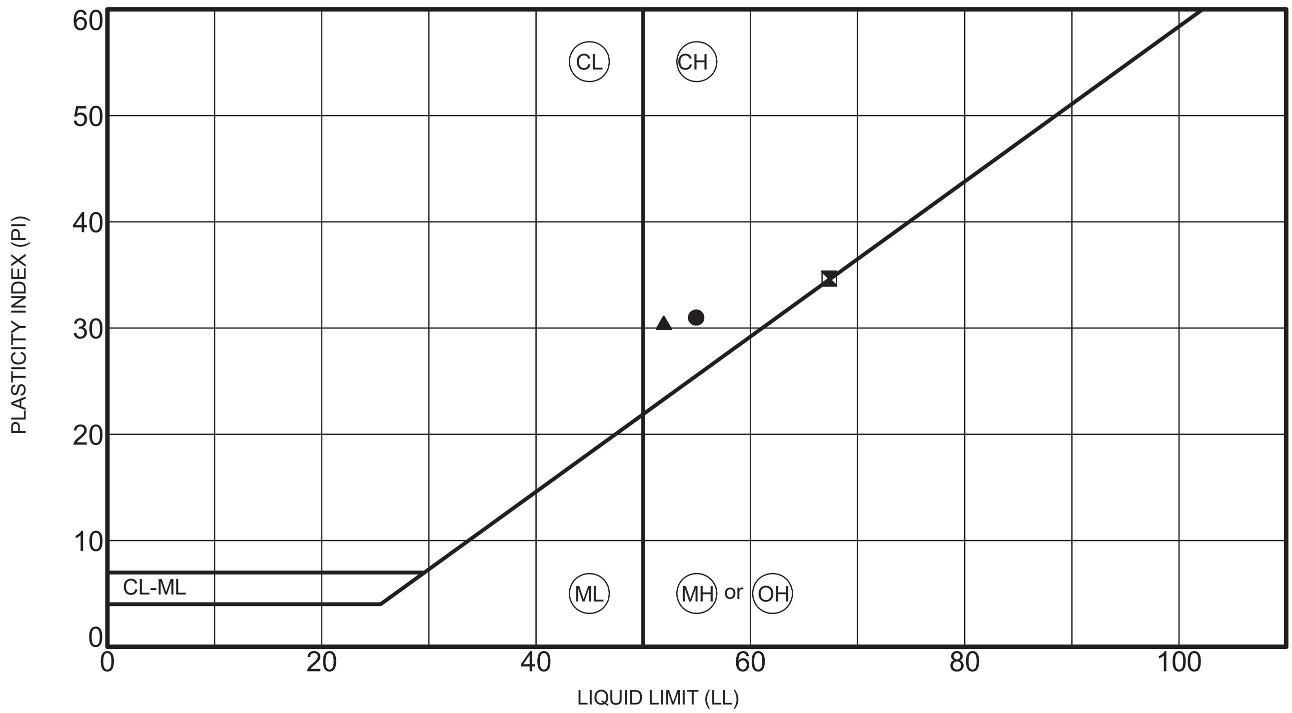
Maximum dry density/optimum moisture tests were performed on selected samples in accordance with ASTM D 1557 on representative bulk samples of the site soils. The test results are as follows:

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	OPIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)
B-1	0-5	Clay (CH)	14.0	116
B-4	0-5	Clay (CH)	13.0	119

EXPANSION INDEX

An expansion tests was performed in accordance with ASTM D 4829 on a bulk sample to assess the expansion potential of the on-site fill soils. The results of the test are summarized below.

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	EXPANSION INDEX
B-1	0-5	Clay (CH)	121



	SAMPLE LOCATION		LL	PL	PI	Fines, %	Classification
●	B-1	7.0	55	24	31		FAT CLAY (CH)
⊠	B-2	20.0	67	33	35		CLAYSTONE (Tc)
▲	B-4	4.0	52	21	31		FAT CLAY (CH)

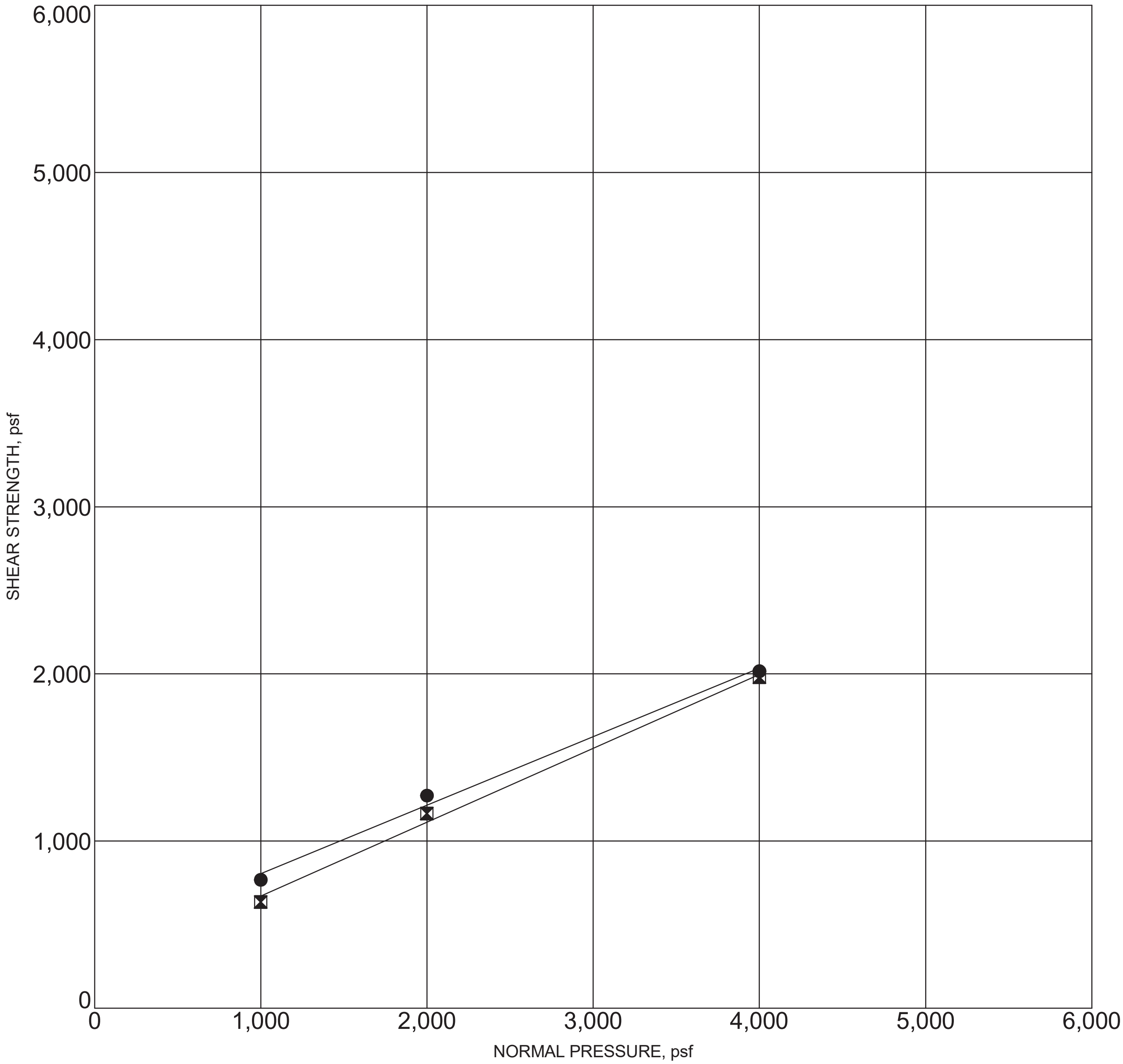
PROJECT: LNTCP LAGUNA NIGUEL TOWN CENTER

PROJECT NO. 2952.I

ATTERBERG LIMITS TEST RESULTS

G148

FIGURE C-1



● **PEAK STRENGTH**
Friction Angle= 22 degrees
Cohesion= 396 psf

⧗ **ULTIMATE STRENGTH**
Friction Angle= 24 degrees
Cohesion= 228 psf

Note: Samples remolded to 90% of maximum dry density

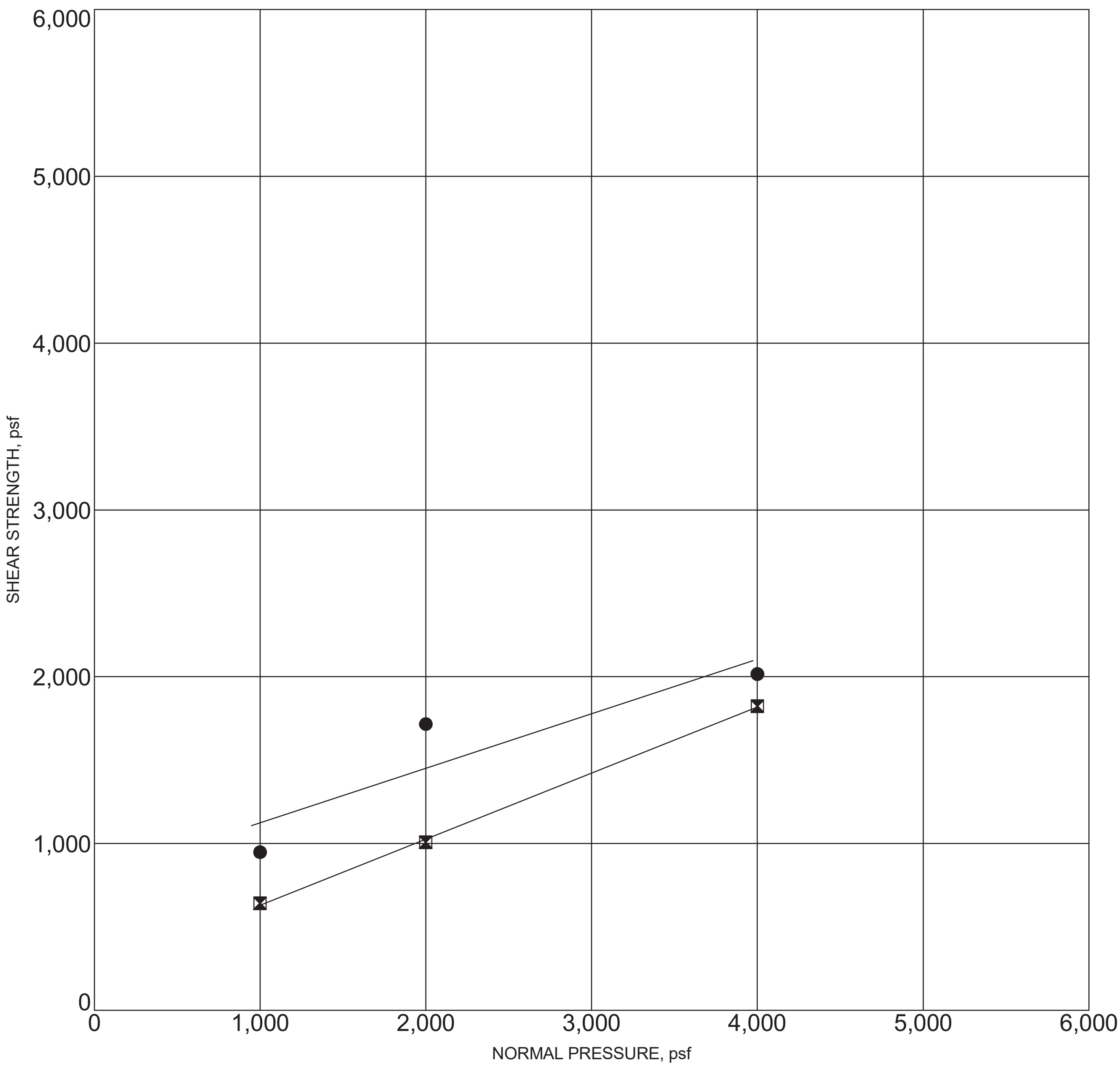
Sample Location		Classification	DD,pcf	MC,%
B-1	0-5	FAT CLAY (CH)	104	14.0

PROJECT: LNTCP LAGUNA NIGUEL TOWN CENTER PROJECT NO.: 2952.I

GPI

DIRECT SHEAR TEST RESULTS

FIGURE C-2



● **PEAK STRENGTH**
Friction Angle= 18 degrees
Cohesion= 798 psf

⧗ **ULTIMATE STRENGTH**
Friction Angle= 22 degrees
Cohesion= 234 psf

Sample Location		Classification	DD,pcf	MC,%
B-3	8.0	FAT CLAY (CH)	88	30.4

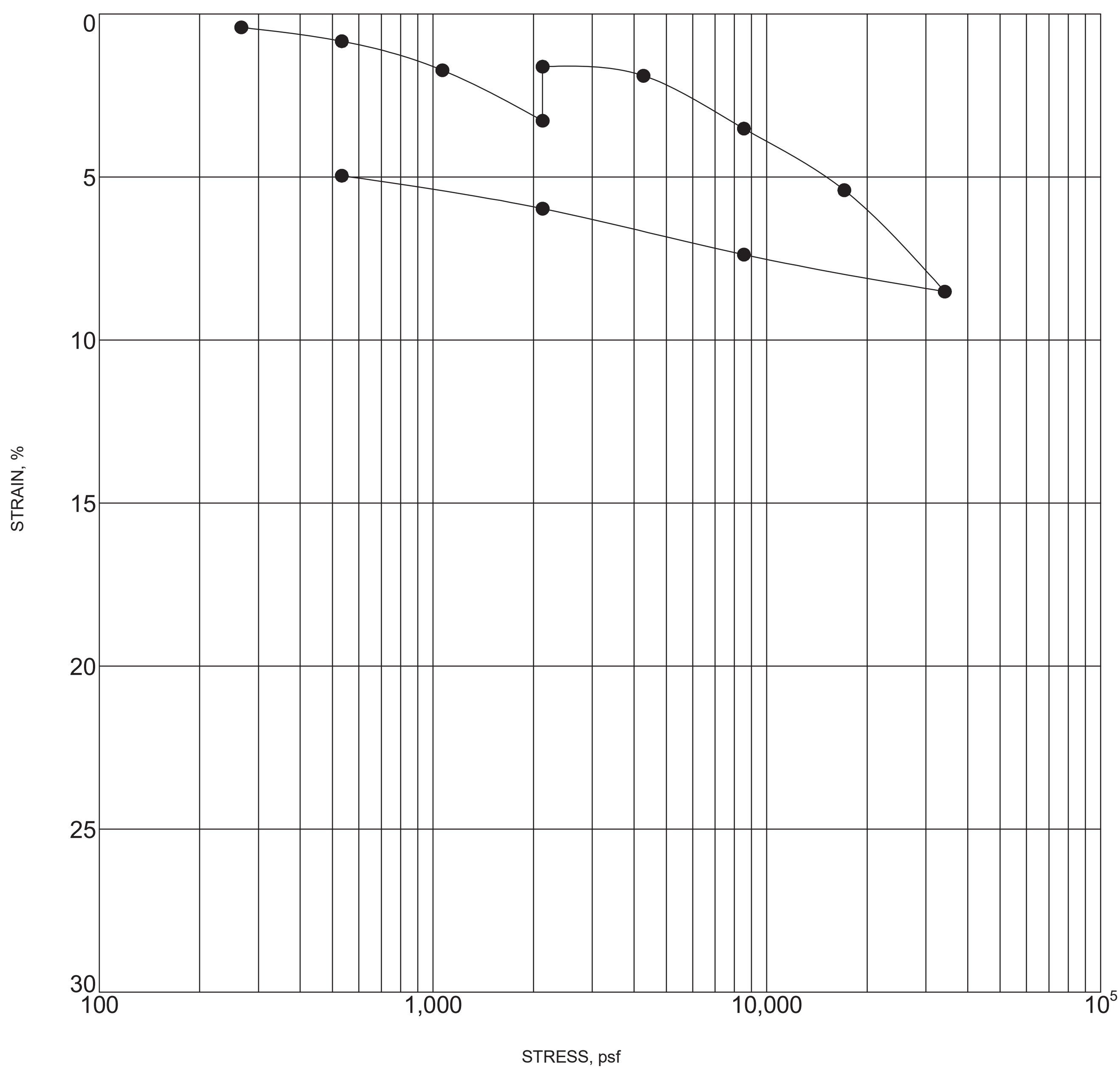
PROJECT: LNTCP LAGUNA NIGUEL TOWN CENTER PROJECT NO.: 2952.I

GPI

DIRECT SHEAR TEST RESULTS

G150

FIGURE C-3



Sample inundated at 2130 psf

	Sample Location		Classification	DD,pcf	MC,%
●	B-2	15.0	CLAYSTONE (Tc)	83	37.7

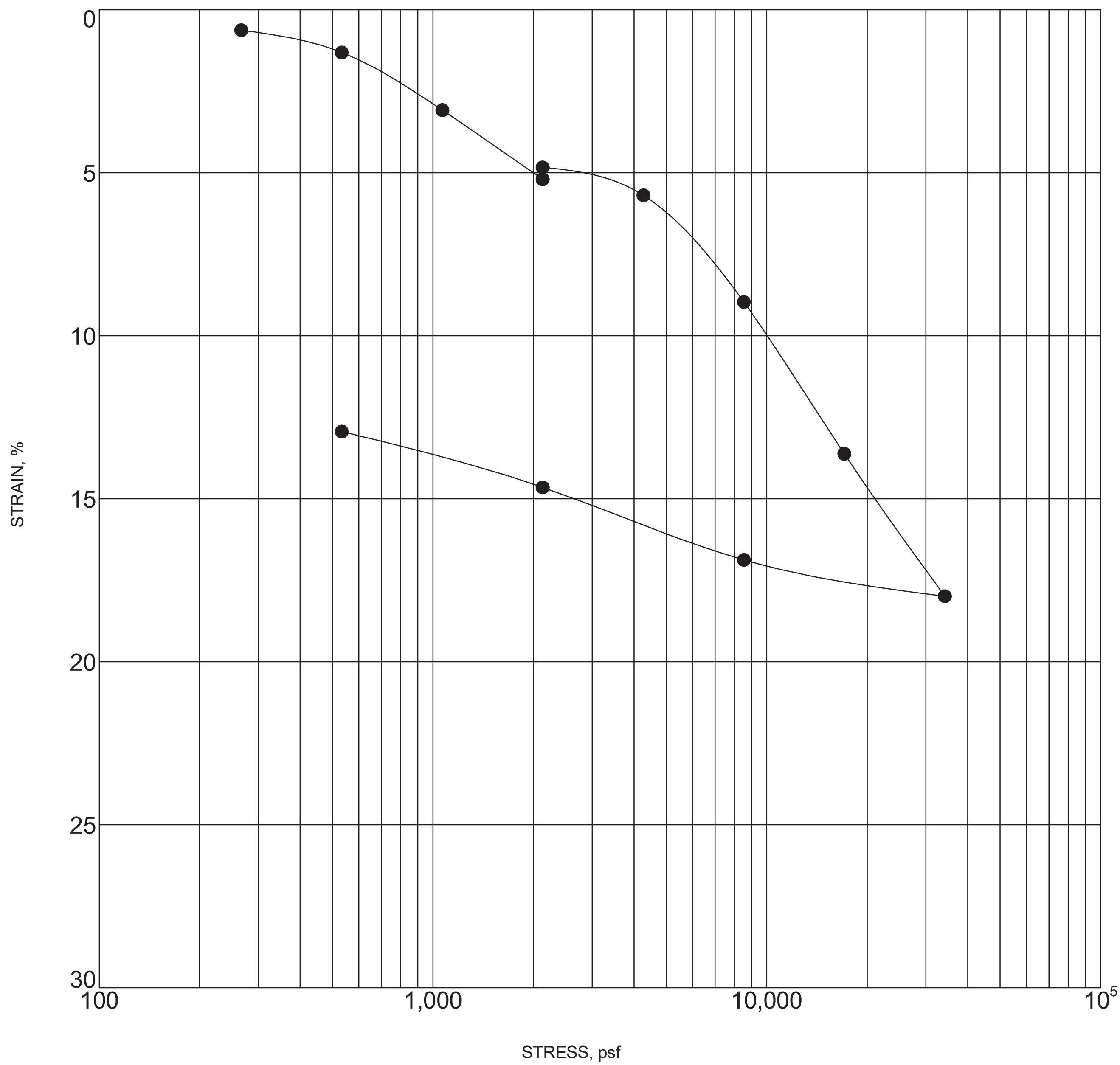
PROJECT: LNTCP LAGUNA NIGUEL TOWN CENTER PROJECT NO.: 2952.I

GPI

CONSOLIDATION TEST RESULTS

G151

FIGURE C-4



Sample inundated at 2130 psf

	Sample Location		Classification	DD,pcf	MC,%
●	B-3	15.0	Fill: FAT CLAY (CH)	82	31.5

PROJECT: LNTCP LAGUNA NIGUEL TOWN CENTER PROJECT NO.: 2952.I

GPI

CONSOLIDATION TEST RESULTS

G152

FIGURE C-5

APPENDIX D

APPENDIX D

CEQA APPENDIX G: ENVIRONMENTAL CHECKLIST FORM INPUT FOR SECTION VII GEOLOGY AND SOILS

The checklist below is provided for input into Section VII. Geology and Soils of the Appendix G CEQA Environmental Checklist Form for the proposed development. A brief explanation is provided below each item.

From CEQA Appendix G: Environmental Checklist	Potentially Significant Impact	Less Than Significant with Mitigation Incorporated	Less Than Significant Impact	No Impact
VI. GEOLOGY AND SOILS. Would the project:				
a) Directly or indirectly cause potential substantial adverse effects, including the risk of loss, injury, or death involving:				
i) Rupture of a known earthquake fault, as delineated on the most recent Alquist-Priolo Earthquake Fault Zoning Map issued by the State Geologist for the area or based on other substantial evidence of a known fault? Refer to Division of Mines and Geology Special Publication 42.	☐	☐	☐	☒
Brief Explanation: The site does not lie within an Alquist-Priolo Earthquake Fault Zone as designated by the California Geological Survey (CGS).				
ii) Strong seismic ground shaking?	☐	☐	☒	☐
Brief Explanation: The site could be subjected to strong ground shaking in the event of an earthquake, which could constitute a potential hazard to the project. The effects of strong seismic ground shaking can be mitigated by design and construction in conformance with current building codes and engineering practices. The project will be designed in accordance with the California Building Code.				
iii) Seismic-related ground failure due to liquefaction?	☐	☐	☐	☒
Brief Explanation: The site is not located within an area designated by the State Geologist as a "zone of required investigation" with respect to the potential for liquefaction. In addition, the subsurface soils consist primarily of fine-grained, highly plastic soils overlying dense to very dense bedrock materials. As such, liquefaction is considered unlikely at this site.				

From CEQA Appendix G: Environmental Checklist	Potentially Significant Impact	Less Than Significant With Mitigation Incorporated	Less Than Significant Impact	No Impact
iv) Seismic-related ground failure due to landslides?	☐	☒	☐	☐
Brief Explanation: The existing slopes ascending from the western and southwestern property boundaries are mapped within an area designated by the State Geologist as a "zone of required investigation" due to the potential for earthquake-induced landsliding (CGS, 2001). Because the majority of the western and southwestern slopes are off-site, mitigation measures for the potential slope instability will include construction of retaining structures to support the slopes where they extend onto the site and establishing adequate offsets between the base of the slopes and the proposed site structures. At this time, we anticipate retaining structures along the property line within the slope will consist of a soldier pile or equivalent retaining wall designed to resist lateral static and seismic earth pressures imposed by the adjacent slope. The design level geotechnical investigation will evaluate the suitability of a soldier pile retaining wall, providing geotechnical design parameters or recommendations for an equally or more effective mitigation measure. In addition to constructing retaining walls, current requirements of the California Building Code require a minimum lateral offset between the toe of a descending slope and the face of buildings at the base of the slope to be the smaller of 15 feet or one-half the height of the slope. The design level geotechnical investigation will evaluate the stability of the on-site and adjacent slopes, confirm the suitability of the offset, or provide an equally or more effective mitigation measure. *Based on a review of the preliminary grading plans, significant new permanent cut or fill slopes are not planned. The potential for seismic-related ground failure due to landsliding for the project is considered less than likely with mitigation measures outlined herein and confirmed in the design level geotechnical investigation report.*				
b) Result in substantial soil erosion or the loss of topsoil?	☐	☐	☒	☐
Brief Explanation: There is a potential for erosion of soils during construction, but this can typically be mitigated and/or significantly reduced with implementation of a Storm Water Pollution Prevention Plan (SWPPP). The potential for ongoing erosion during operation of the project is considered to be very low.				
c) Be located on a geologic unit or soil that is unstable, or that would become unstable as a result of the project, and potentially result in on- or off-site landslide, lateral spreading, subsidence, liquefaction or collapse?	☐	☐	☒	☐
Brief Explanation: The ascending slopes located to the west/southwest of the project site are located in a State designated Landslide Hazard Zone. Building Code requirements to mitigate the adverse effects of landslide will be incorporated into design. As such, there is a low potential that project improvements will be located on unstable soils or soils that would become unstable resulting in on- or off-site seismic-induced landslide, subsidence, or collapse. Because the potential for seismic-related liquefaction is considered unlikely at this site, the corresponding potential for lateral spreading to occur during liquefaction is also considered to be remote.				
d) Be located on expansive soil, as defined in Table 18-1-B of the Uniform Building Code (1994), creating substantial risks to life or property?	☐	☒	☐	☐

From CEQA Appendix G: Environmental Checklist	Potentially Significant Impact	Less Than Significant With Mitigation Incorporated	Less Than Significant Impact	No Impact
Brief Explanation: Highly expansive soils were encountered up to depths of 30 feet in recent and prior explorations at the site. As such, the potential for expansive soils to negatively impact the site of project improvements is considered likely if they are not properly mitigated. The project design should implement appropriate controls to minimize the impact of expansive soils on the proposed project, which will be provided in the design-level geotechnical report. Mitigation measures to reduce the adverse impact of expansive soils may include in-place chemical treatment of the expansive soils (cement or lime treatment or equivalent) or removal and replacement of the expansive soils with non-expansive import soils where the potential for shrink/swell is not tolerable. A structural control method that could be utilized would include design of foundations, floor slabs, and hardscape to resist the potential swell pressures of the expansive soils by increasing concrete reinforcing or using post-tension methods as outlined in the California Building Code.				
e) Have soils incapable of adequately supporting the use of septic tanks or alternative waste water disposal systems where sewers are not available for the disposal of waste water?	☐	☐	☐	☒
Brief Explanation: Septic tanks are not being considered for the project.				
f) Directly or indirectly destroy a unique paleontological resource or site or unique geological feature.	☐	☐	☐	☒
Brief Explanation: The development is not expected to destroy a unique geological feature. Paleontological resources were not evaluated as part of GPI's study.				

GEOTECHNICAL
PROFESSIONALS INC.

November 29, 2019

Laguna Niguel Town Center Partners
1100 Newport Center Drive
Newport Beach, CA 92660

Attention: Mr. Christian Santos
Development Director

Subject: Geotechnical Evaluation Report for CEQA
Proposed Laguna Niguel City Center
30102 Pacific Island Drive (Crown Valley Parkway at Alicia Parkway)
Laguna Niguel, California
GPI Project No. 2952.I/City of Laguna Niguel Reference No. SP19-13

Dear Christian:

As requested, this letter presents our response to comments provided by the City of Laguna Niguel on our report of geotechnical evaluation for CEQA dated October 11, 2019. The City's Geotechnical Review Sheet is dated October 28, 2019. The City's comments, followed by our responses, are as follows:

Comment 1: *The report states that a majority of the proposed grading will be "cut". Given the current ground surface evaluations, it seems like there may be some significant fills.*

- *Please discuss in more detail with regard to any planned fills.*
- *If the site is mostly cut, what type of cut volumes are estimated. Please discuss potential export from the site.*
- *Please provide conceptual/preliminary grading plans and representative cross sections through the site (Preliminary Grading Plans are noted on Page 12 of the report).*
- *Please discuss required cut and fill slopes.*

Response 1: We have recently been provided with the current conceptual grading and drainage plan (Sheet C12.00), conceptual sewer and water plan (Sheet C14.00), and conceptual site sections (Sheets C13.00, C13.01) by Fuscoe Engineering (dated October 18, 2019). The conceptual plans are attached. Based on current plans and our discussions with the Project Team, cuts on the order of 98,000 cubic yards and fills of about 10,000 cubic yards are planned. Based on our initial review of the conceptual plans, the areas of significant fills, greater than 3 feet above existing grades, appear to be limited to the following:

- An area at the northeast side of the site above the descending slope to Alicia Parkway near Pacific Island Drive (beneath the northeast apartment structure wrapping the parking structure), ranging from about 4 to 11 feet of fill,
- Within the footprint of the southernmost building in the northeast apartment structure area (structure with a finished floor elevation of +350 feet), up to about 5 feet of fill,

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- At the eastern side of the northeast apartment structure area, adjacent to the site entrance off of Alicia Parkway, up to about 3 feet of fill,
- Within the footprint of the centermost building in the central development area (structure with a finished floor elevation of +351 feet), up to about 4 feet of fill, and
- The easternmost portion of the southern parking lot, adjacent to the site entrance off of Crown Valley Parkway, up to about 8 feet of fill.

Given the difference between the cut and fill volumes, exporting approximately 90,000 cubic yards of soil is anticipated. The export operations will require trucking off-site to a suitable import site or facility.

Based on our initial review of the provided conceptual plans, cut and fill slopes are planned as part of the proposed site development. As stated above, fill slopes are required to reach planned finished grades on the northeast side of the site above the descending slope to Alicia Parkway and on the easternmost side of the southern parking lot, descending to the site entrance off of Crown Valley Parkway.

Cut slopes appear to be planned for the site entrance off of Pacific Island Drive (downgrades of 7.5 percent into the site). In addition, cuts that will require retaining walls are anticipated on the southwestern side of the southern parking area and adjacent to the northwestern and southwestern parking structures. If elected by the Project Team, retaining walls may not be used adjacent to the northwestern parking structure and the southwestern parking structures and, instead, the structures would be designed to resist lateral earth pressures.

Comment 2: *Please discuss foundation and/or fill loading of the existing fill and colluvial soils on site.*

- *Are settlement magnitudes anticipated to be problematic?*
- *Will there be any time-delayed settlements that could impact the construction schedule?*
- *Will any significant remediation be required?*

Response 2: Because of the limited amount of fill soils placed above the existing grades, as discussed above, areal settlement resulting from fill loading is not anticipated to be problematic. With that, additional explorations and laboratory testing will still be performed during our comprehensive, design-level geotechnical investigation to confirm the subsurface conditions and consolidation properties, including time-rate characteristics, of the soils underlying areas of significant planned fill. If problematic settlement magnitudes or time-rates are determined, remedial measures will be recommended and would likely include overexcavation and recompaction of the problematic soils or surcharging with stockpiled soils.

With respect to settlement of the proposed foundations resulting from consolidation of underlying fill and colluvial soils, the majority of the structures across the site, with the exception of those spanning over the deep in-place fill along the former tributary canyon in the western side of the site, will be supported on shallow foundations established in the undisturbed bedrock. For the structures mentioned above that will be in deeper planned fill areas and those structures along the western side of the site spanning over the deep in-place fill, remedial measures for foundation and floor slab support will be required. The specific remedial measures will be addressed during our design-level geotechnical investigation when further information on the deep fill soils is obtained through additional explorations and laboratory testing, as well as document research (see Response 3 below). At this time, we anticipate the remedial measures to be considered include overexcavation and recompaction, ground improvement methods such as rammed aggregate piers or soil-cement mixed columns/grouted inclusions, or potentially a combination of these measures in combination with increased foundation stiffness from grade beams used with spread footings, post-tension foundations, or a mat foundation. Depending on the expected foundation loads, which have not yet been developed at this early stage of the project, we will also evaluate the need for deep foundations, likely consisting of auger pressure grouted piles.

Comment 3: *Please describe anticipated corrective grading.*

- *Please specifically address the artificial fills encountered on site.*
  - *Are they suitable for development support?*
  - *Are they certified?*
  - *To what depth and extent are the artificial fills anticipated to be removed?*
- *Please address the potential impacts of the corrective grading on off-site properties and structures.*

Response 3: The existing on-site fill soils are not considered to be suitable for direct support of the proposed structures in their current condition. The depths and characteristics of the existing fill soils encountered in our limited explorations and the explorations from others was detailed in our prior report (GPI, 2019). More detailed recommendations for remedial grading, including the depths and extent of fill and alluvium/colluvium removals, will be included in our design-level geotechnical report when data is available from additional explorations. Findings and recommendations for remedial grading based on the available information are presented below.

In an early report of compacted fill and final geologic study (Moore and Taber, 1968), documentation is provided for early rough grading performed along Alicia Parkway between Crown Valley Parkway and Pacific Island Drive, as well as in the center portion of the site, east of the former tributary canyon that was open at the time (1968). The report copy available included text and a plan with the location of field tests, but did not include the test results or details on the depths of the testing or newly placed fills. The report does indicate that the fill soils were found to be compacted to 90 percent of the maximum dry density determined by ASTM D1557-64T (modified proctor test with five layers), with the exception of the upper 3 feet from finished grade, which was compacted to between 85 and 90 percent, lower than the current standard of at least 90 percent. Given that the available documentation is limited, we will recommend specific testing in these fill areas to better determine the suitability of the soils to support additional fill, structures, and pavements.

As noted above, we anticipate the majority of the proposed structures to be supported directly on the undisturbed bedrock because of the planned cuts. It appears that localized structures, such as some of the apartment structure at the northeast corner of the site (Pacific Island Drive and Alicia Parkway), will require some fill soils to meet the proposed finished grades, so in this case the existing surficial fill soils and any colluvium or alluvium will need to be overexcavated and properly recompacted within the building area, including construction of an adequate key if building a fill slope, before additional fill is placed.

As noted in Response 2 above, additional information is required regarding the deep fill soils placed within the former tributary canyon along the western side of the site. We have been provided with a geotechnical investigation report by Westland Associates (Westland, 1978) for the proposed South Coast Civic Center project that provided recommendations for remedial grading that was apparently performed in 1981 and 1982 and including the filling of the former tributary canyon. Based on our review of available information by others (Group Delta, 2005 and GMU, 2007), we are aware of geotechnical reports documenting the rough grading and fine grading of the site in 1981 and 1982 for the South County Civic Center, respectively (Westland, 1981; 1982), but we have not reviewed those reports as of yet. It is our understanding that the rough and fine grading reports from Westland address the grading associated with the filling of the tributary canyon that previously existed on the west side of the subject site. Without those reports, the fill soils within this area are considered undocumented or uncertified, similar to the other shallow fill soils encountered across the other areas of the site. Within the former tributary canyon area, we encountered fill soils to depths of 20 to 29 feet below the existing grades. Although not encountered in our limited explorations in this area, we anticipate deeper fills may exist along the former tributary canyon. A prior boring by others encountered the fill soils to a depth of 34 feet in this area (Group Delta, 2005).

If documentation regarding the placement and compaction of the deep fill soils within the former tributary canyon are obtained and found to properly document the materials, we still anticipate that the structures supported over the materials will require ground improvement measures for foundation support as discussed in Response 2 above. Along with the ground improvement measures, some remedial grading (e.g. overexcavation and recompaction) is anticipated for support of floor slabs, hardscape, and pavements. This remedial grading will include the placement of imported, select non-expansive soils or cement/lime treatment of the on-site expansive soils to cap floor slab and hardscape subgrade.

If documentation regarding the placement and compaction of the deep fill soils is not obtained or does not support considering the fill soils as being properly documented or certified, the soils will not be considered suitable for support of foundations or floor slabs. In this event, remedial measures may include overexcavation and recompaction of the fill soils in their entirety, partial removal of the fill soils with ground improvement to support foundations and floor slabs, or deep foundations extending through the fill soils and into the underlying bedrock with a structurally supported floor slab.

Complete removal and recompaction of the existing fill soils may not be feasible within the former tributary canyon because of the potential adverse impact to the temporary stability of the adjacent ascending slopes to the west and southwest. Prior to performing removals or fill placement adjacent to the property lines or within the influence of adjacent sites, we will evaluate the potential impacts of the grading and provide recommendations for remedial measures. These measures will be evaluated during the design-level geotechnical investigation and may include temporary shoring and slot cutting.

Comment 4: *Please specifically address the following with regard to groundwater:*

- *During grading of the LN City Hall site, there was significant groundwater encountered.*
    - *Please specifically comment upon the groundwater observations made during design and construction of the LN City Hall site and how they may*

impact the proposed project. Note: groundwater was observed during construction seeping out of the back western comer of the lot in the vicinity of "C-6" very close to the ground surface.

- *Please address whether groundwater will impact anticipated corrective grading removals.*
- *Please address whether any dewatering will be required during construction and how the discharge will be handled.*
- *How will the development change the groundwater conditions at the site?*
    - *Will special sub-drainage be required?*
    - *How will any increase in groundwater affect off-site properties?*
- *Please address whether groundwater will impact the proposed development in any significant way.*

Response 4: Groundwater is discussed in our October 11, 2019 report. We noted that historical groundwater depths range from 5 to 20 feet in the vicinity of the site. Based on the information available, including the experience of GMU and the City of Laguna Niguel during construction of the City Hall, we anticipate that groundwater will be an issue that will need to be addressed

in our design-level geotechnical investigation report. Similar to the City Hall project, the potential for groundwater seepage will need to be accounted for in both construction and design of the planned project.

During construction, the contractor should anticipate the potential for groundwater seepage when planning cuts below the existing grades. Measures, such as trench drains, to collect and discharge water seepage in a suitable manner may be required during remedial grading. Groundwater will also need to be accounted for in the installation of deep ground improvement methods, such as rammed aggregate piers, or temporary shoring. Discharge of groundwater should be performed by the project contractor in accordance with regulatory requirements.

For design, the potential for groundwater seepage will need to be considered for below grade structures such as retaining walls and basement walls. Such considerations will include subdrains for below grade walls and floor slabs, or waterproofing and designing below grade structures to resist the hydrostatic pressures in addition to the earth pressures.

The project is not anticipated to increase the extent of groundwater from the existing seasonal variations, as the majority of the site will be covered with buildings, hardscape, and pavements with surface water collection and discharge systems as opposed to the current exposed ground surface conditions. As such, we do not anticipate off-site properties being impacted by the proposed development's impact on site groundwater levels.

Comment 5: *Will the cohesive native alluvial materials below the site undergo any significant strength loss during an earthquake so as to induce significant lateral spreading movements?*

- *Please address relative to the ASCE 7-16 and the 2019 CBC*

Response 5: Based on our initial findings, we do not anticipate the native alluvial materials to remain in-place after the planned cuts and remedial grading are performed as part of the planned construction. If, during our design-level geotechnical investigation we determine that there will be highly plastic, sensitive native alluvial soils remaining in-place based on the finished grades and recommended remedial grading, we will evaluate the soils and their potential for strength loss during the design-level earthquake in accordance with the 2019 CBC. In addition, we will also evaluate the potential for lateral spreading at the subject site should our analysis identify the potential for soil strength loss during an earthquake event (i.e. liquefaction) in accordance with requirements of the 2019 CBC.

Comment 6: *Given that the existing slopes and pads to the south and southwest have undergone slope movements, please explicate in further detail with regard to how additional movements during construction will be evaluated and mitigated.*

- *What types of monitoring and pre-construction surveys will be required?*
- *How will any walls be constructed at the toe of slope to minimize the potential for construction-related slope movements?*
- *Will the proposed parking structure have a subterranean component? If so, how many stories are proposed? How will movements be mitigated?*
- *The report mentions the 2016 code. However, the development will be processed under the 2019 CBC. Please address any ramifications.*

Response 6: We will develop recommendations for monitoring the presence of additional slope movement during and after construction along the toe of the existing ascending slopes to the south and southwest of the site in our design-level geotechnical investigation report. At this time, we anticipate the monitoring will include the installation of inclinometer casing at several locations along the property line. The inclinometers will be installed, read to obtain a baseline reading before earthwork commences, and then read at regular intervals during and after construction. In addition to inclinometers, we may recommend other monitoring methods, such as a conventional survey of benchmarks installed along the property line to measure both vertical and lateral movement. We will also recommend a condition survey of the structures at the top of the ascending slope, including a conventional survey and video/photographic survey to document the baseline conditions prior to the commencement of grading activities at the site.

Walls at the toe of the slope will be constructed in a top-down methodology to minimize the potential for construction-related slope movements, likely using permanent soldier pile and lagging walls. The walls will be designed for the relatively high lateral earth and seismic pressures imposed by the inclined, ascending supported soils.

The parking structures will not have a true subterranean level, but the back of the structures near the toe of the existing slopes will extend below grade. Current plans call for the finished floor to extend up to about 9 feet below grade. The project team will evaluate whether the back wall of the parking structure will be designed as a retaining wall to resist the lateral earth and seismic pressures, or if a separate retaining wall using top-down construction as discussed previously will be used.

The subject project will be designed and constructed to meet the requirements of the 2019 CBC. We do not anticipate significant ramifications for the project under the 2019 CBC as opposed to geologic and geotechnical issues previously identified in our October 11, 2019 report and this letter. The changes, such as with respect to the seismic design parameters and requirements for site-specific response spectra, will be properly addressed in our design-level report.

We trust that this letter provides the requested clarification. Please contact us if you have questions on the above responses or need additional information.

Very truly yours,
Geotechnical Professionals Inc.

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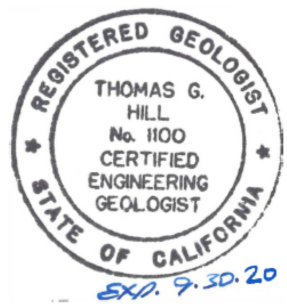

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