GEOTECHNICAL PROFESSIONALS INC.

GEOTECHNICAL INVESTIGATION
PROPOSED ATHLETIC FIELD RENOVATION
LYNWOOD HIGH SCHOOL
4050 E IMPERIAL HIGHWAY
LYNWOOD, CALIFORNIA

Prepared for:
Lynwood Unified School District
11321 Bullis Road
Lynwood, California 90262

Prepared by:
Geotechnical Professionals Inc.
5736 Corporate Avenue
Cypress, California 90630
(714) 220-2211

GPI Project No. 3328.I

October 3, 2025

GEOTECHNICAL PROFESSIONALS INC.

August 1, 2025 (revised October 3, 2025)

Lynwood Unified School District
11321 Bullis Road
Lynwood, California 90262

Attention: Dr. Gretchen Janson
Asst. Superintendent Business Services

Subject: Geotechnical Investigation
Proposed Athletic Field Renovation
Lynwood High School
4050 E Imperial Highway
Lynwood, California
GPI Project No. 3328.I

Dear Gretchen:

Transmitted herewith is an electronic copy of our revised geotechnical investigation report for the proposed athletic field renovation at Lynwood High School in Lynwood, California.

We appreciate the opportunity of offering our services to your organization and look forward to seeing the project through its successful completion. Please do not hesitate to call us if you have any questions on the contents of our report or need further geotechnical assistance.

Very truly yours,
Geotechnical Professionals Inc.

Dylan J. Boyle, G.E.
Senior Engineer

Justin J. Kempton, G.E.
Principal

Distribution: Rikki Perez, Huckabee
Jorge Ramirez, Lynwood Unified School District

5736 Corporate Avenue • Cypress, CA 90630 • (714) 220-2211 , FAX (714) 220-2122

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1.0 INTRODUCTION

1.1 GENERAL

This report presents the results of a comprehensive geotechnical investigation performed by Geotechnical Professionals Inc. (GPI) for the proposed athletic field upgrades at Lynwood High School in Lynwood, California. The site location is shown on the Site Location Map, Figure 1.

As requested, our scope of work was performed in general compliance with the California Geologic Survey (CGS) Note 48 for geologic seismic hazard requirements. Our approved scope of work did not include site-specific ground motion hazard analysis. We are providing an updated geologic-hazard assessment that was originally prepared for a prior project at the subject school (Project No. 3070.I; dated November 12, 2021).

1.2 PROJECT DESCRIPTION

We understand the Lynwood Unified School District (LUSD) is planning on upgrades to the athletic fields. The upgrades will include a new, single-story storage/locker room, sixteen +/- 80-foot-tall Musco light poles, new synthetic multi-purpose field, new synthetic softball and baseball fields with associated fencing, bullpens, dugouts, a batting cage structure, permanent bleachers, a new rubberized field event area, a sand volleyball court, and new hardscape and paving. The existing track and field will also be replaced with a new synthetic track and field and new scoreboards. A new entrance arch/gate structure is also planned in the walkway to the east of the parking structure and running track. Other improvements will include underground utilities and associated infrastructure.

The existing site conditions are shown on the Aerial Site Plan, Figure 2. The proposed site conditions are shown on the Proposed Site Plan, Figure 3.

Structural loads (vertical or lateral) for the new storage/locker room, Musco light poles, scoreboards, goal posts, or other minor structures were not yet available at the time this report was prepared. Based on our experience with similar projects, the static loads for the planned storage/locker room building are anticipated to be relatively light (less than 50 kips for columns and 3 kips per lineal foot for walls). Proposed grades are not anticipated to change significantly from the existing grades. Site grading is anticipated to include cuts and fills of less than 2 feet. The new structures will be designed in accordance with the requirements of the 2022 CBC.

Infiltration of stormwater is planned as part of the renovation project but details regarding the location and type of infiltration structures have not been finalized.

Our recommendations are based upon the above-assumed structural and finish grade information. We should be notified if the actual loads and/or grades differ or change during the project design to allow our office to either confirm or modify our recommendations.

Also, when the project grading and foundation plans become available, we should be provided with a copy for review and comment.

1.3 PURPOSE OF INVESTIGATION

The primary purpose of this investigation and report is to provide an evaluation of the existing geotechnical conditions at the site as they relate to the design and construction of the proposed development.

1.4 PREVIOUS SITE WORK

GPI previously performed multiple geotechnical investigations at the subject campus as detailed below:

- Solar power shade/carport structures located in the existing parking lot at the northern end of the campus and along Imperial Highway (GPI Project No. 2526.09I, report dated February 6, 2013)
- Distressed pavement replacement throughout the campus (GPI Project No. 2705.I, report dated December 8, 2015).
- Replacement of Classroom Building G and construction of a new kiln building (GPI Project No. 3070.C, report dated November 12, 2021).
- A new stormwater infiltration system along the south side of the baseball field in the southwest corner of the overall campus (GPI Project No. 3226.I, report dated March 8, 2024).

GPI is also currently providing construction-phase geotechnical observation and testing services during construction of the Building G replacement and new kiln building directly east of the subject athletic fields.

The geotechnical data (i.e., explorations and laboratory testing) collected from our prior site investigations were referenced when preparing this report. Due to the proximity to the currently proposed athletic field improvements, we are incorporating the exploration boring log and laboratory test results from GPI Project No. 3226.I in this report. The remaining prior subsurface information and laboratory testing have not been included in this report but are available in the referenced documents.

2.0 SCOPE OF WORK

Our scope of work included a review of prior investigations at the campus, field investigation, laboratory testing, update to prior geologic and seismic evaluation, engineering analyses, and preparation of this report.

Our recent field investigations consisted of four Cone Penetration Tests (CPT's), CPT's C-5 through C-8, and eight exploratory borings, Borings B-7 through B-14. The CPT's were performed to approximate depths of 50 feet below existing grades. The borings were performed to approximate depths of 6 to 51 feet below the existing grades. Descriptions of field procedures and logs of the CPT's and borings are presented in Appendices A and B, respectively.

As previously noted, we are also presenting select prior explorations where relevant to the currently planned improvements. For our prior 2024 investigation in the athletic fields (GPI Project No. 3226.I), we performed one boring to an approximate depth of 55½ feet below existing site grades. The log of this prior exploration is also included within Appendix B.

Our laboratory testing program included evaluations of in-place moisture content, Atterberg limits, fines content, shear strength (direct shear), consolidation, expansion index, maximum dry density and optimum moisture content, and corrosivity. Laboratory test procedures and results are presented in Appendix C.

Soil corrosivity testing was performed by Project X Corrosion Engineers (Project X) under subcontract to GPI. Their test results are presented at the end of Appendix C.

An evaluation of geologic and seismic hazards is presented in Appendix D.

Engineering evaluations were performed to provide earthwork criteria and foundation design parameters. The results of our evaluations are presented in the remainder of the report.

3.0 SITE CONDITIONS

3.1 SURFACE CONDITIONS

The overall school site is bounded by Imperial Highway to the north, Harris Avenue to the east, and residential properties to the south and west. The campus is roughly 31 acres in size, with the campus buildings generally concentrated on the eastern half of the campus and the athletic fields on the western half of the campus. The proposed improvements are generally located in the southwestern portion of the site, within the approximate limits of the existing football, baseball, and softball fields, and the running track.

The topography of the site and the surrounding area is relatively flat (grades of less than 1 percent). Ground surface elevations within the athletic fields, based on internet sources (i.e., Google Earth), ranges from a low of approximately +85 feet in the southeast corner of the existing baseball field to a high of approximately +90 feet within the existing football field and track. Pavement sections at our recent and prior borings located in pavement areas consisted of 3 inches of asphalt concrete over 5 inches of aggregate base at Boring B-7 and 7 inches of portland cement concrete pavement over 5 inches of aggregate base at Boring B-13. Our remaining explorations were located in unpaved areas.

We reviewed historical aerial photographs of the site dating back to 1952. Between 1952 and 1994, the site appears to have been occupied by a previous configuration of a school and a portion of the existing site was occupied by Lynwood Adventist Academy. Between 1994 and 1998, the older campuses were demolished, and the site was redeveloped to the existing Lynwood High School configuration. Since 1998, the site appears to have remained unchanged.

3.2 SUBSURFACE SOIL CONDITIONS

Our recent and prior field investigations disclosed a subsurface profile consisting of undocumented fill soils over natural soils. Detailed descriptions of the conditions encountered in our explorations are shown on the Log of the CPT's and Borings in Appendices A and B, respectively.

Undocumented fill soils, generally to depths of 5 to 6 feet, were encountered in our recent borings. The fill soils at the boring locations consisted predominantly of slightly moist to moist, dense to very dense silty sands and very stiff to hard sandy silts. The fill soils are considered undocumented. Based on recent and prior laboratory testing, the upper fill soils exhibited a very low to low potential for expansion (Expansion Indices, EIs, of 8 and 30). For reference, an EI of between 20 and 50 is considered to be low. Within the upper 7 feet, the moisture contents of the fill and natural soils ranged from approximately 4 to 23 percent, with an average moisture content of about 11 percent. The optimum moisture content of the upper soils based on recent and prior laboratory testing ranged from approximately 10 to 15 percent.

The underlying natural materials consisted predominantly of interbedded layers of medium dense to dense silty sands and stiff to very stiff sandy silts, silts, and clays in the

upper 30 feet. In general, the soils encountered below depths of 30 feet consisted predominantly of medium dense to dense silty sands with lesser amounts of stiff to very stiff clays and silts.

The natural soils encountered in the upper 25 feet of our recent explorations were found to be generally moist to wet, with higher moisture contents generally encountered within the fine-grained soils. Moisture contents within the upper 25 feet ranged from a low of about 3 percent to a high of about 44 percent, with an average moisture content of about 16 percent. Below depths of 25 feet, the natural soils were generally moist to wet, with moisture contents generally ranging from 8 to 31 percent.

In general, the natural soils generally exhibited moderate to high strength and low compressibility characteristics.

The subsurface conditions in the explorations performed for this study are comparable to the subsurface conditions encountered in prior explorations by GPI for the adjacent new classroom building project on the school campus. The field exploration program by GPI for the adjacent new classroom building included borings, CPTs, and shear wave velocity testing (GPI, 2021).

The site is not located in within 300 feet of an oil or gas well, or within 1,000 feet of a methane producing site, as designated by the County of Los Angeles (SWIMS).

3.3 GROUNDWATER AND CAVING

Groundwater was encountered at an approximate depth of 44 feet in one of our recent borings at the site (Boring B-10). We also obtained piezometric measurements in our recent CPT C-8 at an approximate depth of 50 feet below existing site grades and measured a stabilized pressure of approximately 2.2 pounds per square inch (psi). This corresponds to a stabilized groundwater depth of approximately 45 feet below existing site grades.

Historical data provided by the California Geologic Survey (CGS) indicates the shallowest depth to groundwater is approximately 8 feet in the vicinity of the site.

Caving was not observed in our relatively small diameter borings as they were drilled with hollow stem auger equipment, which is essentially a cased boring excavation.

3.4 GEOLOGIC - SEISMIC HAZARDS

A detailed evaluation of the geologic conditions at the site, including seismic hazards, is presented in Appendix D. Ground motion and seismic settlement is addressed in a following section of this report.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 OVERVIEW

Based on the results of our investigation, it is our opinion that from a geotechnical viewpoint it is feasible to develop the site as proposed. Due to the potential for excessive seismic-induced settlement, the proposed storage/locker room is recommended to be supported on a shallow mat foundation following remedial grading. The proposed Musco light poles are recommended to be supported on conventional cast-in-place concrete piles (drilled piers) as planned. Minor, at-grade structures that may be less sensitive to the effects of liquefaction-induced settlement (e.g., scoreboards, dugout walls, shorter light poles, the arch/gate entrance structure, netting poles, fencing, and bleachers) may be supported on shallow spread or continuous footings or mat foundations following remedial grading or on drilled pier foundations. The most significant geotechnical issues that will affect the design and construction of the proposed structures are as follows:

- The existing undocumented fill soils encountered to depths of 5 to 6 feet in our borings are not considered to be suitable for direct uniform support of shallow foundations and floor slabs in their current state. To provide uniform support for the planned shallow foundations, we recommend that these materials be removed and replaced as properly compacted fill.

- The upper natural soils are generally loose to medium dense and firm to stiff. To provide uniform support for the proposed shallow foundations and floor slabs, we recommend that the upper portion of the natural materials be removed and replaced as properly compacted fill.

- The site is located in a seismic hazard zone for soil liquefaction. Groundwater was encountered as shallow as 44 feet in our recent explorations. Historical data published by the State indicated a shallowest groundwater measured in the site vicinity is approximately 8 feet below grades. As required by the Division of the State Architect (DSA) and the California Building Code (CBC), a groundwater depth of 8 feet was used in our liquefaction analyses.

- Based on our analyses, we computed a potential total seismic-induced liquefaction settlement of 2 to 3 inches. Differential seismic settlement is estimated to be between 1 and 1½ -inch across a span of 30 feet.

- In evaluating feasible foundation types per ASCE 7-16 criteria, we noted that the estimate differential settlement over a 30-foot span exceeds ¼ of the threshold differential settlement values provided in Table 12.13-3 of ASCE 7-16. As such, the design of structures supported on shallow foundations should be designed to account for differential settlements.

- The foundation system selected will depend on the tolerable total and differential settlements. Based on the structural loads assumed, the

estimated combined static and seismic settlements exceed the generally accepted tolerable limits (typically 1½ inches of total settlement and ¾-inch differential settlement) to support the building on shallow isolated and continuous spread footings. The estimated combined static and seismic settlements are within the typical tolerable settlement criteria for mat foundations or interconnected grade beams (typically 4 inches of total settlement and 2 inches differential settlement). Based on our conversations with the District and Design Team, we understand that a mat foundation is the likely preferred foundation type relative to interconnected grade beams.

- Based on the above, the proposed storage/locker room building may be supported on a mat foundation following remedial grading of the undocumented fill and upper native soils. Other settlement-sensitive structure should also be supported on mat foundations or drilled piers.

- Conventional cast-in-place concrete piles (e.g., drilled piers) may be used to support light standards and similar pole structures. The effects of downdrag do not need to be accounted for in the design of such single pile supported structures.

- Minor structures that may be less sensitive to potential liquefaction-induced settlement, such as scoreboards, dugout walls, shorter light poles, the arch/gate entrance structure, netting poles, fencing, and bleachers may be supported on shallow spread footings or small diameter shallow drilled piers (pole foundations).

- Moisture contents of the near surface soils (within 7 feet of the existing grades) are variable, ranging between roughly 4 and 23 percent. The average moisture content of the samples obtained within the upper 7 feet is approximately 11 percent, which is slightly below the optimum moisture content for the near surface soils. Therefore, mixing and moisture conditioning of the onsite soils will be required prior to being placed as properly compacted fill. Over-optimum subgrade soils exposed during grading may require stabilization or removal (where the wet soil layer is thin) in order to support compaction equipment. Stabilization may be accomplished using crushed aggregate base and geogrid or in-place cement treatment.

- Corrosivity testing performed by Project X on samples provided from our recent explorations indicates a negligible level of soluble sulfate content with respect to concrete. The soils are also considered to be corrosive to severely corrosive to buried ferrous metals.

Our recommendations related to the geotechnical aspects of the development of the site are presented in the subsequent sections of this report.

4.2 SEISMIC DESIGN

4.2.1 General

Details of our site-specific geologic and seismic evaluation for the site are presented in Appendix D.

We understand the seismic design of the proposed development will be in accordance with the California Building Code (CBC), 2022 edition, as well as ASCE 7-16 (ASCE, 2017). During our prior investigation for the adjacent new classroom building at the subject campus (GPI, 2021), we obtained direct shear-wave velocity measurements in a CPT within the upper 100 feet of the subsurface profile. Analysis of the prior data indicated a "weighted average" shear-wave velocity within the upper 100 feet of the subject campus, VS30, of 806 feet per second (about 246 meters per second). In accordance with Chapter 20 of ASCE 7-16, this shear-wave velocity corresponds to a Site Class D (Stiff Soil).

For the 2022 CBC and ASCE 7-16, a Site Class D may be used. The seismic code values can be obtained directly from the tables in the building code using the above Site Class value and pertinent internet website (https://seismicmaps.org/). As mapped, the ASCE 7-16 seismic code values are as follows:

S_S = 1.709g	$S_{MS} = F_a * S_S$ = 1.709g	$S_{DS} = 2/3 * S_{MS}$ = 1.139g
S_1 = 0.610g	$S_{M1} = F_V * S_1$ = 1.037g	$S_{D1} = 2/3 * S_{M1}$ = 0.691g

4.2.2 Strong Ground Motion Potential

Based on published information, the most significant faults in the proximity of the site are the Puente Hills (Los Angeles Segment) and Newport-Inglewood Faults, which are located about 2.0 and 4.9 miles from the subject site, respectively.

During the life of the project, the site will likely be subject to strong ground motions due to earthquakes on nearby faults. Based on the OSHPD website (https://seismicmaps.org/), we computed that the site could be subjected to a peak ground acceleration (PGA_M) of 0.80g for a magnitude 7.3 earthquake (Newport Inglewood Fault).

This acceleration has been computed using the mapped Maximum Considered Geometric Mean peak ground acceleration from ASCE 7-16 (ASCE, 2017) and a site coefficient (F_{PGA}) based on Site Class. The predominant earthquake magnitude was determined using a 2-percent probability of exceedance in a 50-year period, or an average return period of 2,475 years. The structural design will need to incorporate measures to mitigate the effects of strong ground motion.

4.2.3 Potential for Ground Rupture

There are no known active faults crossing or projecting through the site. The site is not located in an Alquist-Priolo Earthquake Fault Zone. Therefore, ground rupture due to faulting is considered unlikely at this site.

4.2.4 Liquefaction and Lateral Spreading

Liquefaction is a phenomenon in which saturated cohesionless soils undergo a temporary loss of strength during severe ground shaking and acquire a degree of mobility sufficient to permit ground deformation. In extreme cases, the soil particles can become suspended in groundwater, resulting in the soil deposit becoming mobile and fluid-like. Liquefaction is generally considered to occur primarily in loose to medium dense deposits of saturated sandy soils. Thus, three conditions are required for liquefaction to occur: (1) a sandy soil of loose to medium density; (2) saturated conditions; and (3) rapid, large strain, cyclic loading, normally provided by earthquake motions.

The site is located in a Seismic Hazard Zone for liquefaction, as mapped by the State of California (South Gate Quadrangle). Groundwater was encountered as shallow as 44 feet below existing grades in our recent explorations. Historical high groundwater levels provided by the California Geologic Survey indicate a shallowest groundwater level of approximately 8 feet below existing grades in the general site vicinity (CGS, 1999). As required by DSA and the CBC, we assumed a groundwater depth of 8 feet in our liquefaction evaluation.

In accordance with the 2022 CBC, ASEC 7-16, Special Publication 117A (CGS, 2008), and the 2022 revision of CGS Note 48, the ground motion used for this evaluation should be based on the Peak Ground Acceleration (PGA_M), adjusted for Site Class effects, or a site-specific response spectrum. As such, we considered a ground acceleration of 0.80g for a magnitude 7.3 earthquake for our analyses, which was determined based on the mapped values previously detailed.

The potential for liquefaction was evaluated using the methods presented by the NCEER and updated by Robertson (Robertson, 2009) and modifications provided in Special Publication 117A. Criterion for liquefaction susceptibility of the fine-grained soils was based on methods presented in Bray and Sancio (2006).

The materials encountered below the historical high groundwater level generally consisted of alternating layers of firm to very stiff fine-grained soils (silts and clays) and loose to dense coarse-grained soils (sands and silty sands). Overall, the soils encountered exhibited moderate strength.

Per the requirements of SP 117, liquefaction analyses are typically limited to depths of 50 feet below the structural foundation. Based on our analyses, and assuming a potential groundwater depth of 8 feet, we computed potential total seismic-induced liquefaction settlements of 2 to 3-inches. Differential seismic settlements (across a 30-foot span) are estimated to be between 1 and 1½-inch. Approximately half of the estimated seismic-induced liquefaction settlement was found to occur within the upper 20 feet of the subsurface profile. The above settlements, should they occur, would be expected to affect the overall athletic field and campus area and not be limited to a single structure or improvement item.

The maximum total and differential combined settlements (static plus seismic) that are generally tolerable for conventional isolated and continuous spread foundations are 1½

inches and ¾ inches, respectively. The maximum total and differential combined settlements (static plus seismic) that are generally tolerable for a mat foundation are 4 inches and 2 inches, respectively. We also noted that the estimate differential settlement over a 30-foot span exceeds ¼ of the threshold differential settlement values provided in Table 12.13-3 of ASCE 7-16. Accordingly, for structures supported on shallow foundations, the shallow foundations should consist of a mat foundation or interconnected (i.e., tied) grade beams. Based on our conversations with the District and Design Team, we understand that a mat foundation is the likely preferred foundation type relative to interconnected grade beams.

The results of our liquefaction analyses are presented in Appendix E.

Because the site is in a relatively level area, the potential for lateral spreading to occur during liquefaction is considered to be remote.

4.2.5 Seismic Ground Subsidence

Seismic ground subsidence or dry sand settlement (not related to liquefaction induced settlements) occurs when strong earthquake shaking results in densification of loose to medium dense sandy soils above groundwater. For the condition considering the upper soils are overexcavated and replaced as properly compacted fill and using the historical high depth to groundwater that was used in our liquefaction analysis (8 feet), the potential for dry sand seismic settlement to adversely affect the site is considered to be negligible.

Under current groundwater conditions, where groundwater was encountered as shallow as 44 feet, the total seismic ground subsidence was found to be highly variable. For our CPT C-5, total seismic ground subsidence is estimated to be on the order of 2 to 2½ inches. For our remaining CPTs, seismic ground subsidence is estimated to be on the order of ½ inch or less. Following the recommend earthwork, total seismic ground subsidence for C-5 is estimate to be approximately 1 to 1½ inches and differential seismic ground subsidence (across a 30-foot span) is estimated to be on the order of ¼ to ¾ inches. Differential seismic ground subsidence is estimated to be less than ¼ inch at the other CPT locations. These above total and differential seismic ground subsidence values are based on groundwater levels of approximately 44 feet below existing grades (the existing groundwater levels at the site) and are in lieu of the estimated liquefaction settlements previously presented.

4.3 EARTHWORK

The earthwork at the project site is anticipated to consist of clearing and grubbing, removals of undocumented fill and loose/disturbed soils, subgrade preparation, placement and compaction of fill, and foundation excavations.

4.3.1 Clearing

Prior to grading, performing excavations, or constructing the proposed improvements, the areas to be developed should be cleared of debris, vegetation, existing utilities, structures, and pavements. Buried obstructions, such as footings, abandoned utilities,

and tree roots should be removed from areas to be developed. Deleterious material generated during the clearing operation, including organic topsoil or material within the existing undocumented fill, should be removed from the site. If approved by the District, inert demolition debris, such as concrete, asphalt, and brick may be crushed for reuse in engineered fills outside the planned building areas in accordance with the criteria presented in the "Materials for Fill" section of this report. It is our experience that such material will be required to be exported from the site.

If cesspools or septic systems are encountered during grading, they should be removed in their entirety. The resulting excavation should be backfilled as recommended in the "Subgrade Preparation" and "Placement and Compaction of Fill" sections of this report.

As an alternative, cesspools can be backfilled with lean sand-cement slurry. At the conclusion of the clearing operations, a representative of the personnel from GPI should observe and accept the site prior to further grading.

4.3.2 Excavations

Excavations at this site will include removals of undocumented fill soils, soils disturbed during demolition and portions of the upper native soils, foundation excavations, and trenching for new utility lines. Excavations will also be dependent on whether the proposed improvements will be supported on shallow foundations (i.e., mat foundations or spread footings) or deep foundations (i.e., drilled, cast-in-place concrete piles).

Prior to placing fills or construction of the shallow-foundation supported structures or pavements and hardscape, the existing undocumented fill, loose soils disturbed during demolition, and portions of the upper natural soils should be removed and replaced as properly compacted fill. For planning purposes, we anticipate average removal depths as follows:

- For the proposed storage/locker room building or other structures supported on shallow foundations, removals should extend to depths of 7 feet below existing grades or 3 feet below the base of foundations, whichever is deeper.
- For minor structures (e.g., site walls, trash enclosures), removals should extend to 4 feet below grade or 2 feet below the base of foundations, whichever is deeper.
- For new pavements and hardscape, removals should extend to depths of at least 1-foot below the existing or proposed subgrade, whichever is deeper.

Existing grades refer to the grades at our exploration locations. Deeper removals may be required where deep disturbed soils are encountered. The actual depths of removals should be determined in the field during grading by a representative of GPI.

For deep foundations (i.e., drilled, cast-in-place concrete piles), removals of the undocumented fills and low-density natural soils are not considered necessary. We do recommend that the disturbed soils near the existing ground surface be densified to result in properly compacted fill. For planning purposes, we recommend the densification of at

least the upper 1-foot of the existing soil profile adjacent to deep foundations. This can be accomplished by scarification, moisture conditioning as necessary, and recompaction.

The Project Surveyor should accurately stake the corners of the areas to be overexcavated in the field. Where space is available, the base of the excavations should extend laterally a minimum distance equal to the depth of overexcavation/compaction below the base of foundations (i.e., a 1:1 projection projected down and outward from the lower outside edge of the foundation to the outside edge of the base or removal). Building lines include the footprint of the building and other foundation supported improvements, such as canopies and attached site walls.

In general, the upper fill soils are not considered to be susceptible to significant caving in shallow excavations (less than 4 feet). Temporary construction excavations may be made vertically without shoring to a depth of 4 feet below the adjacent grade. For deeper cuts up to 10 feet, the slopes should be properly shored or sloped back to at least 1:1 or flatter.

For cuts deeper than 10 feet but not exceeding 15 feet, slopes should be properly shored or sloped back to at least 1½:1 (horizontal:vertical) or flatter. Some raveling of the sandy deposits should be anticipated at the slope inclinations recommended. If raveling cannot be tolerated, flatter slope inclinations should be considered. The exposed slope face should be kept moist (but not saturated) during construction to reduce local sloughing.

In areas where removals are performed adjacent to existing structures or other areas where temporary slopes are not feasible, temporary shoring or “ABC” slot cuts should be utilized. The slots should be no wider than 6 feet and no deeper than 8 feet and should be backfilled immediately. If desired, we can evaluate the potential for deeper slot cuts as necessary to achieve the required excavations. The slots should be backfilled immediately to finished grade prior to excavation of the adjacent slots. If the slots are performed adjacent to an existing building that has perimeter pad footings in addition to a continuous footing, the slots should be aligned so that not more than one-half of the pad footing is exposed at a time. We should review the plans for excavation adjacent to existing buildings when they are developed in order to review and update the slot cut recommendations accordingly.

Surcharge loads should not be permitted within a horizontal distance equal to the height of cut from the top of the excavation or 5 feet from the top of the slopes, whichever is greater, unless the cut is properly shored. Excavations that extend below an imaginary plane inclined at 45 degrees below the edge of the adjacent existing site facilities should be properly shored to maintain support of adjacent elements. Excavations and shoring systems should meet the minimum requirements given in the State of California Occupational Safety and Health Standards.

In general, the excavation can be accomplished by conventional soil excavation equipment such as backhoes, loaders, scrapers, or dozers. However, rubber tire earthwork equipment should not be used where the exposed subgrade soils have moisture contents well above optimum because of an increased potential for “pumping” and subgrade disturbance.

4.3.3 Subgrade Preparation

Prior to placing fills, the subgrade soils at the bottom of overexcavations should be scarified to a depth of 12 inches, moisture conditioned as necessary, and compacted to at least 90 percent of the maximum dry density determined in accordance with ASTM D1557. This recommendation also pertains to the subgrade areas of asphalt pavement and hardscape. Scarification may be omitted based on evaluation by a representative of GPI if very moist to wet soils are exposed.

During our investigation, soils with variable moisture contents (ranging from 4 percent to a local max of 23 percent) were encountered within the upper 7 feet. The earthwork subcontractors should review the moisture content information presented on the boring logs, as wet soils may be encountered that will require mixing, drying, deeper removals, or stabilization prior to compaction. Also, heavy rubber-tired equipment is likely to cause pumping or yielding of wet subgrade. We recommend that the earthwork be performed outside of wet-weather seasons (generally October 15 through March 15) if possible.

If wet soils are encountered or if the exposed soils become wet from seasonal rains, the wet, yielding soils can be removed to expose a stable subgrade or subgrade stabilization may be required to support compaction equipment. For planning purposes, the stabilization would require the placement of 12 to 18 inches of crushed aggregate base (CAB) over a geogrid, such as Tensar BX1100. A thicker section of CAB could be used if the geogrid is omitted. As an alternative, the wet soils can be cement treated. For planning purposes, we anticipate stabilization can be achieved by mixing 4 percent cement within the upper 15 inches of exposed soil by unit weight (assume 120 pounds per cubic foot).

The cement treatment should be performed by a subcontractor experienced with the process, using equipment that can thoroughly mix the soil-cement prior to compaction.

4.3.4 Material for Fill

Soils available from on-site excavations, less debris or organic matter, will be suitable for re-use in compacted fills. Soils placed behind retaining walls and within 1-foot of the finished subgrade for building floor slabs and hardscape should be predominately granular (containing no more than 40 percent fines - portion passing No. 200 sieve) and non-expansive (E.I. of 20 or less). Such materials are anticipated to be available in sufficient quantities within the upper 7 feet below existing grades but may require selective grading.

Imported fill material should be predominately well-graded, granular, and non-expansive as defined above. Import or on-site materials used in compacted fills should not contain particles larger than 6 inches in diameter. GPI should be provided with a sample (at least 50 pounds) and notified of the location of soils proposed for import at least 72 hours in advance of importing. Each proposed import source should be sampled, tested, and accepted for use prior to delivery of the soils to the site. Soils imported prior to acceptance by GPI may be rejected if not suitable. If the District has environmental requirements for import material, they should also be followed.

If open-graded gravel is used as backfill, such as for stormwater infiltration or retention systems, the material should be separated from the adjacent soils with a suitable non-woven filter fabric, such as Mirafi 140N.

If the District approves of inert demolition debris, such as concrete, asphalt, and brick being crushed for reuse in engineered fills outside the planned building areas, the materials should meet the following processing requirements:

- If the inert debris is crushed to a well-graded mixture with maximum particle sizes of 1½ inches, the crushed material may be used directly in the fill without further blending.
- Inert debris up to a maximum size of 3 inches may also be used in fills, provided it is thoroughly blended with enough soils to form a well-grade mixture (typically a 4:1 soil to debris ratio).

4.3.5 Placement and Compaction of Fills

Fill soils should be placed in horizontal lifts, moisture-conditioned, and mechanically compacted to densities equal to at least 90 percent of the maximum dry density, determined in accordance with ASTM D1557. The optimum lift thickness will depend on the compaction equipment used and can best be determined in the field. The following uncompacted lift thickness can be used as preliminary guidelines.

Plate compactors	4-6 inches
Small vibratory or static rollers (5-ton±) or track equipment	6-8 inches
Scrapers, heavy loaders, or heavy vibratory rollers	8-12 inches

The maximum lift thickness should not be greater than 12 inches and each lift should be thoroughly compacted and accepted prior to subsequent lifts.

Fills should be placed at moisture contents of 0 to 2 percent over the optimum moisture content for granular soils and silts, and 1 to 3 percent over optimum for clays. The moisture content of the soils encountered in the upper 7 feet in the explorations was variable but, on average, slightly below optimum. As such, adequate mixing and some moisture conditioning may be necessary prior to replacing the soils as properly compacted fill. The properly compacted on-site soils should not be allowed to dry out prior to covering or additional moisture conditioning and processing will be required. The moisture content of the subgrade soils should be confirmed by GPI prior to covering.

During backfill of excavations, the fill should be properly benched into the construction slopes as it is placed in lifts.

4.3.6 Shrinkage and Subsidence

Shrinkage is the loss of soil volume caused by compaction of fills to a higher density than before grading. Subsidence is the settlement of in-place subgrade soils caused by loads generated by large earthmoving equipment. For earthwork volume estimating purposes, an average shrinkage value of 10 to 15 percent may be assumed for the surficial soils.

Higher values may be realized if deep undocumented fills are encountered. Subsidence is anticipated to be about 0.1 feet. These values are estimates only and exclude losses due to removal of vegetation or debris. Actual shrinkage and subsidence will depend on the types of earthmoving equipment used and should be verified during grading.

4.3.7 Trench/Wall Backfill

Utility trench and wall backfill, consisting of the on-site materials or imported sand, should be mechanically compacted in lifts. Clays and silts should not be used for retaining wall or wall-below-grade backfill or for direct support of building slabs or hardscape. Lift thickness should not exceed those values given in the "Placement and Compaction of Fill" section of this report. Moisture conditioning of the on-site soils will be required prior to re-use as backfill. Jetting or flooding of backfill materials should not be permitted. A representative of GPI should observe and test trench and wall backfill as they are placed.

In backfill areas where mechanical compaction of soil backfill is impractical due to space constraints, sand-cement slurry may be substituted for compacted backfill. The slurry should contain at least one sack of cement per cubic yard. Within building areas, the slurry should contain at least two sacks of cement per cubic yard. When set, such a mix typically has the consistency of compacted soil. Under foundations, concrete equal in strength to the foundation concrete should be used if fill is required.

4.3.8 Observation and Testing

A representative of GPI should observe excavations, subgrade preparation, and fill placement activities. Sufficient in-place field density tests should be performed during fill placement and in-place compaction to evaluate the overall compaction of the soils. Soils that do not meet minimum compaction requirements should be reworked and tested prior to placement of additional fill.

4.4 FOUNDATIONS

4.4.1 General

The foundation system selected will depend on the tolerable total and differential settlements. Based on the structural loads assumed, the estimated combined static and seismic (liquefaction induced) settlements exceed the generally accepted tolerable limits (typically 1½ inches of total settlement and ¾-inch differential settlement) to support the planned storage/locker room building on shallow isolated and continuous spread footings. The estimated combined static and seismic settlements are within the typical tolerable settlement criteria for mat foundations or interconnected grade beams (typically 4 inches of total settlement and 2 inches differential settlement). Based on the above:

- The proposed storage/locker room building may be supported on a mat foundation or interconnected grade beams following remedial grading of the undocumented fill and upper native soils as recommended in the Earthwork section of this report. Based on our conversations with the District and Design Team, we understand that

mat foundations are the preferred foundation type relative to interconnected grade beams. Other settlement-sensitive structures should also be supported on mat foundations or drilled piers.

- The proposed Musco light poles may be supported on cast-in-place concrete piles (drilled piers).

- Minor structures that may be less sensitive to potential liquefaction-induced settlement, such as scoreboards, dugout walls, shorter light poles, the arch/gate entrance structure, netting poles, fencing, and bleachers may be supported on shallow spread footings following remedial grading or small diameter shallow drilled piers (pole foundations).

4.4.2 Mat Foundation

Allowable Bearing Capacity

The allowable bearing pressure for a mat foundation is generally not the governing geotechnical issue as compared to the anticipated settlement. At this time, we have not been provided with estimated static mat foundation pressures for the proposed structure.

If a mat foundation is to be considered, we should be provided with a detailed plot of the anticipated bearing pressures to review.

For the elastic design of the mat foundation, a modulus of subgrade reaction (k-value) of 150 pounds per cubic inch (pci; pounds per square inch per inch of deflection) may be used. This value is for a 1-foot by 1-foot square loaded area and should be adjusted for the area of the mat foundation using appropriate elastic theory. Using generally accepted methods and our site-specific consolidation test results, we recommend using a value of 38 pci for the adjusted k-value in designing the mat foundation. As previously discussed, we should be provided with the anticipated mat pressures when they are developed so that we can review and confirm the recommendations provided, as well as provide an estimate for the anticipated maximum static settlements for the mat foundations.

The allowable soil bearing pressure will be significantly greater than the average bearing pressures required for the mat foundation as discussed above. At localized areas of the mat, such as columns and point of load applications along exterior walls, a static allowable net bearing pressure of 2,000 pounds per square foot may be used. These allowable bearing pressures are for dead-plus-live loads, and may be increased one-third for short-term, transient, wind and seismic loading.

Anticipated Settlement

Static loads for the planned storage/locker room building are anticipated to be relatively light (less than 50 kips for columns and 3 kips per lineal foot for walls). Under the anticipated lightly loaded static conditions for the storage/locker room and typical of minor structures (i.e., trash enclosures, site walls), maximum total static settlement of the proposed structures is expected to be on the order of ½ to ¾-inch. Maximum differential

static settlement between similarly loaded adjacent footings is estimated to be on the order of ¼ to ⅓-inch across a lateral distance of 30 feet.

As discussed earlier, we computed seismic (liquefaction-induced) settlements of 2 to 3 inches for the purpose of evaluating total foundation settlement. As such, combined total (static plus seismic) settlement is expected to be between 2½ to 3¾ inches. Differential static plus seismic settlements are expected to be about 1¼ to 1¾ inch over a distance of 30 feet.

Lateral Resistance

Soil resistance to lateral loads will be provided by a combination of frictional resistance between the bottom of foundations and underlying soils and by passive soil pressures acting against the embedded sides of the foundations. For frictional resistance, a coefficient of friction of 0.35 may be used for design. In addition, an allowable lateral bearing pressure equal to an equivalent fluid weight of 300 pounds per cubic foot may be used, provided the foundations are poured tight against the compacted fill. A one-third increase in the above allowable lateral bearing pressure (but not the frictional resistance) may be taken for short-term wind and seismic loads. The passive pressure provided also assumes a level ground surface extending to a horizontal distance from the wall or foundation at least twice the depth of embedment. These values may be used in combination without reduction.

The above coefficient of friction of 0.35 may be used whether the concrete mat foundation is supported on soils or on a moisture vapor barrier.

4.4.3 Spread Footings

Minor structures that may be less sensitive to potential liquefaction-induced settlement, may be supported on shallow spread footings following remedial of the undocumented fill and upper native soils as recommended in the Earthwork section of this report.

Allowable Bearing Capacity

Based on the shear strength and elastic settlement characteristics of the recompacted on-site soils, a static allowable net bearing pressure of up to 1,500 pounds per square foot (psf) may be used for both continuous footings and isolated column footings. The actual bearing pressure used may be less, such that economics and structural loads will determine the minimum width for footings as discussed below. These bearing pressures are for dead load-plus-live loads, and may be increased one-third for short-term, transient, wind and seismic loading. The maximum edge pressures induced by eccentric loading or overturning moments should not be allowed to exceed these recommended values.

A minimum footing width of 18 inches and depth of 18 inches should be used even if the actual bearing pressure is less than 1,500 psf.

Anticipated Settlement

Under the anticipated lightly loaded static conditions typical of minor structures (i.e., trash enclosures, site walls), maximum total static settlement of the proposed minor structures is expected to be on the order of ½ to ¾-inch. Maximum differential static settlement between similarly loaded adjacent footings is estimated to be on the order of ¼ to ⅓-inch across a lateral distance of 40 feet. As discussed earlier, we computed total seismic settlements of 2 to 3 inches for the purpose of evaluating total foundation settlement.

The above settlement estimates are based on the assumption that the recommended earthwork will be performed and that the footings will be sized in accordance with our recommendations.

Lateral Resistance

Soil resistance to lateral loads will be provided by a combination of frictional resistance between the bottom of footings and underlying soils and by passive soil pressures acting against the embedded sides of the footings. For frictional resistance, a coefficient of friction of 0.35 may be used for design. In addition, an allowable lateral bearing pressure equal to an equivalent fluid weight of 300 pounds per cubic foot may be used, provided the footings are poured tight against the compacted fill. A one-third increase in the above allowable lateral bearing pressure (but not the frictional resistance) may be taken for short-term wind and seismic loads. The passive pressure provided also assumes a level ground surface extending to a horizontal distance from the wall or footing face at least twice the depth of embedment. These values may be used in combination without reduction.

4.4.4 Light Standards and Poles

The Musco Light poles, new goal posts, scoreboard, and similar structures may be supported on drilled pier foundations. The design of such piers is typically governed by lateral loading conditions.

Lateral and Axial Capacity

Soil resistance to lateral loads can be provided by the piers. The design of the piers will be governed by lateral force considerations. For design by the simplified pole formula presented in Section 1807A.3.2.1 of the 2022 California Building Code, a unit passive resistance of 250 pounds per square foot per foot (to a maximum of 2,500 pounds per square foot) may be used for the piers with level ground in lieu of the presumptive lateral bearing values presented in Table 1806A.2. Based on our analyses, we recommend a unit passive resistance of 500 pounds per square foot per foot (to a maximum of 5,000 pounds per square foot) be used for isolated piles that can tolerate ½-inch of deflection under short-term loads.

Because the above structures will be supported on isolated piles and the calculated liquefaction settlements generally extend across the full anticipated pile depths, the

potential effects of down drag do not need to be accounted for in the design of the drilled pile foundations.

Due to the presence of undocumented fills across the subject athletic fields, we recommend the following depths of the subgrade soils be ignored in determining the required depth of embedment for lateral and/or axial capacity:

- The upper 5 feet of subgrade soils for the proposed scoreboard, Musco light poles, and other comparably large pole-supported structures.
- The upper 3 feet of subgrade soils for chain-link or comparable fencing greater than 10-feet in height.
- The upper 1 foot of subgrade soils for minor-pile supported structures such as chain-link-fencing less than 10 feet in height.

Where undocumented fills were removed in their entirety and replaced as properly compacted fill, we recommend that the upper 1 foot of the subgrade soils be ignored in determining the required depth of embedment for lateral and/or capacity to allow for future surface disturbance adjacent to the pile, regardless of structure type.

A drilled pier designed for adequate embedment to resist the anticipated lateral loads will likely have adequate axial capacity to support the anticipated vertical loads. The net allowable vertical compressive capacity can be conservatively calculated based on a unit side friction of 250 pounds per square foot, neglecting end-bearing contribution.

Constructability

Based on the density, moisture content, and fines content of the soils encountered, the caving potential of the soils is considered to be low to moderate. If caving soil conditions are encountered, casing, slurry, or other methods should be utilized to maintain the pile excavation. In addition, caving soils would increase the required volume of concrete.

Based on our explorations, we do not anticipate encountering groundwater during the installation of the piles. However, some seasonal groundwater seepage is possible if pile excavations are left open for extended periods.

Pile excavations should be filled with concrete on the same day they are drilled. Concrete mix designs should include provisions to minimize shrinkage, which can lead to lower frictional resistance of the pile shaft and reduced allowable capacity. The concrete should be placed with special equipment so that it is not allowed to fall freely more than 5 feet or strike the walls of the excavations. Drilling for piles should not be performed within 5 feet of recently excavated or recently poured piles until the concrete has been allowed to set for at least 6 hours. The piles should be poured in a manner that will not result in concrete flowing into adjacent drilled pile excavations and prevent segregation of aggregate. Drilled pile construction should be performed in accordance with the latest edition of ACI 336.1, “Standard Specifications for the Construction of Drilled Piles.”

Downhole inspection of drilled pile excavations (by lowering an inspector into the excavation) is not needed if the piles are designed as frictional elements. However, pile and footing excavations should be observed by a representative of GPI to confirm and document the depth, diameter, and embedment in suitable materials.

4.4.5 Foundation Concrete

Laboratory testing by Project X indicates that the on-site soils have a soluble sulfate contents of approximately 33 to 909 mg/kg (0.0033 to 0.0909 percent by weight). In accordance with the 2022 CBC, foundation concrete should conform to the requirements outlined to the requirements outlined in ACI 318, Section 19.3 for a negligible level of soluble sulfate exposure for soil (Category S0). Chloride contents of approximately 15 to 78 mg/kg (0.0015 to 0.0078 percent by weight) were measured, which are considered to be low (Category C1).

4.4.6 Foundation Observation

Prior to placement of steel and concrete, a representative of GPI should observe and approve foundation excavations. Footing excavations should be moistened immediately prior to concrete placement.

4.5 CONCRETE SLABS

Concrete slabs should be supported on relatively non-expansive (EI less than 20), compacted soils as discussed in the “Material for Fill” and “Placement and Compaction of Fill” sections. Suitable soils appear to be readily available on-site in sufficient quantities, but selective grading will be required. Clayey soils are not suitable for direct support of slabs and hardscape. The subgrade soils should not be allowed to dry out prior to concrete placement or additional processing and moisture conditioning will be required.

A moisture vapor retarder should be placed under slabs that are to be covered with moisture-sensitive floor coverings (parquet, vinyl, tile, etc.). Currently, common practice is to use a 10 or 15 mil polyolefin product such as Stego Wrap for this purpose. Whether to place the concrete slab directly on the vapor barrier or place a clean sand layer between the slab and vapor barrier is a decision for the Project Architect, as it is not a geotechnical issue. If covered by sand, the sand layer should be about 2 inches thick and contain less than 5 percent by weight passing the No. 200 sieve. Based on our explorations and laboratory testing, the soils at the site are not suitable for this purpose.

The function of the sand layer is to protect the vapor retarder during construction and to aid in the uniform curing of the concrete. This layer should be nominally compacted using light equipment. The sand placed over the vapor retarder should only be slightly moist. If the sand gets wet (for example as a result of rainfall or excessive moistening) it must be allowed to dry prior to placing concrete. Care should be taken to avoid infiltration of water into the sand layer after placement of the concrete slab, such as at slab cut-outs and other exposures. A sand layer is not required beneath the vapor retarder, but we take no exception if one is provided.

It should be noted that the material used as a vapor retarder is only one of several factors affecting the prevention of moisture accumulation under floor coverings. Other factors include maintaining a low water-cement ratio for the concrete used for the floor slab and effective sealing of joints and edges (particularly at pipe penetrations). The manufacturer of the floor coverings should be consulted for establishing acceptable criteria for the condition of the floor surface prior to placing moisture-sensitive floor coverings.

For the elastic design of slabs supporting sustained concentrated loads, a modulus of subgrade reaction (k) of 150 pounds per cubic inch (pounds per square inch per inch of deflection) may be used. This value is for a 1-foot by 1-foot square loaded area and should be adjusted by the structural designer for the area of the proposed building slab using appropriate elastic theory.

4.6 PAVED AREAS

Prior testing of the on-site upper soils at the school campus indicated an R-value of 53 (GPI Project No. 2705.I). Pavement design has been based on an assumed R-value of 40. R-value testing should be performed prior to construction of the pavement sections to confirm the preliminary design. The California Division of Highways Design Method was used for design of the recommended preliminary pavement sections. These recommendations are based on the assumption that the pavement subgrades will consist of existing near surface soils. The following pavement sections are recommended for various assumed Traffic Indexes (TIs).

PAVEMENT AREA	TRAFFIC INDEX	SECTION THICKNESS (inches)	
		ASPHALT/PORTLAND CONCRETE	AGGREGATE BASE COURSE
Asphalt Concrete			
Pedestrian (no vehicles)	---	2.0	3.0
Automobile Parking	4.0	3.0	4.0
Automobile Drives	5.0	3.0	6.0
Truck/Bus Drives	6.0	3.5	7.0
Heavy Truck/Bus Drives	7.0	4.0	8.0
Portland Cement Concrete			
Pedestrian (no vehicles)	---	4.0	4.0
Automobile Parking	4.0	5.5	4.0
Automobile Drives	5.0	6.0	4.0
Truck/Bus Drives	6.0	6.5	4.0
Heavy Truck/Bus Drives	7.0	7.0	4.0

For pavement design of fire lanes, assuming an anticipated usage of one round trip per month by a 68,000-pound fire truck, a pavement section corresponding to a traffic index (TI) of 5 is considered to be suitable. If the fire lane will also accommodate additional traffic consisting of trucks and buses, the pavement sections should correspond to either a TI of 6 (truck/bus drives) or 7 (heavy truck/bus drives).

The portland cement concrete used for paving should have a modulus of rupture of at least 550 psi (equivalent to an approximate compressive strength of 3,700 psi at the time the pavement is subjected to traffic). If the site is base paved prior to the start of building

construction, the above pavement sections should be re-evaluated based on the anticipated construction traffic loads.

The pavement subgrade underlying the aggregate base should be properly prepared and compacted in accordance with the recommendations outlined under "Subgrade Preparation".

The pavement base course should be compacted to at least 95 percent of the maximum dry density (ASTM D1557). Aggregate base should conform to the requirements of Section 200-2 of the Standard Specifications for Public Works Construction (Green Book) for crushed aggregate base (CAB) materials.

The above recommendations are based on the assumption that the base course and compacted subgrade will be properly drained. The design of paved areas should incorporate measures to prevent moisture build-up within the base course, which can otherwise lead to premature pavement failure. For example, curbing adjacent to landscaped areas should be deep enough to act as a barrier to infiltration of irrigation water into the adjacent base course.

4.7 RETAINING STRUCTURES AND SHORING

Based on information available to us at the time this report was prepared, significant retaining walls are not planned. The following recommendations are provided for cantilevered retaining walls up to 8 feet in height. We recommend that walls be properly drained and backfilled with sandy soils (less than 40 percent passing the No. 200 sieve).

The onsite silts and clays are not suitable for use as retaining wall backfill where conventional backfill is used.

Data provided by CGS indicates an approximate historical high groundwater level of 8 feet below existing grades. However, we encountered groundwater at an approximate shallowest depth of 44 feet in our recent explorations. Based on current groundwater management practices, the potential for groundwater to negatively impact the proposed development, including retaining walls or shoring, is considered to be negligible. We understand subterranean construction is not currently planned as part of the proposed improvements.

4.7.1 Retaining Walls

Active earth pressures can be used for designing walls that can yield at least ½-inch laterally per 10 feet of wall height under the imposed loads. For level, drained backfill, derived from non-expansive granular soils (EI $\leq$ 20), a lateral pressure of an equivalent fluid weighing of 38 pounds per cubic foot may be used. At-rest pressures should be used for restrained walls that remain rigid enough to be essentially non-yielding. For select, non-expansive, level, drained backfill, a lateral pressure of an equivalent fluid weighing 54 pounds per cubic foot can be used. If the wall backfill is not drained, the combined earth and water pressures could be much higher.

A seismic lateral pressure should be used for the design of retaining walls supporting more than 6 feet or more of backfill. We recommend a seismic lateral pressure of 38 pounds per cubic foot be added to the active earth pressure recommended above. If at-rest pressure is used to design the retaining wall, the total lateral pressure used (at-rest plus seismic) is not required to exceed the total active plus seismic pressure (76 pounds per cubic foot).

Walls subject to surcharge loads should be designed for an additional uniform lateral pressure equal to one-third and one-half the anticipated surcharge pressure for unrestrained and restrained walls, respectively. In addition to the recommended earth pressure, the upper 10 feet of the walls adjacent to the streets should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pounds per square foot surcharge behind the retaining walls due to normal street traffic. If traffic is kept at least 10 feet from the walls, the traffic surcharge may be neglected.

The recommended pressures assume that the supported earth will be fully drained, preventing the build-up of hydro-static pressures. For traditional backfilled retaining walls, a drain consisting of perforated pipe and gravel, wrapped in a suitable filter fabric should be used. As a minimum, one cubic foot of rock should be used for each lineal foot of drain. The fabric (non-woven filter fabric, Mirafi 140N or equivalent) should be lapped at the top.

4.7.2 Temporary Shoring

Where there is not sufficient space for sloped embankments, such as along the property limits or adjacent to existing structures, shoring will be required. One method of shoring would consist of steel soldier piles placed in drilled holes and backfilled with concrete.

Driven or vibrated soldier piles may also be more economical alternative to drilled holes, and they can be used for supporting cuts that do not support existing structures.

For cantilever shoring with level backfill, the magnitude of active pressure is equivalent to the pressures imposed by a fluid weighing 38 pounds per cubic foot (pcf). For sloping backfill with a 1:1 inclination, the active pressure would be 65 pcf. The active pressure should be applied to the full embedment depth of the cantilever shoring.

In addition to the recommended earth pressure, the shoring should be designed for surcharge loads due the adjacent structures and construction traffic surcharge loads. The upper 10 feet of the shoring adjacent to streets should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pound per square foot surcharge behind the shoring due to normal street traffic. If traffic is kept at least 10 feet from the shoring, the traffic surcharge may be neglected.

Existing adjacent structures will impart a surcharge load on shoring. The location and depth of the adjacent building footings, as well as the loading, will need to be determined to estimate the surcharge pressure on the shoring.

For design of soldier piles spaced at least two diameters on centers, the allowable lateral bearing value (passive value) of the soils below the excavation may be taken to be 550 pounds per square foot at the excavated surface, up to a maximum of 5,500 psf. To develop the full lateral value, provisions should be made to assure firm contact between the soldier piles and the undisturbed soils. The concrete placed in the soldier pile excavation below the excavated level may be a lean mix, but it should be of adequate strength to transfer the imposed loads to the surrounding soils.

The shoring contractor should evaluate the potential drilling conditions when planning the installation methods.

Driven or vibrated soldier piles may be a feasible and more economical alternative. If soldier piles are vibrated or driven, predrilling should not be allowed below the planned excavation level. Predrilling should be performed with a continuous flight auger capable of reversing the auger to minimize the removal of soil during the process. The diameter used for predrilling should not exceed 80 percent of the maximum depth of the soldier pile section. For design, the width of the driven or vibrated pile should be taken as the width of the flange.

The frictional resistance between the soldier piles and the retained earth may be used in resisting the downward component of the anchor load, if used. The coefficient of friction between the soldier pile and the retained earth may be taken as 0.35. This value is based on the assumption that uniform full bearing will be developed between the steel soldier beam and the lean-mix concrete and between the lean mix concrete and the retained earth. In addition, provided the portion of the soldier piles below the excavated level is backfilled with structural concrete, the soldier piles below the excavated level may be used to resist downward loads. The frictional resistance between the concrete soldier piles and the soils below the excavated level may be taken as equal to 400 pounds per square foot.

Continuous lagging will be required between the soldier piles. Careful installation of the lagging will be necessary to achieve bearing against the retained earth. We recommend that the voids between the lagging and retained earth be backfilled with a lean-mix sand-cement slurry prior to continuing the excavation deeper. The soldier piles should be designed for the full anticipated lateral pressure. However, the pressure on the lagging will be less because of arching of the soils between piles. We recommend that the lagging be designed for the recommended earth pressure but limited to a maximum value of 400 pounds per square foot, provided the soldier beam spacing is 8 feet or less.

It is difficult to accurately predict the amount of deflection of the shored embankment. It should be realized, however, that some deflection will occur. Adjacent to city right-of-way, the shoring should be designed to limit deflection to 1-inch. We recommend limiting the lateral deflection of shoring adjacent to structures to ½-inch. If greater deflection occurs during construction, additional bracing may be necessary. In areas where less deflection is desired, such as adjacent to existing settlement sensitive improvements, the shoring should be designed for higher lateral earth pressures.

We recommend performing a detailed survey of the improvements to be supported above the planned shoring prior to and during the shoring installation. The survey should include topographic data and a video account of the condition of the existing improvements, including cracks or signs of distress. During construction, the monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of the soldier piles.

We suggest weekly readings during the excavation and for the first three weeks after achieving the bottom of the excavation. After that time, the readings should be performed every other week until the completion of the retaining walls or basement walls (if planned).

4.8 EXTERIOR CONCRETE AND MASONRY FLATWORK

Exterior concrete and masonry flatwork should be supported on a zone of non-expansive compacted fill if differential heave is not tolerable. The use of clayey soils within 1-foot of concrete hardscape subgrade is not recommended. Prior to placement of concrete, the subgrade should be prepared as recommended in “Subgrade Preparation” section. The moisture content of subgrade soils should be maintained above the optimum moisture content and confirmed by a representative of GPI prior to covering. Subgrade soils allowed to dry out will require moisture conditioning, including the potential for additional processing.

We recommend the Project Structural or Civil Engineer design the concrete hardscape and sidewalks, including determination of thickness and reinforcing. For exterior flatwork, hardscape, and sidewalks, we recommend a minimum slab thickness of 4 inches with minimum slab reinforcement of No. 3 rebar placed at 16 inches on-center, in both directions. Control joints to direct shrinkage cracking in exterior slabs and sidewalks should be provided at maximum spacing of 8 and 6 feet on center in two directions, respectively. Where flatwork is planned at building entries, we recommend it be structurally connected to the perimeter building foundation to reduce the potential differential settlement at this joint. These recommendations should be considered as minimums based on the geotechnical site conditions, and the Project Civil Engineer should confirm if more stringent recommendations are needed for other purposes.

The concrete hardscape, stairs, flatwork, and sidewalks should be underlain by at least 4 inches of crushed aggregate base (CAB). Base course materials should be compacted to at least 95 percent of the maximum dry density (ASTM D1557). Aggregate base should conform to the requirements of Section 200-2 of the Standard Specifications for Public Works Construction (Green Book) for CAB materials.

4.9 CORROSION

Resistivity testing by Project X indicates that the on-site soils are corrosive to severely corrosive to buried ferrous metals. The results of the resistivity testing are presented in Appendix C. GPI does not practice corrosion engineering. We recommend that a corrosion engineering firm, such as Project X, be consulted if corrosion protection recommendations are required.

4.10 SURFACE DRAINAGE AND INFILTRATION

Positive surface gradients should be provided adjacent to structures so as to direct surface water run-off and roof drainage away from foundations and slabs toward suitable discharge facilities. Long-term ponding of surface water should not be allowed on pavements or adjacent to buildings.

Infiltration of stormwater is planned as part of the renovation project but details regarding the location and type of infiltrations structures have not been finalized. Infiltration testing was performed at one location near the southern property line as part of our prior 2024 investigation (GPI Project No. 3226.I). Additional field infiltration testing was not included in our recently approved scope of services. If stormwater infiltration is planned in other areas of the site, additional infiltration in the alternate areas should be performed at the appropriate locations and depths. Due to the potential for liquefaction to occur in the upper site soils, infiltration of ground water within 30 feet of the proposed building is not recommended.

4.11 GEOTECHNICAL OBSERVATION AND TESTING

We recommend that a representative of GPI observe the earthwork during construction to confirm that the recommendations provided in our report are applicable during construction. The earthwork activities include grading, compaction of fills, subgrade preparation, pavement construction and foundation excavations. If conditions are different than expected, we should be afforded the opportunity to provide an alternate recommendation based on the actual conditions encountered.

5.0 LIMITATIONS

This report, exploration logs, and other materials resulting from GPI's efforts were prepared exclusively for use by the Lynwood Unified School District and their consultants in designing the proposed development. The report is not intended to be suitable for reuse on extensions or modifications of the project or for use on project other than the currently proposed development as it may not contain sufficient or appropriate information for such uses.

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.

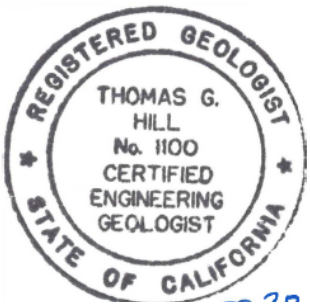
Furthermore, our recommendations were developed with the assumption that a proper level of field observation and construction review will be provided by GPI during grading, excavation, and foundation construction. If construction phase services are performed by others, they must accept full responsibility for geotechnical aspects of the project, including this report.

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,
Geotechnical Professionals Inc.

Dylan J. Boyle, G.E.
Associate

Justin J. Kempton, G.E.
Principal



Thomas G. Hill, C.E.G.
Consulting Geologist

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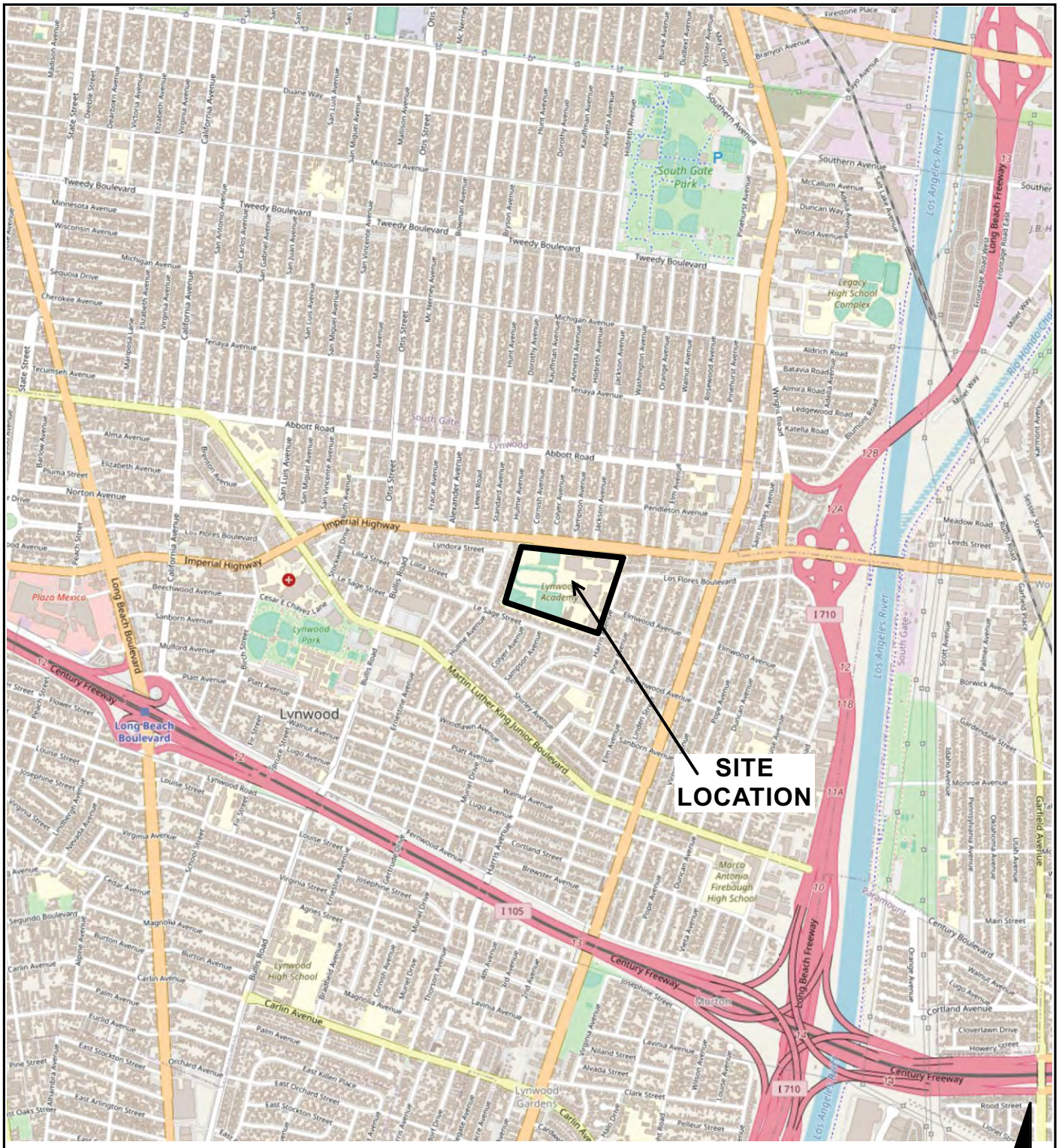


BASE MAP REPRODUCED FROM USGS 7.5' TOPO MAPS © CALTOPO

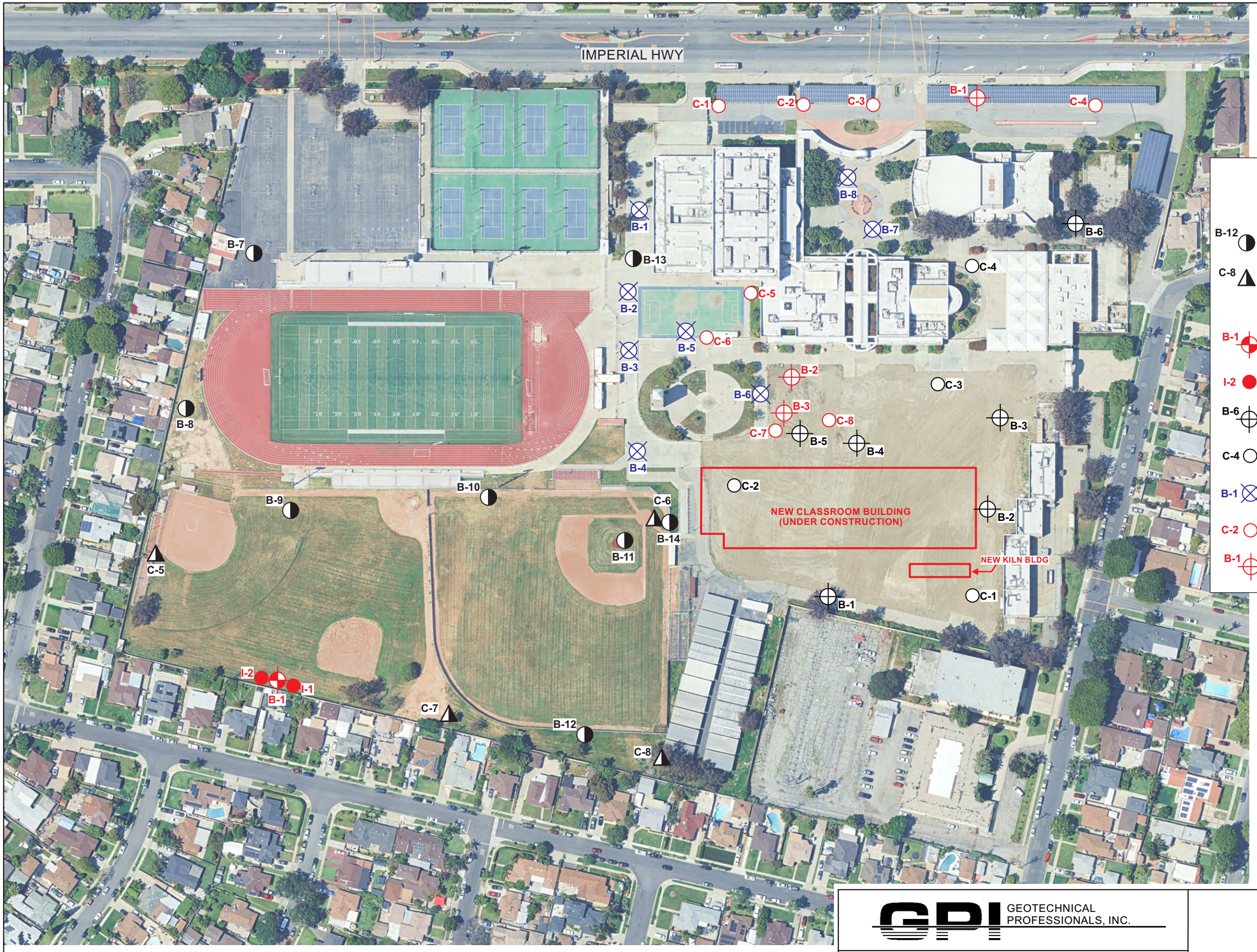
GPI GEOTECHNICAL PROFESSIONALS, INC.

LYNWOOD HIGH SCHOOL FIELD RENOVATIONS

GPI PROJECT NO.: 3328.I | SCALE: 1" = 2000'

SITE LOCATION MAP

FIGURE 1



EXPLANATION

CURRENT EXPLORATIONS

B-12 — **APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING (GPI PROJ. 3328.I)**

C-8 — **APPROXIMATE LOCATION AND NUMBER OF CONE PENETRATION TEST (GPI PROJ. 3328.I)**

PRIOR EXPLORATIONS

B-1 — APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING (GPI PROJ. 3226.I)

I-2 — APPROXIMATE LOCATION AND NUMBER OF INFILTRATION WELLS (GPI PROJ. 3226.I)

B-6 — APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING (GPI PROJ. 3070.I)

C-4 — APPROXIMATE LOCATION AND NUMBER OF CPT (GPI PROJ. 3070.I)

B-1 — APPROXIMATE LOCATION AND NUMBER OF PRIOR BORING (GPI PROJ. 2705.I)

C-2 — APPROXIMATE LOCATION AND NUMBER OF PRIOR CPT (GPI PROJ. 2526.09I)

B-1 — APPROXIMATE LOCATION AND NUMBER OF PRIOR BORING (GPI PROJ. 2526.09I)

0 150 300 FEET

BASE MAP REPRODUCED FROM GOOGLE EARTH © 2025

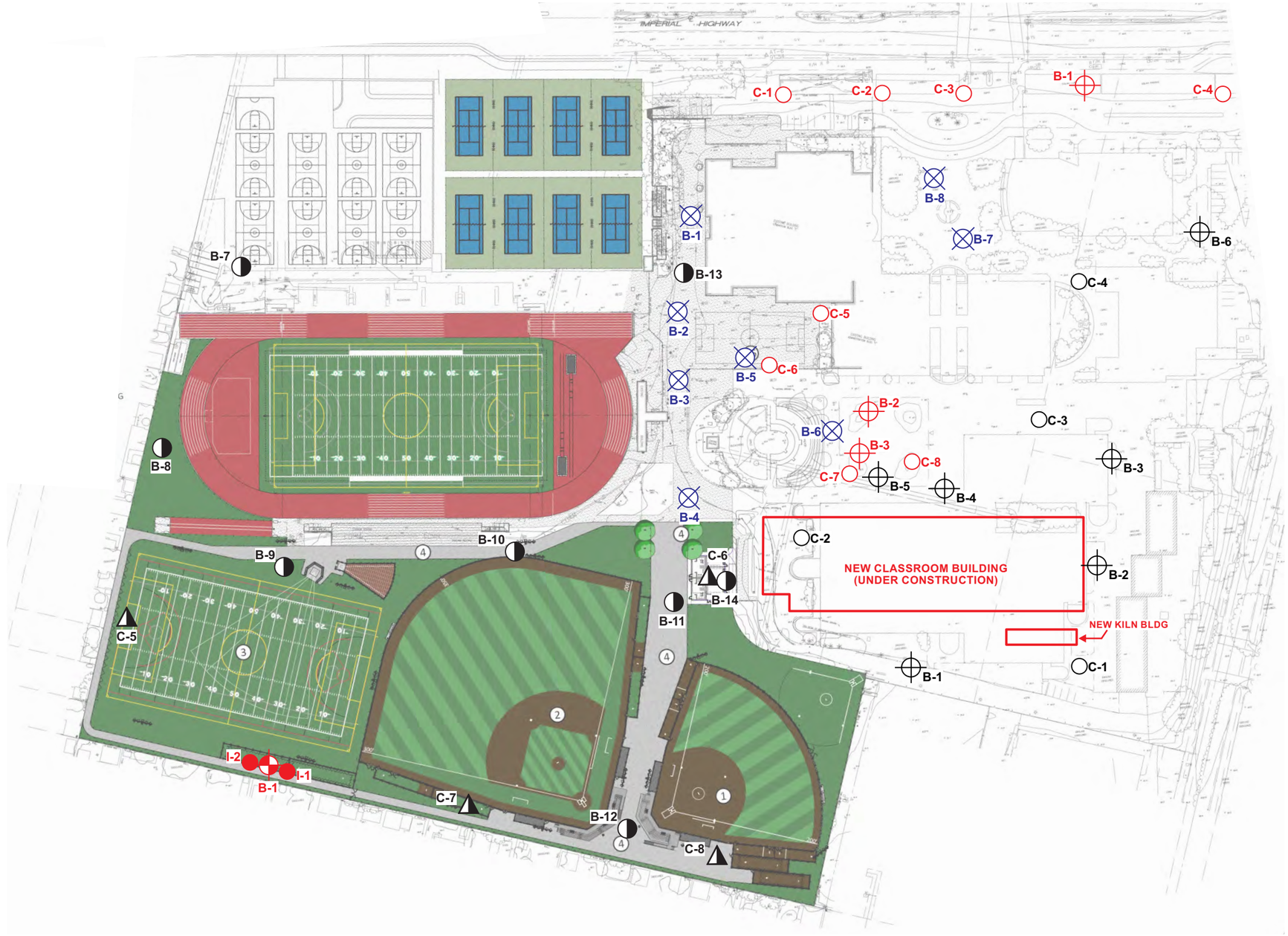
GEOTECHNICAL PROFESSIONALS, INC.

LYNWOOD HIGH SCHOOL FIELD RENOVATION

GPI PROJECT NO.: 3328.I | SCALE: 1" = 150'

AERIAL SITE PLAN

FIGURE 2



EXPLANATION

CURRENT EXPLORATIONS

B-12 — **APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING (GPI PROJ. 3328.I)**

C-8 — **APPROXIMATE LOCATION AND NUMBER OF CONE PENETRATION TEST (GPI PROJ. 3328.I)**

PRIOR EXPLORATIONS

B-1 — APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING (GPI PROJ. 3226.I)

I-2 — APPROXIMATE LOCATION AND NUMBER OF INFILTRATION WELLS (GPI PROJ. 3226.I)

B-6 — APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING (GPI PROJ. 3070.I)

C-4 — APPROXIMATE LOCATION AND NUMBER OF CPT (GPI PROJ. 3070.I)

B-1 — APPROXIMATE LOCATION AND NUMBER OF PRIOR BORING (GPI PROJ. 2705.I)

C-2 — APPROXIMATE LOCATION AND NUMBER OF PRIOR CPT (GPI PROJ. 2526.09I)

B-1 — APPROXIMATE LOCATION AND NUMBER OF PRIOR BORING (GPI PROJ. 2526.09I)

0 150 300 FEET

SITE PLAN REPRODUCED FROM SITE COLOR LAYOUT PLAN PROVIDED BY TBP ARCHITECTURE PLANNING: DATED 02-21-25

GPI GEOTECHNICAL PROFESSIONALS, INC.

LYNWOOD HIGH SCHOOL FIELD RENOVATION

GPI PROJECT NO.: 3328.I | SCALE: 1" = 150'

PROPOSED SITE PLAN

FIGURE 3

APPENDIX A

APPENDIX A

CONE PENETRATION TESTS

The subsurface conditions were investigated by performing four Cone Penetration Tests (CPT's) (CPTs C-7 through C-8) at the site. The soundings were advanced to approximate depths of 50 feet below existing grades. The locations of our recent and prior the CPT's at the site are shown on the Aerial 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. The CPT described in this report was conducted in general accordance with ASTM specifications (ASTM D5778) 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.

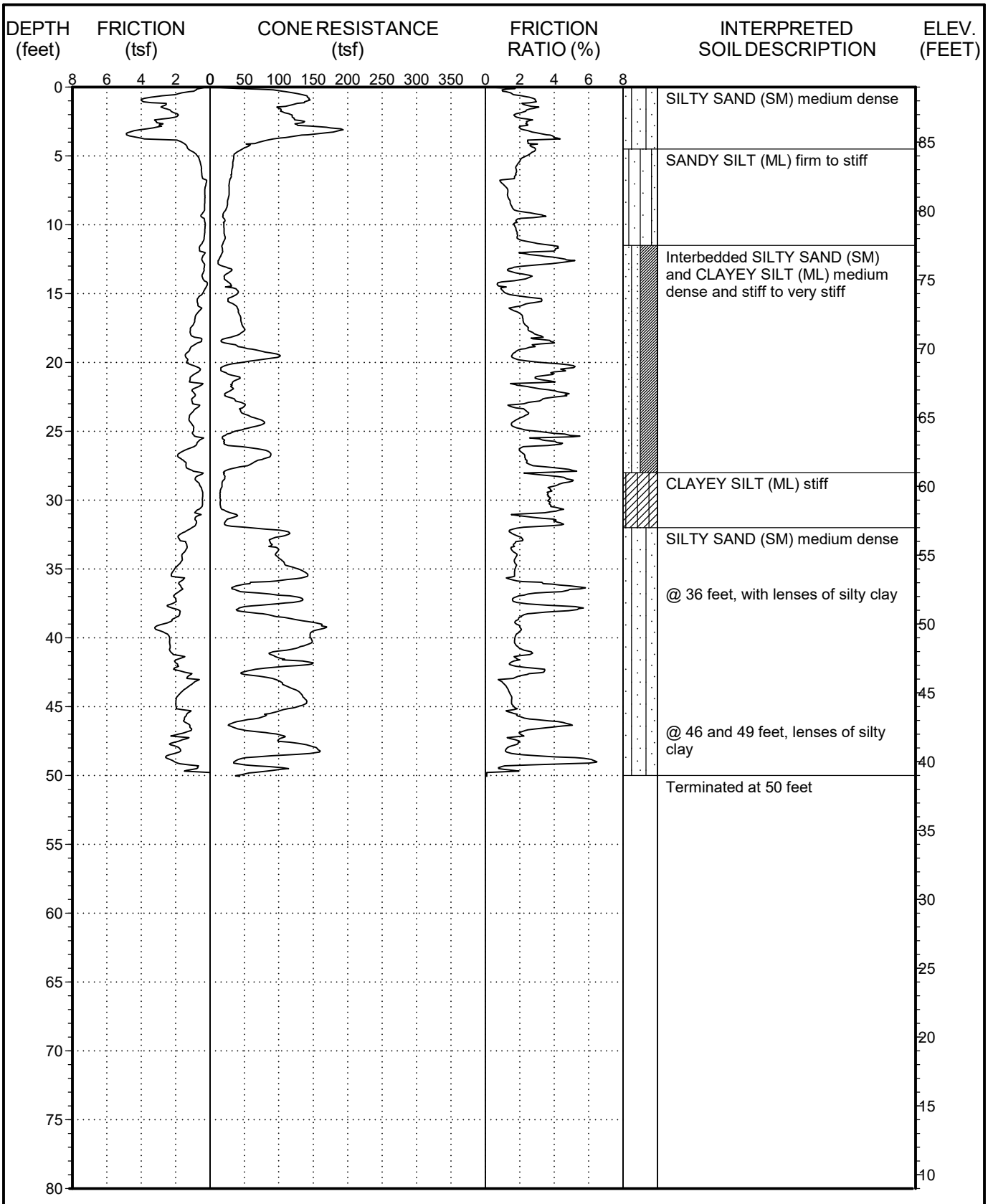
Granular soils typically have low friction ratios and high cone resistance, while cohesive or organic soils have high friction ratios and low cone resistance. These stratigraphic material categories form the basis for all subsequent calculations which utilize the CPT data.

Computer plots of the reduced CPT data acquired for this investigation are presented in Figures A-1 to A-4 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.

We obtained piezometric measurements at CPT C-8. At an approximate depth 50 feet below existing grades, we measured a stabilized pressure of approximately 2.2 pounds per square inch (psi), corresponding to an approximate piezometric head of 5.1 feet. As such, the resulting stabilized groundwater depth is approximately 45 feet below existing site grades.

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.

CPT's performed in asphalt or concrete areas were patched with cold-patch asphalt or rapid-set concrete, respectively. Ground surface elevations at the CPT locations were estimated from internet sources (Google Earth) and should be considered approximate.



Date performed:7-2-25

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.

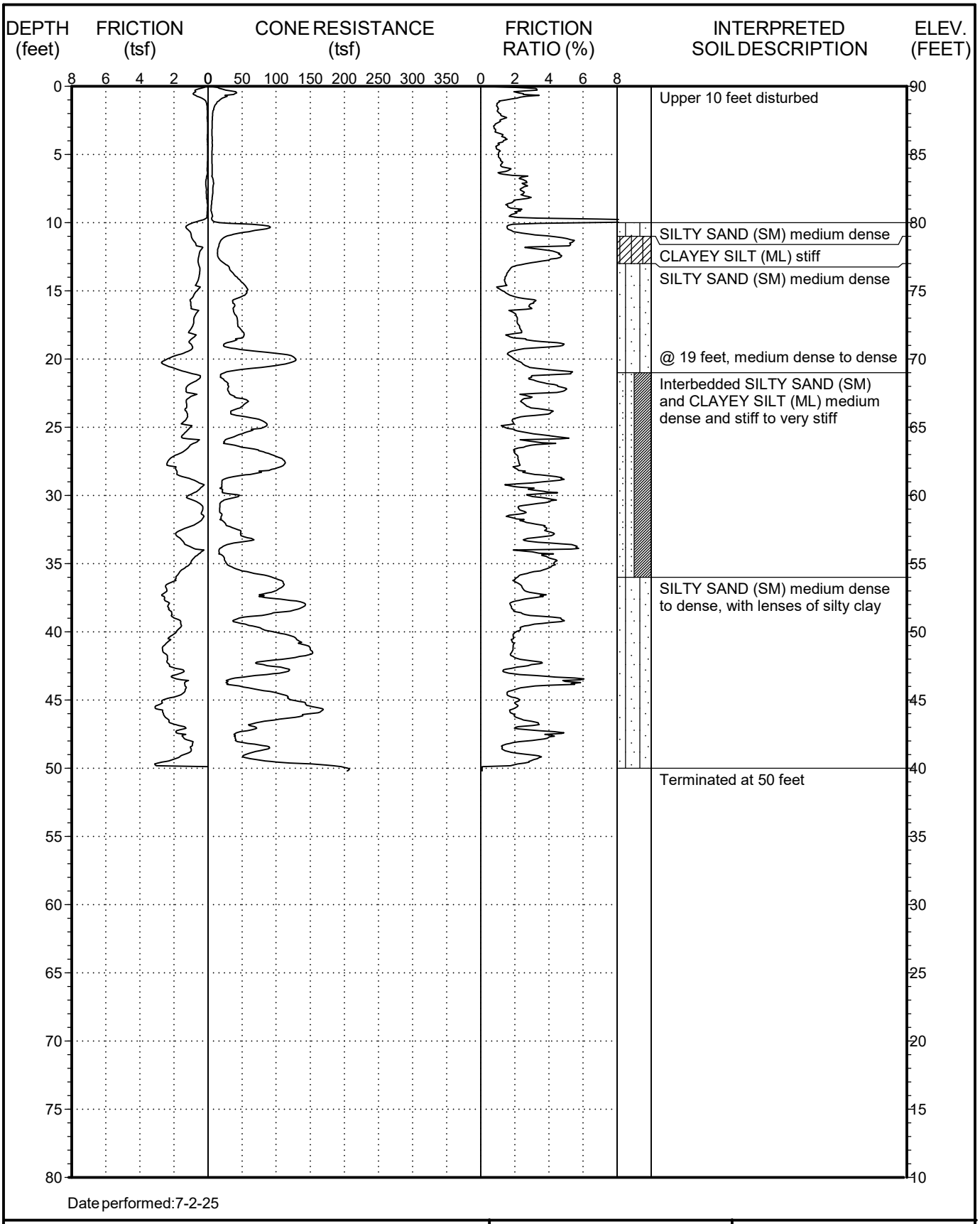
GPI

PROJECT NO.: 3328.I

LYNWOOD HS FIELD RENOVATION

LOG OF CPT NO. C-5

FIGURE A-1



Date performed:7-2-25

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.

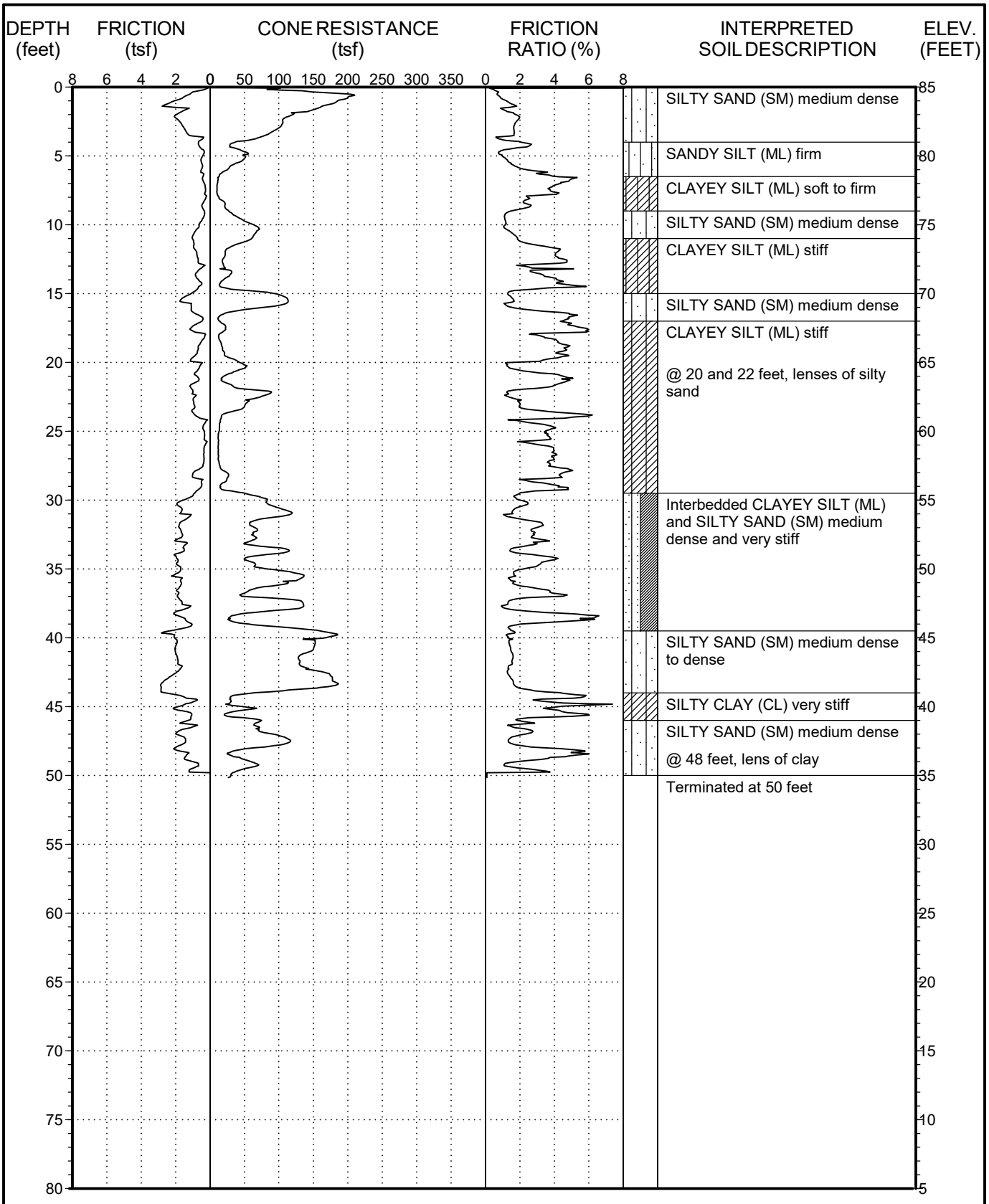
GPI

PROJECT NO.: 3328.I

LYNWOOD HS FIELD RENOVATION

LOG OF CPT NO. C-6

FIGURE A-2



Date performed:7-2-25

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.

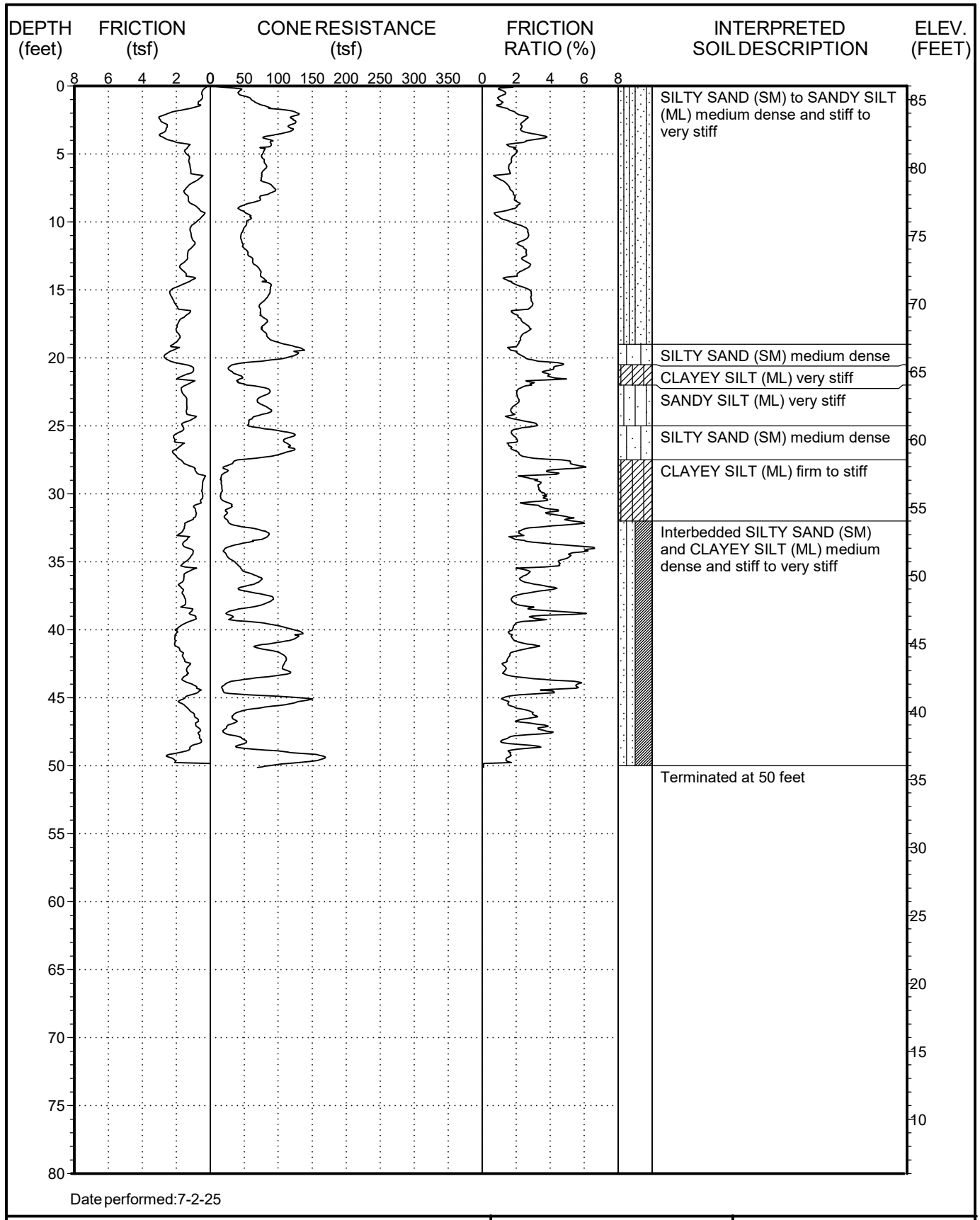
GPI

PROJECT NO.: 3328.I

LYNWOOD HS FIELD RENOVATION

LOG OF CPT NO. C-7

FIGURE A-3



Date performed:7-2-25

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.

GPI

PROJECT NO.: 3328.I

LYNWOOD HS FIELD RENOVATION

LOG OF CPT NO. C-8

FIGURE A-4

APPENDIX B

APPENDIX B

EXPLORATORY BORINGS

We investigated the subsurface conditions at the site by drilling and sampling eight exploratory borings (B-7 through B-14) during our recent investigation. The recent borings were advanced to approximate depths ranging from 6 to 51 feet below the existing ground surface. During a prior 2024 investigation in the athletic field, we drilled one boring (B-1; GPI Project No. 3226.I) to an approximate depth of 55½ feet below site grades. The locations of the recent and prior explorations are shown on the Aerial and Proposed Site Plans, Figures 2 and 3, respectively.

The borings were drilled using hollow-stem auger drill equipment mounted on a truck rig.

Relatively undisturbed samples were obtained using a brass-ring lined sampler (ASTM D3550). 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 D6066). 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-9 in this appendix.

The boring locations were laid out in the field by measuring from existing site features.

The borings were backfilled with drill spoils and patched with concrete. Ground surface elevations at the exploration locations were estimated from internet sources (Google Earth) and should be considered approximate.

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	*DESCRIPTION OF SUBSURFACE MATERIALS*	ELEVATION (FEET)
				0	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.	
					3-Inch AC over 5-Inch Base	
4.9	107	50/6"	D		Fill: **SILTY SAND (SM)** brown, slightly moist, very dense	85
					@ 4 feet, metal debris	
13.4	94	31	D	5	Natural: **SILTY SAND (SM)** brown, moist to very moist, medium dense	
					Total Depth 6 feet	

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

DATE DRILLED: 7-2-25

EQUIPMENT USED: 8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft): Not Encountered

GPI

PROJECT NO.: 3328.I

LYNWOOD HS FIELD RENOVATION

LOG OF BORING NO. B-7

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	Fill: **SILTY SAND (SM)** brown, moist, very dense, trace gravel	
	8.7	89	50/3"	D			85
	14.0	86	44	D	5	Natural: **SANDY SILT (ML)** brown, moist, very stiff	
						SILTY SAND (SM) brown, moist, medium dense	
	7.7	85	21	D		**SANDY SILT (ML)** dark brown, moist, stiff	80
	29.0	86	31	D	10	**CLAYEY SILT (ML)** dark brown, wet, very stiff, trace sand	75
	13.7	88	35	D	15	**SILTY SAND (SM)** greyish brown, moist to very moist, medium dense	
						Total Depth 16 feet	

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

DATE DRILLED: 7-2-25

EQUIPMENT USED: 8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft): Not Encountered

GPI

PROJECT NO.: 3328.I

LYNWOOD HS FIELD RENOVATION

LOG OF BORING NO. B-8

FIGURE B-2

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)
				0	Fill: **SILTY SAND (SM)** brown, slightly moist, very dense	90
6.9	96	50/6"	D			
16.7	87	81	D	5	Natural: **SILTY SAND (SM)** greyish brown, very moist to wet, very dense	85
6.6	87	48	D		@ 7 feet, slightly moist, dense	
46.0	69	26	D	10	**SILT (ML)** dark brown, wet, very stiff	80
11.4	83	30	D	15	**SILTY SAND (SM)** brown, moist, medium dense	75
13.6	96	44	D	20	**SANDY SILT (ML)** dark brown, moist, very stiff	70
					SILTY SAND (SM) greyish brown, moist to very moist, medium dense to dense	
18.1	90	55	D	25	@ 25 feet, wet, dense	65
31.1	88	49	D	30	**CLAYEY SILT(ML)** greyish brown, wet, hard	60
14.5	101	88	D	35	**SILTY SAND (SM)** grey, very moist, very dense	55
					Total Depth 36 feet	

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

DATE DRILLED: 7-2-25

EQUIPMENT USED: 8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft): Not Encountered

GPI

PROJECT NO.: 3328.I

LYNWOOD HS FIELD RENOVATION

LOG OF BORING NO. B-9

FIGURE B-3

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)
				0	Fill: **SANDY SILT (ML)** brown, slightly moist, hard, with gravel	
9.0	114	50/6"	D			85
8.0	131	55	D	5	Natural: **SILTY SAND (SM)** greyish brown, moist, dense	
5.7	97	34	D		@ 7 feet, slightly moist, medium dense	80
43.8	72	15	D	10	**CLAYEY SILT (ML)** dark brown, wet, stiff	
						75
13.8	87	49	D	15	**SILTY SAND (SM)** greyish brown, moist to very moist, dense	
						70
14.4	95	68	D	20	@ 20 feet, very moist	
						65
35.7	83	72	D	25	**CLAYEY SILT (ML)** dark brown, wet, hard	
					SILTY SAND (SM) greyish brown, very moist, dense	60
30.5	88	38	D	30	**SILTY CLAY (CL)** grey, wet, very stiff	
						55
18.1	110	50/6"	D	35	**SANDY SILT (ML)** greyish brown, wet, hard	
					SILTY SAND (SM) greyish brown, very moist, very dense	
						50

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

DATE DRILLED: 7-2-25

EQUIPMENT USED: 8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft): 44

GPI

PROJECT NO.: 3328.I

LYNWOOD HS FIELD RENOVATION

LOG OF BORING NO. B-10

FIGURE B-4

	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)
	12.9	98	75	D	40	@ 40 feet, grey, moist to very moist, dense to very dense	45
	30.7	92	75	D	45	@ 45 feet, wet	40
	24.5	96	78	D	50	@ 50 feet, very dense	
						Total Depth 51 feet	

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

DATE DRILLED: 7-2-25

EQUIPMENT USED: 8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft): 44

GPI

PROJECT NO.: 3328.I

LYNWOOD HS FIELD RENOVATION

LOG OF BORING NO. B-10

FIGURE B-4

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)
				0	Fill: **SILTY SAND (SM)** light brown, moist, very dense	
12.8	102	77	D			85
13.5	110	50/6"	D	5	Natural: **SILTY SAND (SM)** dark brown, moist to very moist, trace gravel	
9.4	105	84	D		**SAND (SP)** reddish brown, moist, very dense, trace gravel	
9.4	108			10	**CLAYEY SAND (SC)** dark brown, moist, very dense	80
25.1	95	47	D		**SANDY SILT (ML)** dark brown, wet, hard	75
5.2	92	47	D	15	**SILTY SAND (SM)** greyish brown, slightly moist, medium dense to dense	70
15.0	76	50/6"	D	20	**SANDY SILT (ML)** dark brown, moist, hard	
					SILTY SAND (SM) brown, very moist, very dense	65
11.6	100	48	D	25	@ 25 feet, moist, dense	60
31.0	88	24	D	30	**CLAY (CL)** dark grey, wet, very stiff	55
7.8	86	64	D	35	**SANDY SILT (ML)** dark grey, slightly moist, hard	
					Total Depth 36 feet	

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

DATE DRILLED: 7-2-25

EQUIPMENT USED: 8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft): Not Encountered

GPI

PROJECT NO.: 3328.I

LYNWOOD HS FIELD RENOVATION

LOG OF BORING NO. B-11

FIGURE B-5

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: **SANDY SILT (ML)** greyish brown, moist, very stiff	85
20.5	89	42	D		**SANDY CLAY (CL)** dark grey, very moist, very stiff	
					SANDY SILT (ML) brown, moist, very stiff	
4.3	80	31	D	5	@ 5 feet, light grey, slightly moist	80
22.2	89	29	D		Natural: **SANDY SILT (ML)** dark brown, wet, very stiff	
5.4	85	49	D	10	**SILTY SAND (SM)** greyish brown, slightly moist, dense	75
16.6	90	45	D	15	@ 15 feet, very moist to wet	70
33.5	78				@ 16 feet, lens of clay, wet	
9.6	102	53	D	20	**SANDY SILT (ML)** greyish brown, moist, hard	65
31.4	89	50	D	25	**CLAYEY SILT (ML)** grey, wet, very stiff	60
18.5	106	70	D	30	**SILTY SAND (SM)** dark grey, wet, dense	55
19.2	107	75	D	35	@ 35 feet, dense to very dense	50
					Total Depth 36 feet	

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

DATE DRILLED: 7-2-25

EQUIPMENT USED: 8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft): Not Encountered

GPI

PROJECT NO.: 3328.I

LYNWOOD HS FIELD RENOVATION

LOG OF BORING NO. B-12

FIGURE B-6

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	7-Inch AC over 5-Inch Base	90
14.8	107	50/5"	D		**Fill: SANDY SILT (ML)** dark brown, moist, hard	
22.6	96	67	D	5	Natural: **SANDY SILT (ML)** dark brown, wet, hard	85
9.1	90	52	D		**SILTY SAND (SM)** brown, moist, dense	
2.6	85	22	D	10	**SAND WITH SILT (SP-SM)** brown, slightly moist, medium dense	80
					SILTY SAND (SM) brown, slightly moist, medium dense	
8.9	85	40	D	15	**SANDY SILT (ML)** brown, slightly moist, very stiff	75
					Total Depth 16 feet	

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

DATE DRILLED: 7-2-25

EQUIPMENT USED: 8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft): Not Encountered

GPI

PROJECT NO.: 3328.I

LYNWOOD HS FIELD RENOVATION

LOG OF BORING NO. B-13

FIGURE B-7

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)
				0	Fill: **SILTY SAND (SM)** light brown, slightly moist, very dense	90
6.4	101	90	D			
3.5	111	50/6"	D	5	@ 5 feet, trace gravel	85
16.0	89	58	D		Natural: **CLAY (CL)** brown and black, moist, hard, trace sand	
9.1	94				**CLAYEY SAND (SC)** brown, moist, dense	
13.4	96	51	D	10	**SILTY SAND (SM)** grey, very moist, dense	80
					Total Depth 11 feet	

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

DATE DRILLED: 7-2-25

EQUIPMENT USED: 8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft): Not Encountered

GPI

PROJECT NO.: 3328.I

LYNWOOD HS FIELD RENOVATION

LOG OF BORING NO. B-14

FIGURE B-8

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
				0	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.	
					Fill: **SILTY SAND (SM)/SANDY SILT (ML)** light brown, slightly moist, loose/firm	
20.4	88	10	D			
33.9	80	25	D	5	@ 5 feet, medium dense/very stiff	80
33.3	83	21	D		**CLAYEY SILT (ML)** dark brown, moist, stiff	
16.9		17	S	10	**SILTY SAND (SM)** brown, moist, medium dense	75
26.9		20	S		**SILT (ML)** brown, moist, very stiff, with sand	
34.3		17	S		**SILTY CLAY (CL)** brown, moist, very stiff	
18.6		30	S	15	**SANDY SILT (ML)** brown, moist, very stiff to hard	70
40.3					**CLAY (CL)** brown, moist, very stiff to hard	
25.6		23	S		**CLAYEY SILT (ML)** brown, moist, very stiff	
21.8		24	S	20		65
25.9		37	S		**SILTY SAND (SM)** brown, slightly moist, dense	
16.4						
32.1		25	S	25	**CLAYEY SILT (ML)** brown, moist, very stiff	60
35.7		17	S		**SILTY CLAY (CL)** grey brown, moist, very stiff	
31.6		16	S			
20.6		38	S	30	**SILTY SAND (SM)** dark brown, slightly moist, dense	55
20.9		41	S			
13.4		41	S	35	@ 34 feet, less fines	50
9.3		47	S			
25.2		48	S		@ 38 feet, more fines	
					THIS BORING IS FROM A PRIOR GEOTECHNICAL INVESTIGATION	45

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

DATE DRILLED: 2-15-24

EQUIPMENT USED: 8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft): 42

GPI

PROJECT NO.: 3226.I

LYNWOOD HIGH SCHOOL

LOG OF BORING NO. B-1

FIGURE B-9

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.	
24.3		48	S	40	**SILTY SAND (SM)** grey, wet, dense	
26.7		50	S			
24.8		43	S		**CLAYEY SILT (ML)** grey, moist, hard	40
				45	**SILTY SAND (SM)** grey, wet, dense	
29.2		59	S		@ 47 feet, very dense	
31.5		36	S		**SILT (ML)** grey, slightly moist, hard	
39.5					**SILTY CLAY (CL)** grey, slightly moist, hard	35
28.3		35	S	50	**SANDY SILT (ML)** grey, slightly moist, hard	
25.7		32	S		**CLAYEY SILT (ML)** grey, slightly moist, hard	
23.4		46	S	55		30
					Total Depth 55.5 feet	

THIS BORING IS FROM A PRIOR GEOTECHNICAL INVESTIGATION

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

DATE DRILLED: 2-15-24

EQUIPMENT USED: 8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft): 42

GPI

PROJECT NO.: 3226.I

LYNWOOD HIGH SCHOOL

LOG OF BORING NO. B-1

FIGURE B-9

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 samples. The samples were weighed to determine the wet weight and then were dried in accordance with ASTM D2216. 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 select samples in accordance with ASTM D4318. The results of the Atterberg Limits tests are presented in Figure C-1.

GRAIN SIZE DISTRIBUTION

Select 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-1*	10	Silty Sand (SM)	39
B-1*	12	Silt (ML) with Sand	76
B-1*	14	Silty Clay (CL)	88
B-1*	30	Silty Sand (SM)	49
B-1*	34	Silty Sand (SM)	33
B-1*	38	Silty Sand (SM)	43
B-8	10	Clayey Silt (ML)	84
B-10	2	Sandy Silt (ML)	60
B-11	5	Silty Sand (SM)	48
B-12	15	Silty Sand (SM)	47

***** From GPI Project No. 3226.I; report dated March 8, 2024

DIRECT SHEAR

Direct shear tests were performed on select undisturbed in accordance with ASTM D3080. The samples were placed in the shear machine, and pre-selected normal loads were applied. The samples were submerged, allowed to consolidate, and then were sheared to failure. Shear stress and sample deformation were monitored throughout the tests. The results of the direct shear test are presented in Figures C-2 to C-3.

CONSOLIDATION

One-dimensional consolidation testing was performed on selected undisturbed samples in accordance with ASTM D2435. After trimming the ends, the samples were placed in the consolidometer and loaded to 0.4 ksf. Thereafter, the samples were incrementally loaded to a maximum load of 25.6 ksf. The samples were inundated at 1.6 ksf. Sample deformation was measured to 0.0001 inch. Rebound behavior was investigated by unloading the samples back to 0.4 ksf. Results of the consolidation tests, in the form of percent consolidation versus log pressure, are presented in Figures C-4 and C-5.

EXPANSION INDEX

An expansion index test was performed on a select bulk sample. The test was performed in accordance with ASTM D4829 to assess the expansion potential of on-site soils. The results of the test are summarized below:

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	EXPANSION INDEX
B-12	0 – 5	Sandy Silt (ML)	8

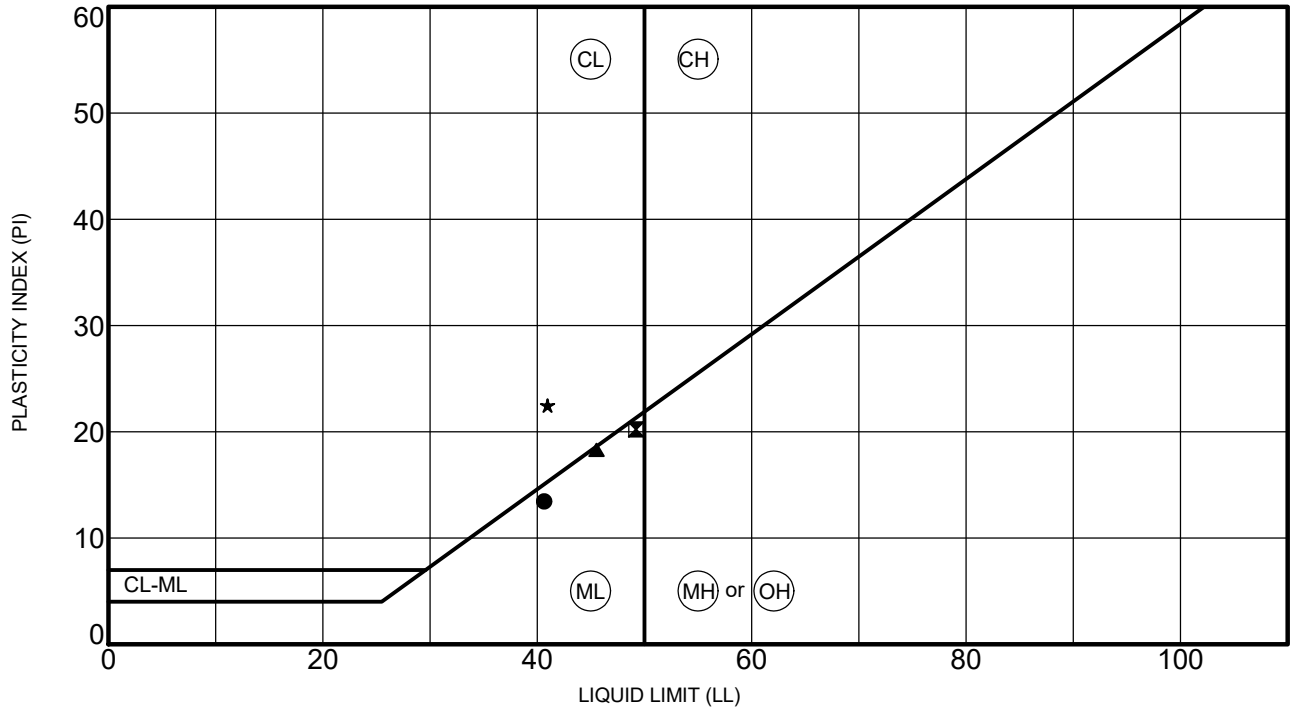
COMPACTION TEST

A maximum dry density/optimum moisture test was performed in accordance with ASTM D1557 on a representative bulk sample of the surficial soils. The test results is as follows.

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)
B-12	0 – 5	Sandy Silt (ML)	114	15

CORROSIVITY

Soil corrosivity testing was performed by Project X on selected soil samples provided by GPI. The test results are summarized in the table, Soil Analysis Lab Results, attached to this Appendix.



	SAMPLE LOCATION		LL	PL	PI	Fines, %	Classification
●	B-08	10.0	41	27	13		CLAYEY SILT (ML)
⊠	B-09	30.0	49	29	20		CLAYEY SILT (ML)
▲	B-12	25.0	46	27	18		CLAYEY SILT (ML)
★	B-14	7.0	41	18	22		CLAY (CL)

PROJECT: LYNWOOD HS FIELD RENOVATION PROJECT NO. 3328.I

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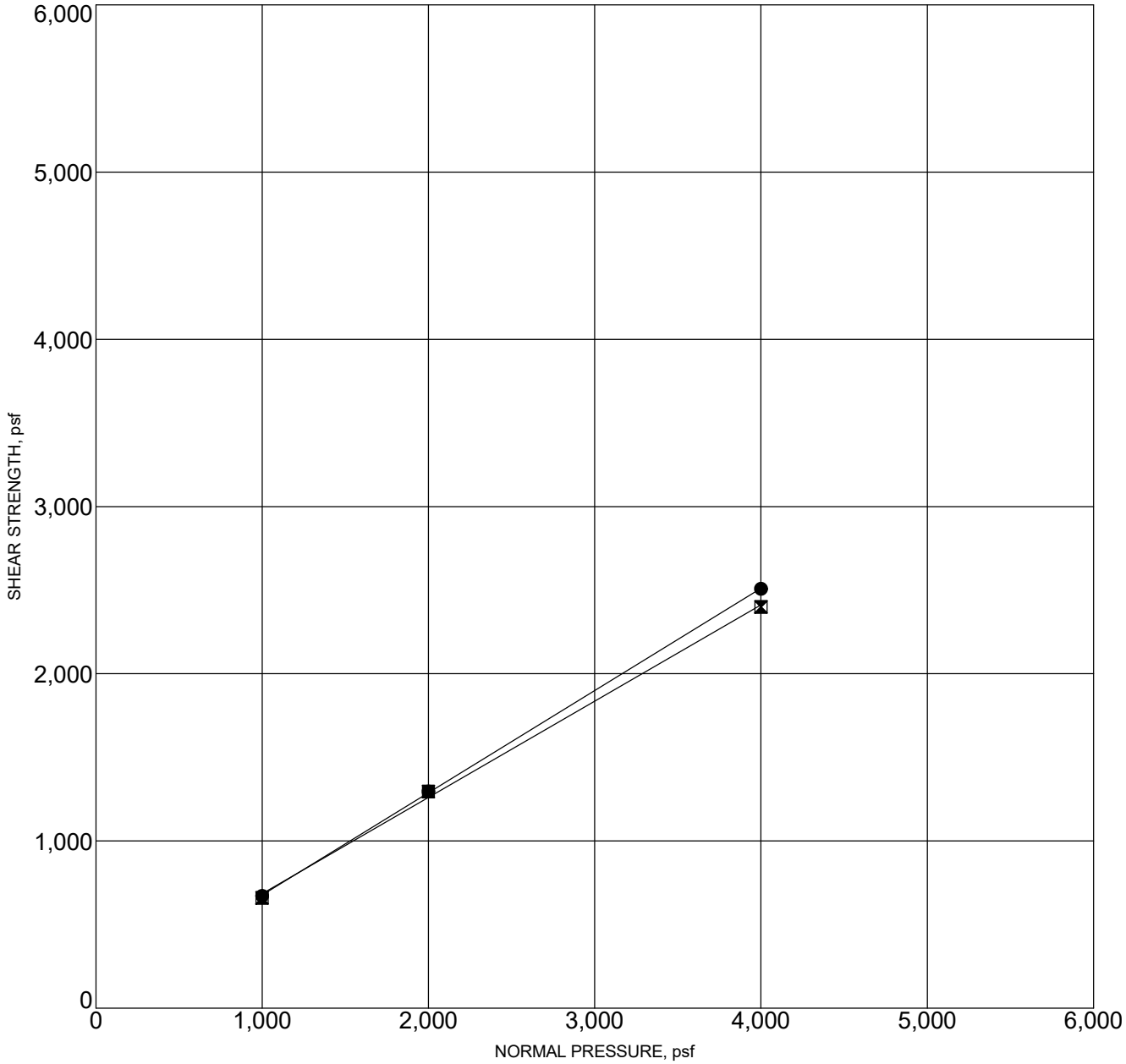
ATTERBERG LIMITS TEST RESULTS

FIGURE C-1



● **PEAK STRENGTH**
Friction Angle= 31 degrees
Cohesion= 66 psf

⧓ **ULTIMATE STRENGTH**
Friction Angle= 30 degrees
Cohesion= 108 psf

Sample Location		Classification	DD,pcf	MC,%
B-9	5.0	SILTY SAND (SM)	87	16.7

PROJECT: LYNWOOD HS FIELD RENOVATION PROJECT NO.: 3328.I

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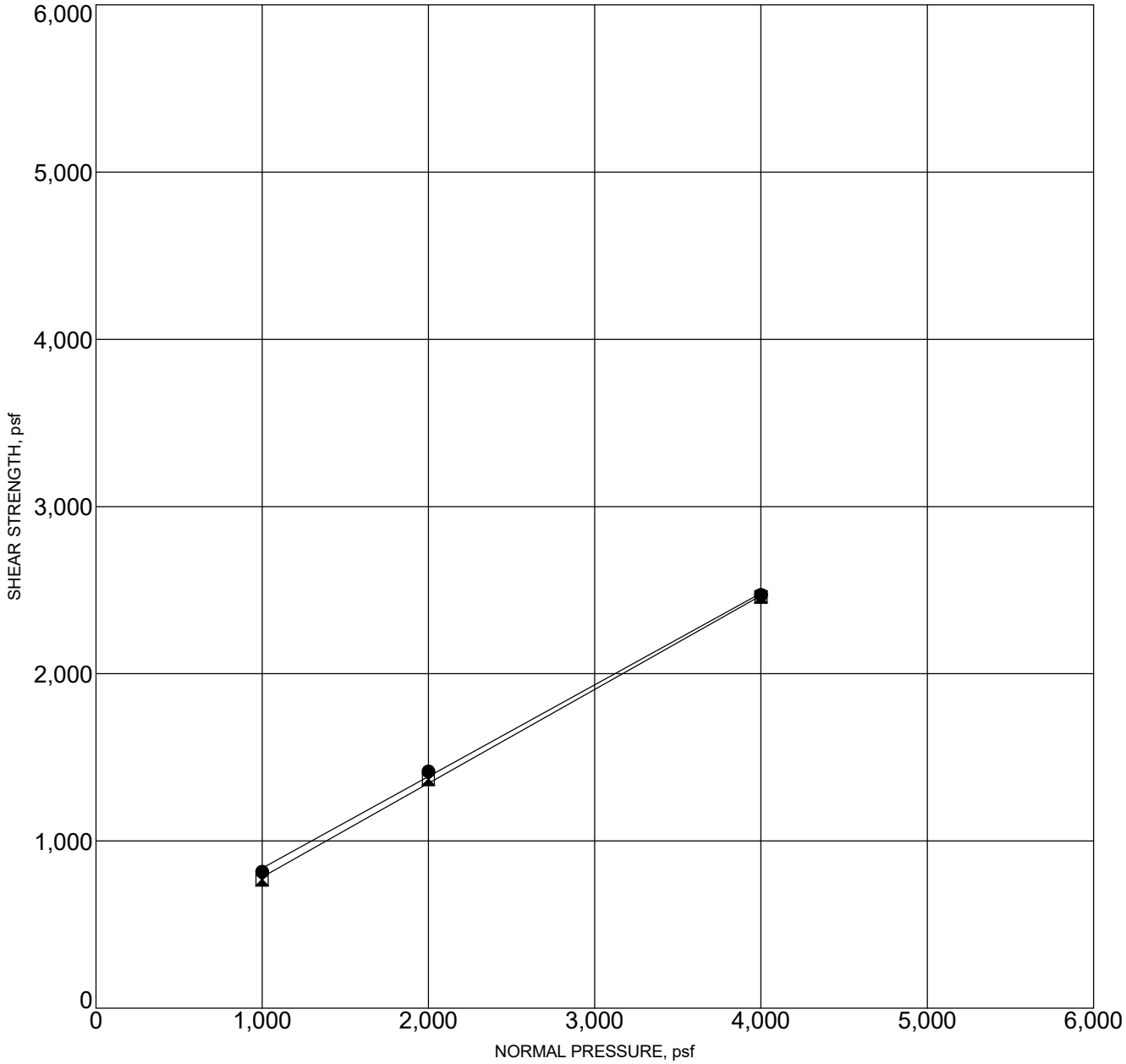
DIRECT SHEAR TEST RESULTS

FIGURE C-2



● **PEAK STRENGTH**
Friction Angle= 29 degrees
Cohesion= 288 psf

⧓ **ULTIMATE STRENGTH**
Friction Angle= 29 degrees
Cohesion= 222 psf

Sample Location		Classification	DD,pcf	MC,%
B-11	7.0	CLAYEY SAND (SC)	105	9.4

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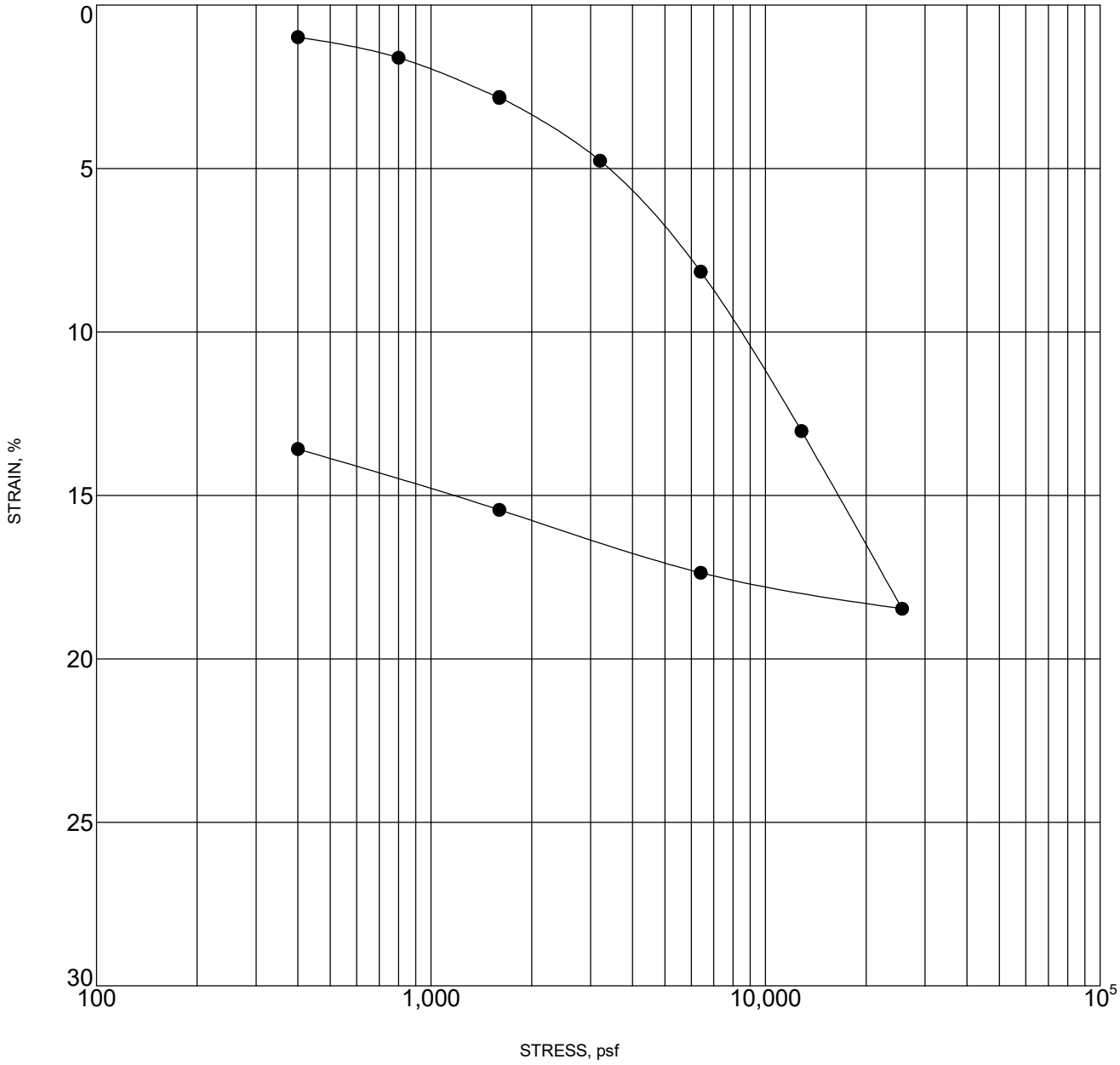
DIRECT SHEAR TEST RESULTS

FIGURE C-3



Sample inundated at 1600 psf

	Sample Location		Classification	DD,pcf	MC,%
●	B-10	10.0	CLAYEY SILT (ML)	72	43.8

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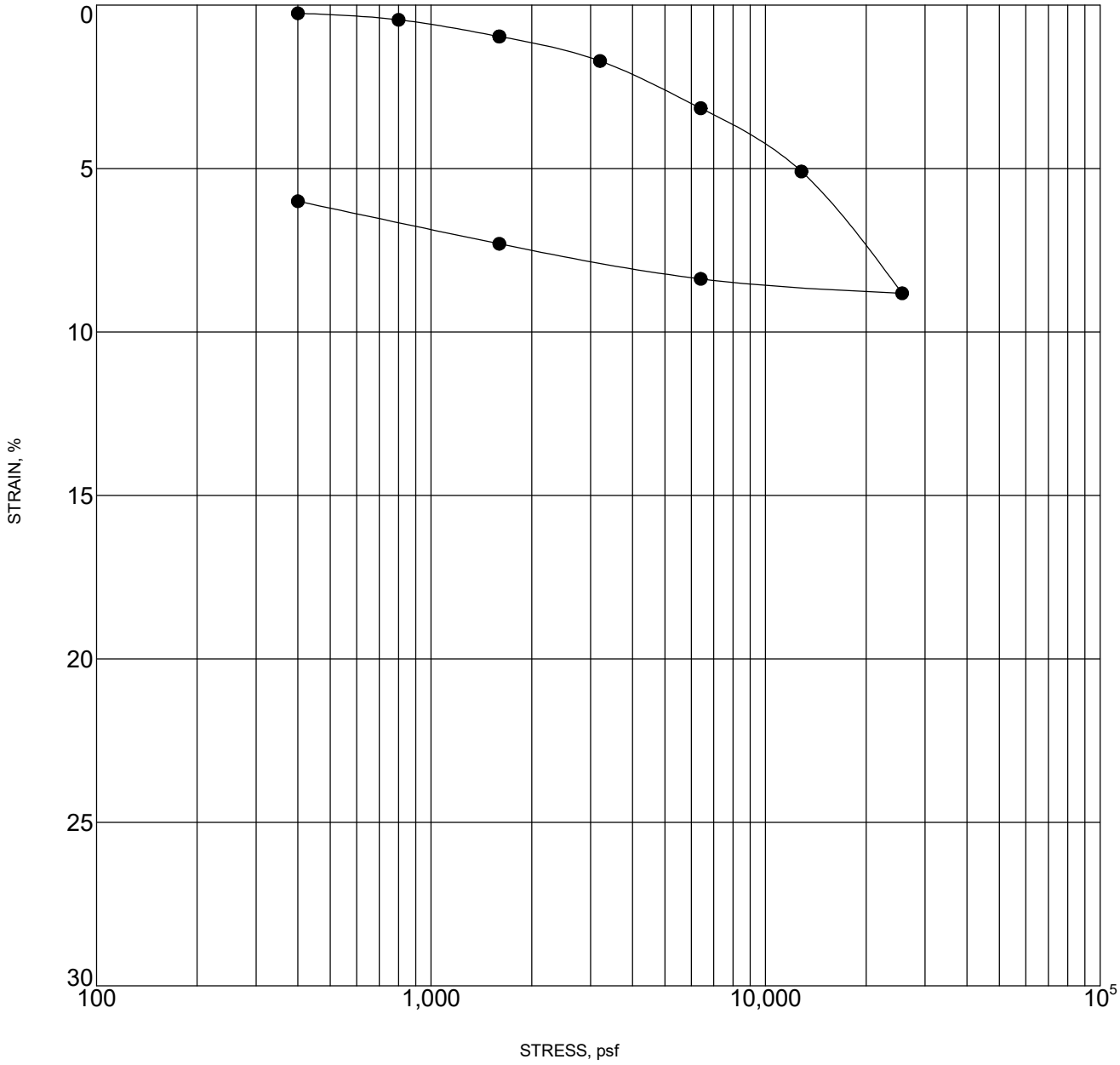
CONSOLIDATION TEST RESULTS

FIGURE C-4



Sample inundated at 1600 psf

	Sample Location		Classification	DD,pcf	MC,%
●	B-12	7.0	SANDY SILT (ML)	89	22.2

PROJECT: LYNWOOD HS FIELD RENOVATION PROJECT NO.: 3328.I

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CONSOLIDATION TEST RESULTS

FIGURE C-5

Soil Analysis Lab Results

Client: Geotechnical Professionals Inc.
Job Name: Lynwood H. S.
Client Job Number: 3328.I
Project X Job Number: S250715D
July 16, 2025

	Method	ASTM D4327		ASTM D4327		ASTM G187		ASTM G51
Bore# / Description	**Depth**	**Sulfates** SO_4^{2-}		**Chlorides** Cl^-		**Resistivity** As Rec'd | Minimum		**pH**
	(ft)	**(mg/kg)**	**(wt%)**	**(mg/kg)**	**(wt%)**	**(Ω-cm)**	**(Ω-cm)**	
PS-1	0-5	43.0	0.0043	19.7	0.0020	34,840	4,489	8.3
B-8	0-5	909.3	0.0909	77.8	0.0078	14,070	871	7.9
B-12	0-5	32.6	0.0033	14.5	0.0014	34,170	6,566	7.9

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

For AWWA 105C: 0-3mg/kg sulfide = Negative; 3-6mg/kg = trace; >6mg/kg = Positive

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

If one sample pops up much more corrosive than all others, we would recommend collecting more samples surrounding the problem sample location to determine if the peak is isolated to it. This allows us to conclude it was a contaminated sample and able to declare it an outlier.

Try out our new online forms: SOIL CORROSIVITY & THERMAL RESISTIVITY LAB REQUEST FORM & IN-SITU WENNER 4 PIN QUOTE REQUEST FORM

APPENDIX D

APPENDIX D

GEOLOGIC-SEISMIC HAZARD EVALUATION

INTRODUCTION

This geologic and seismic assessment presents a summary of geologic and seismic conditions at Lynwood High School located at 4050 East Imperial Highway in Lynwood, California. The purpose of this assessment was to identify and evaluate geologic constraints, which are likely to be factors with respect to the proposed development. In order to accomplish this objective, the following scope of services was performed:

- Research and review of available published and unpublished geologic literature and maps pertaining to the site and vicinity (see References), as well as subsurface exploration data from our recent investigation.

- Geologic analysis of the reviewed information.

- Preparation of this assessment report, which includes a summary of the researched information and a discussion of the possible geologic-seismic hazards that may affect the subject site and the proposed construction.

SITE CONDITIONS

The subject school site is approximately 31 acres and is located in an older residential and commercial neighborhood in Lynwood.

Site topography at the school and in the surrounding area is relatively flat. Ground surface elevations within the main campus area range from approximately +85 to +90 feet based on internet sources (i.e., Google Earth).

The proposed project consists of upgrades to the athletic fields. The upgrades will include a new storage/locker room, sixteen +/- 80-foot-tall Musco light poles, new synthetic multi-purpose fields, new synthetic softball and baseball fields with associated fencing, bullpens, dugouts, a batting cage structure, permanent bleachers, a new rubberized field event area, a sand volleyball court, and new hardscape and paving. The existing track and field will also be replaced with a new synthetic track and field and new scoreboards.

REGIONAL AND LOCAL GEOLOGIC SETTING

Regional Geology

The school site is located in the Central Block of a regional geologic structure termed the Los Angeles Basin, a northeast-trending structural basin filled with Tertiary age marine sedimentary rocks mantled by Recent and Pleistocene age non-marine alluvial sediments deposited by washes and streams flowing southward from local hill and mountain ranges (e.g., the Santa Monica Mountains, Elysian Hills, Repetto Hills, etc.).

The subject site is more specifically located on nearly flat terrain in the City of Lynwood in a broadly alluviated area known regionally as the Los Angeles Coastal Plain, a generally low area that extends from the Los Angeles Metropolitan area to the Pacific Ocean to the west. This area is underlain by a thick succession of largely marine sedimentary rocks of Miocene to Pleistocene age, which are several thousands of feet thick. In the vicinity of the site, the marine deposits are overlain by Holocene alluvium, which consists of unconsolidated silts, sands, and gravels. The Holocene alluvium is underlain by moderately to well consolidated, later Pleistocene alluvium, consisting of sand, silt, and gravel (Department of Water Resources, 1961).

The nearest geologic structures to the site are the Puente Hills Blind Thrust and Newport-Inglewood Zone of deformation, both considered active fault zones. Deformation and uplift along the Newport-Inglewood fault zone has resulted in a northwest trending series of hills, including Signal Hill and the Dominguez Hills to the south of the site, and the Baldwin Hills to the northwest of the site. Based on published maps and the USGS Source Parameter website (see References), the site is approximately 2.1 miles and 5.0 miles from the closest known traces of the Puente Hills Blind Thrust and Newport-Inglewood fault, respectively (see Table 1).

Regionally the site is located near the border between two of California's geomorphic provinces, the Transverse ranges to the north and the Peninsular Ranges to the south. The Transverse Ranges are characterized by east-west trending mountain ranges, including the Santa Monica and San Gabriel Mountains, which are oriented oblique to the trend of the other major structural trends in California, including the San Andreas Fault, Sierra Nevada Mountains, and other mountain ranges in Southern California, which trend northwesterly.

The Peninsular Ranges are characterized by northwesterly trending active faults and mountain ranges related to the San Andreas and other major fault systems in the province. The province extends from the Los Angeles Basin, where the project is located, southeast to Baja California.

Site Geologic Conditions

The site is underlain by Quaternary age alluvial sediments mapped as younger, alluvial plain deposits. These sediments are described as loose to medium dense, very coarse- to very fine-grained sand, gravel, and silt. (CGS, 1998). These sediments are horizontally stratified in this site area and combined with the flat topography at the site, do not pose slope stability (landslide) risks. The geologic conditions in the site area are shown on the Regional Geologic Map, Figure D-1.

As encountered in our recent explorations advanced to depths of about 51 feet and prior explorations advanced to depths of about 100 feet, the soils consist of shallow undocumented fill soils over natural younger and older alluvial soils. The fill soils at the boring locations consisted of medium dense to very dense silty sands. The fill soils are considered undocumented and were likely placed during site grading for the existing high school in the 1990's or during prior site development.

The underlying natural materials consisted predominantly of interbedded layers of medium dense to dense silty sands and stiff to very stiff sandy silts, silts, and clays in the upper 30

feet. In general, the soils encountered below depths of 30 feet consisted predominantly of medium dense to dense silty sands with lesser amounts of stiff to very stiff clays and silts.

The natural soils encountered in the upper 25 feet of our recent explorations were found to be generally moist to wet, with higher moisture contents generally encountered within the fine-grained soils. Moisture contents within the upper 25 feet ranged from a low of 3 percent to a high of 44 percent, with an average moisture content of about 16 percent.

Optimum moisture content of the upper soils based on laboratory testing ranged from approximately 10 to 15 percent. Below depths of 25 feet, the natural soils were generally moist to wet, with moisture contents generally ranging from 8 to 31 percent.

Groundwater Conditions

Data published by the State of California indicates that historical high groundwater depth in the site vicinity is approximately 8 feet below existing grades. Groundwater was encountered at an approximate depth of 44 feet in one of our recent borings at the site (Boring B-10). We also obtained piezometric measurements in our recent CPT C-8 at an approximate depth of 50 feet below existing site grades and measured a stabilized pressure of approximately 2.2 pounds per square inch (psi). This corresponds to a stabilized groundwater depth of approximately 45 feet below existing site grades.

The historical high groundwater depths in the site vicinity are shown on the Groundwater Map, Figure D-2.

TECTONIC SETTING

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 all considered active and are all associated with the San Andreas, which collectively form 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 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.

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. Some uplands that surround the Los Angeles Basin, including the Elysian Park and Repetto Hills, are products of blind thrusts. Because blind thrusts do not intersect the ground surface, primary surface fault rupture is considered unlikely.

Major portions of the Los Angeles Basin are now believed to be underlain by various blind thrusts ramps. Due to continued north-south convergence (shortening) across the Los Angeles Basin, slippage along these features will generate earthquakes.

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 such as Los Angeles and surrounding cities.

Blind thrusts are not known to underlie the site, although the Puente hills and Elysian Park thrusts are in close proximity.

Nearby Seismogenic Sources

We reviewed the 2008 National Seismic Hazard Maps Source Parameters (USGS, 2008) to identify known active faults within a 50-mile radius of the project site. The names and distances of the faults lying within 25 miles of the project site are provided in the following table (Table 1). We present a map showing the significant regional faults in Figure D-3, Regional Fault Map.

Table 1 – Significant Regional Faults

Fault Name	Approximate Distance* (mi)
Puente Hills Blind Thrust (Los Angeles)	2.1
Puente Hills (Santa Fe Springs)	3.3
Newport-Inglewood	5.0
Puente Hills (Coyote Hills)	8.7
Elsinore	9.3
Elysian Park (Upper)	9.3
Palos Verdes	12.6
Raymond	13.2
Hollywood	13.3
Santa Monica Connected	13.6
Verdugo	14.1
Santa Monica	16.0
Sierra Madre	18.4
San Jose	19.3
Clamshell-Sawpit	20.3
Malibu Coast	20.5
Anacapa-Dume	21.8
San Joaquin Hills	21.8
Sierra Madre (San Fernando)	24.8

* 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, 1999). Surface faults have not been mapped projecting towards or through the site area.

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

Puente Hills Blind Thrust

The Puente Hills Blind Thrust (Shaw, 1999) is a north dipping blind thrust extending from the Santa Fe anticline northward to the Montebello anticline. Movement on the fault is responsible for the 1987 Whittier Narrows earthquake. Research on the earthquake and its aftershocks, as well as fault plane reflections, have resulted in the conclusions that the fault is located between 3 and 7 kilometers below sea level. Data on the slip rate and possible recurrence intervals are still being researched.

Newport-Inglewood Fault

The Newport-Inglewood Fault forms the southwesterly side of the Los Angeles Basin and is defined by a series of low disconnected hills and mesa surfaces. Strike slip faulting is associated with anticlinal folding. This has resulted in the accumulation of petroleum resources along its entire length from offshore Newport Beach to the Santa Monica Mountains. 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, all of which have been at a magnitude of 4.5 or less. The fault lies at a distance of approximately 5.0 miles to the southwest of the project sites 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.

Elysian Park Blind Thrust

The north to south structural convergence in the region is a result of deep-seated fault movement along features called “blind thrusts.” These are buried low angle north and some south dipping faults which do not project to the ground surface but cause uplift by folding during major earthquakes. In 1987, the magnitude 5.9 Whittier Narrows Earthquake occurred on a previously unknown blind thrust, which has now been given the name Elysian Park Blind Thrust or Structural Zone. This fault underlies the Elysian Park Hills at 3 km and deepens northward to 10 km of depth. Because of the 1987 event, the fault has been placed into an active category and has been tentatively mapped to underlie a major portion of the eastern Los Angeles Basin and adjacent San Gabriel Valley to the north.

Subsequent to this earthquake was the 1994 M6.7 Northridge Earthquake in the San Fernando Valley. This earthquake occurred along a previously unknown similar blind thrust fault. This type of active faulting and resulting earthquake activity are considered relatively common in regions undergoing convergence. The Elysian Park Thrust has a length of 34 km, slip rate of 1.50 mm/yr and is capable of generating a maximum earthquake of M6.7 (Shaw and Suppe, 1996).

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 experienced strong ground shaking due to earthquakes in historic time. The locations of earthquake epicenters with respect to the subject site are shown graphically on Figure D-4, Regional Seismicity.

SECONDARY SEISMIC EFFECTS

General

Secondary effects of seismic activity normally considered as possible hazards to a particular site include several types of ground failure as well as induced flooding. Various types of ground failures, which might occur as a consequence of severe ground shaking of a site include landsliding, ground subsidence, ground lurching, shallow ground rupture and liquefaction. The probability of occurrence of each type of ground failure depends on the severity of the earthquake, distance from faults, topography, subsoils, and groundwater conditions, in addition to other factors. Based on a review of available published literature, landsliding, ground subsidence, ground lurching and shallow ground ruptures are considered unlikely at the site.

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. Since the site is located approximately 13 miles inland from the Pacific Ocean at an elevation of approximately 90 feet above mean sea level. The campus lies 0.8 miles west of the Los Angeles River (the bottom of the river lies approximately 16 feet below the campus grades).

The site does not lie in proximity to an enclosed body of water and does not lie within a mapped FEMA flood zone. As such, the probability of flooding due to a tsunami or seiche is considered to be nonexistent.

Liquefaction Considerations

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 South Gate Quadrangle, the site is located in an area designated by the State Geologist as a "zone of required investigation" due to the potential for earthquake-induced liquefaction. The location of the site relative to the Liquefaction Zone mapped by the State Geologist is shown on Figure D-5, Seismic Hazard Map. Based on our analysis, the anticipated liquefaction-induced seismic settlement at the site is on the order of 2 to 3 inches. For details and results on our liquefaction and seismic settlement evaluation, refer to Section 4.2.4 in the text of our report.

SUMMARY OF GEOLOGIC CONSTRAINTS

Based on the results of our geotechnical investigation and a review of the information provided in the referenced literature, it is recommended that the following geologic constraints be taken into account during the initial planning stages of the proposed development.

- 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. The proposed project should be designed and constructed in accordance with the seismic design parameters provided in the building code and our final geotechnical investigation report.

- Faults have not been mapped projecting towards or through the site. The site is also not located within an Alquist-Priolo Earthquake Fault Zone.

- The site is located in an area designated by the State Geologist as a “zone of required investigation” for liquefaction potential. Details of our liquefaction and seismic settlement evaluation are presented in Section 4.2.4 of the report. Based on this analysis, the potential liquefaction-induced seismic settlement at the site is estimated to be on the order of 2 to 3 inches. Therefore, the potential for liquefaction settlements to negatively impact the proposed site modifications is considered to be moderate to high unless mitigated or incorporated into the planned design.

- Based on a review of available published literature, landsliding, ground subsidence, ground lurching and shallow ground rupture are considered unlikely at the site.

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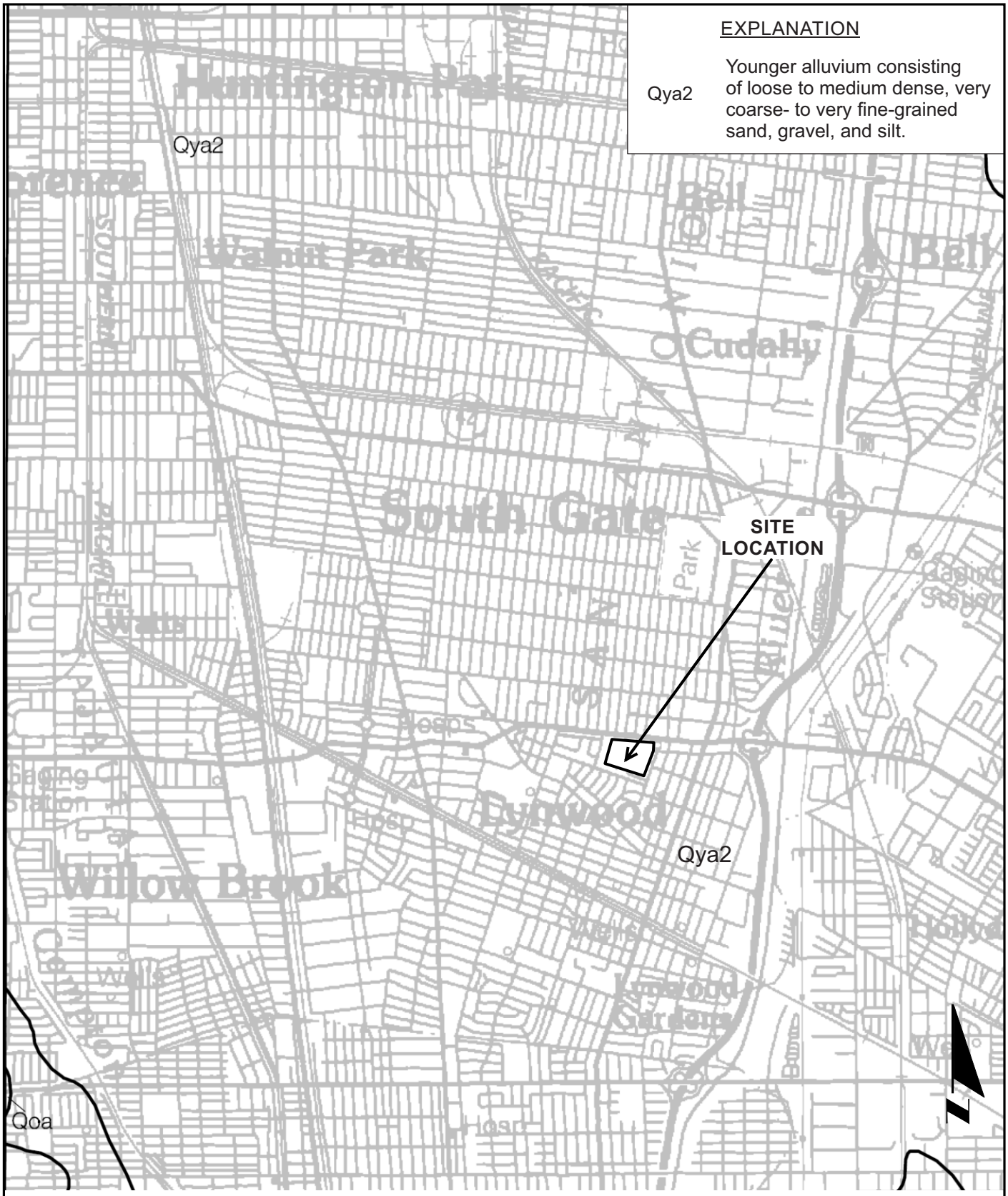


BASE MAP REPRODUCED FROM THE GEOLOGIC MAP OF THE SOUTH GATE QUADRANGLE PREPARED BY THE STATE OF CALIFORNIA DEPARTMENT OF CONSERVATION - DIVISION OF MINES AND GEOLOGY: DATED 1998

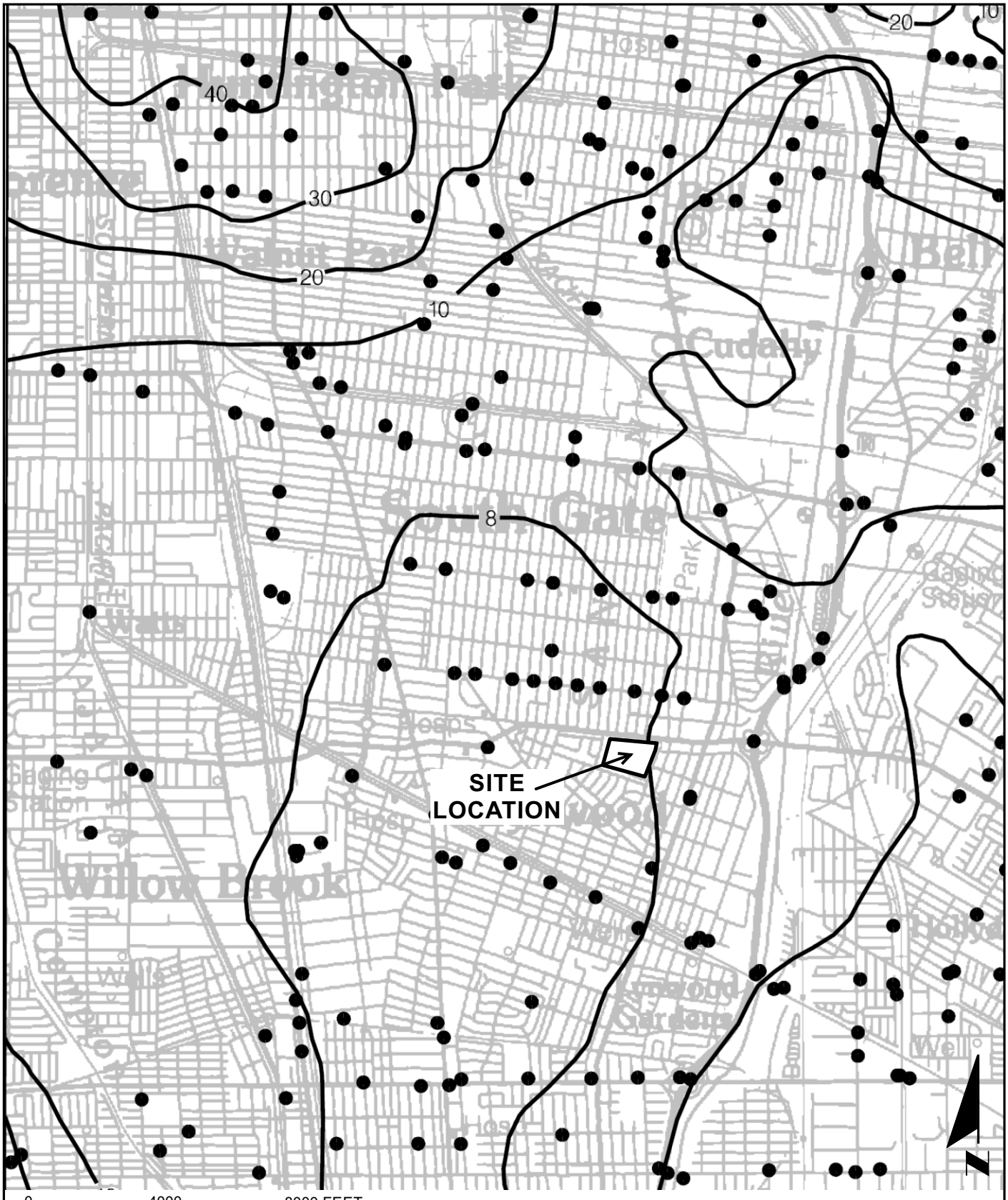
GEOTECHNICAL PROFESSIONALS, INC.

LYNWOOD HIGH SCHOOL FIELD RENOVATION

GPI PROJECT NO.: 3328.I | SCALE: 1" = 4000'

REGIONAL GEOLOGIC MAP

FIGURE D-1



BASE MAP REPRODUCED FROM HISTORICAL GROUNDWATER MAP
(PLATE 1.2) FROM SEISMIC HAZARD ZONE REPORT 034 (1998)

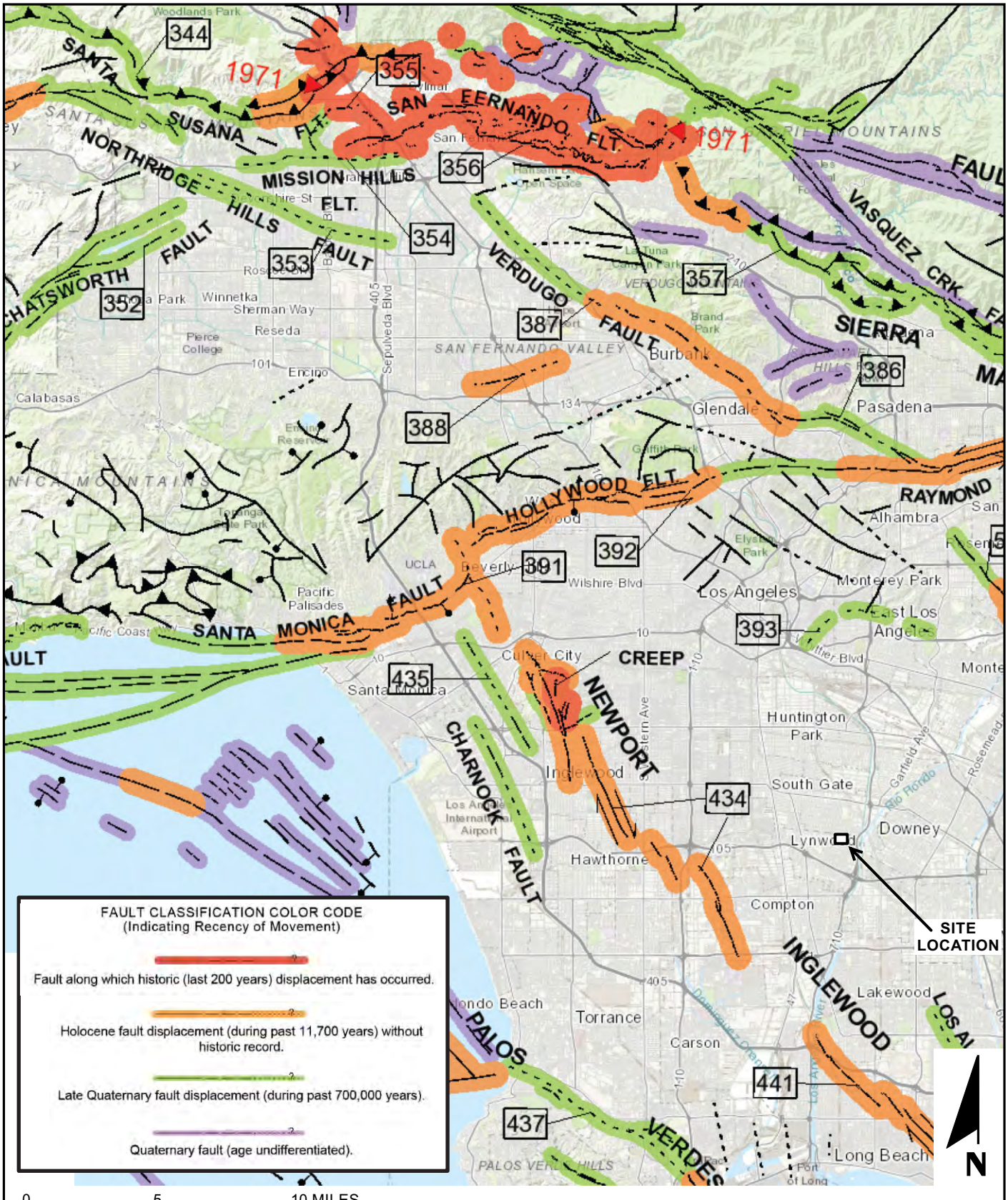
GPI GEOTECHNICAL PROFESSIONALS, INC.

LYNWOOD HIGH SCHOOL FIELD RENOVATION

GPI PROJECT NO. 3328.I | SCALE: 1" = 4000'

GROUNDWATER MAP

FIGURE D-2



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

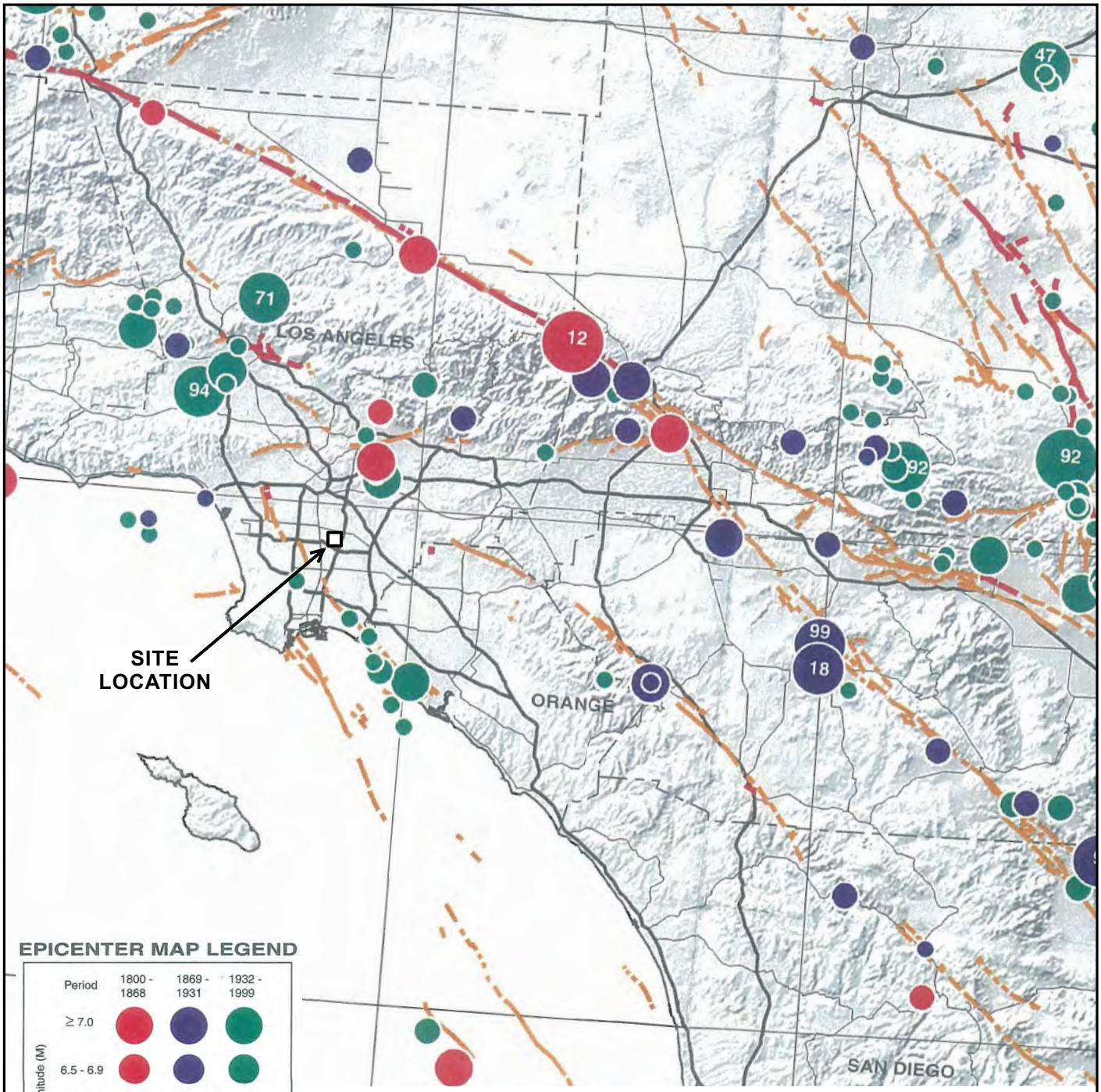
GPI GEOTECHNICAL PROFESSIONALS, INC.

REGIONAL FAULT MAP

LYNWOOD HIGH SCHOOL FIELD RENOVATION

GPI PROJECT NO. 3328.I | SCALE: 1" = 5 MILES

FIGURE D-3



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

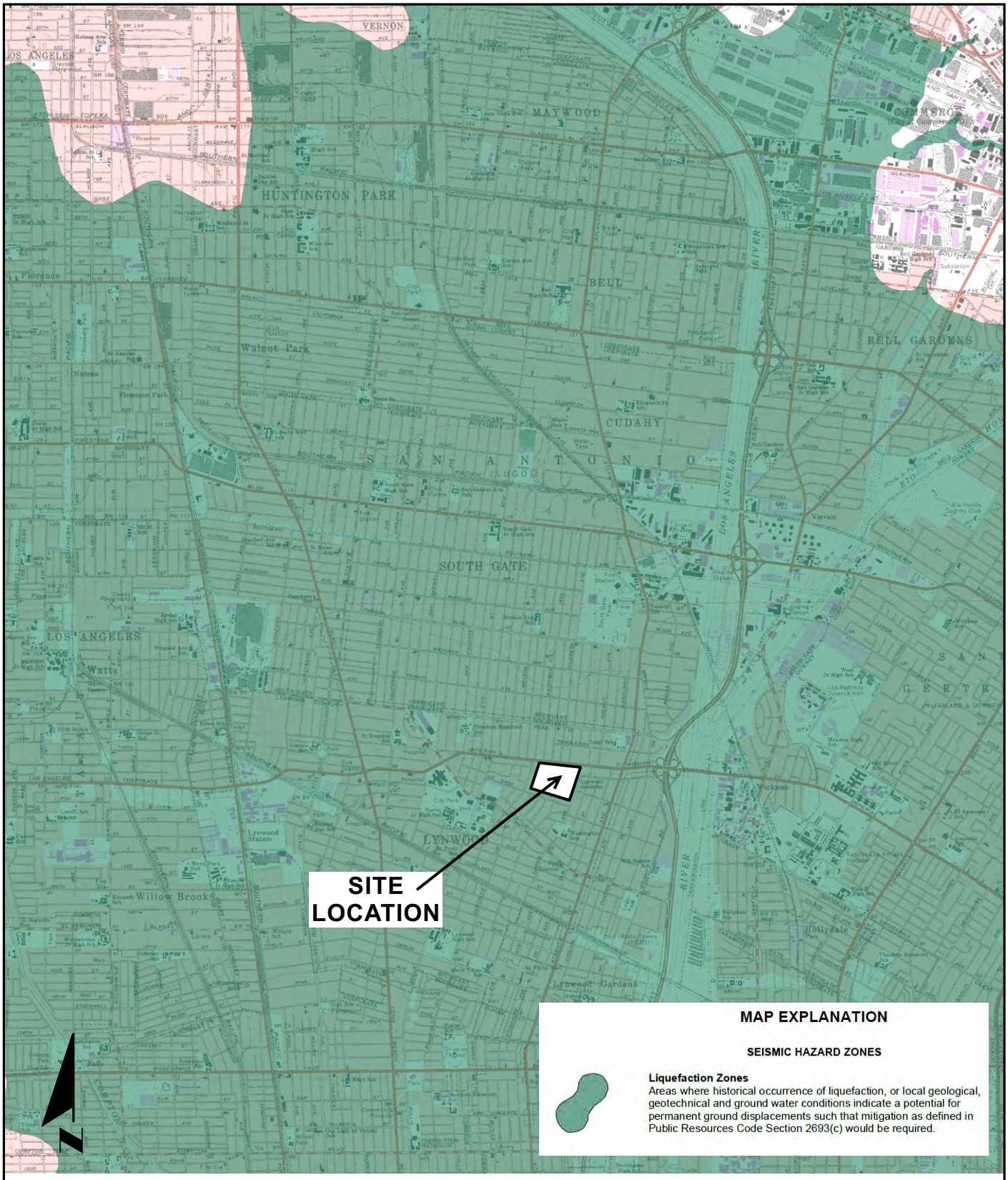
GPI GEOTECHNICAL PROFESSIONALS, INC.

LYNWOOD HIGH SCHOOL FIELD RENOVATION

GPI PROJECT NO. 3328.I | SCALE: 1" = 20 MILES

REGIONAL SEISMICITY

FIGURE D-4



BASE MAP REPRODUCED FROM SEISMIC HAZARD ZONES MAP FOR THE SOUTH GATE QUADRANGLE PREPARED BY THE STATE OF CALIFORNIA DIVISION OF MINES AND GEOLOGY: DATED 03/25/1999

GEOTECHNICAL PROFESSIONALS, INC.

SEISMIC HAZARD MAP

LYNWOOD HIGH SCHOOL FIELD RENOVATION

GPI PROJECT NO. 3328.I | SCALE: 1" = 6000'

FIGURE D-5

APPENDIX E

Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

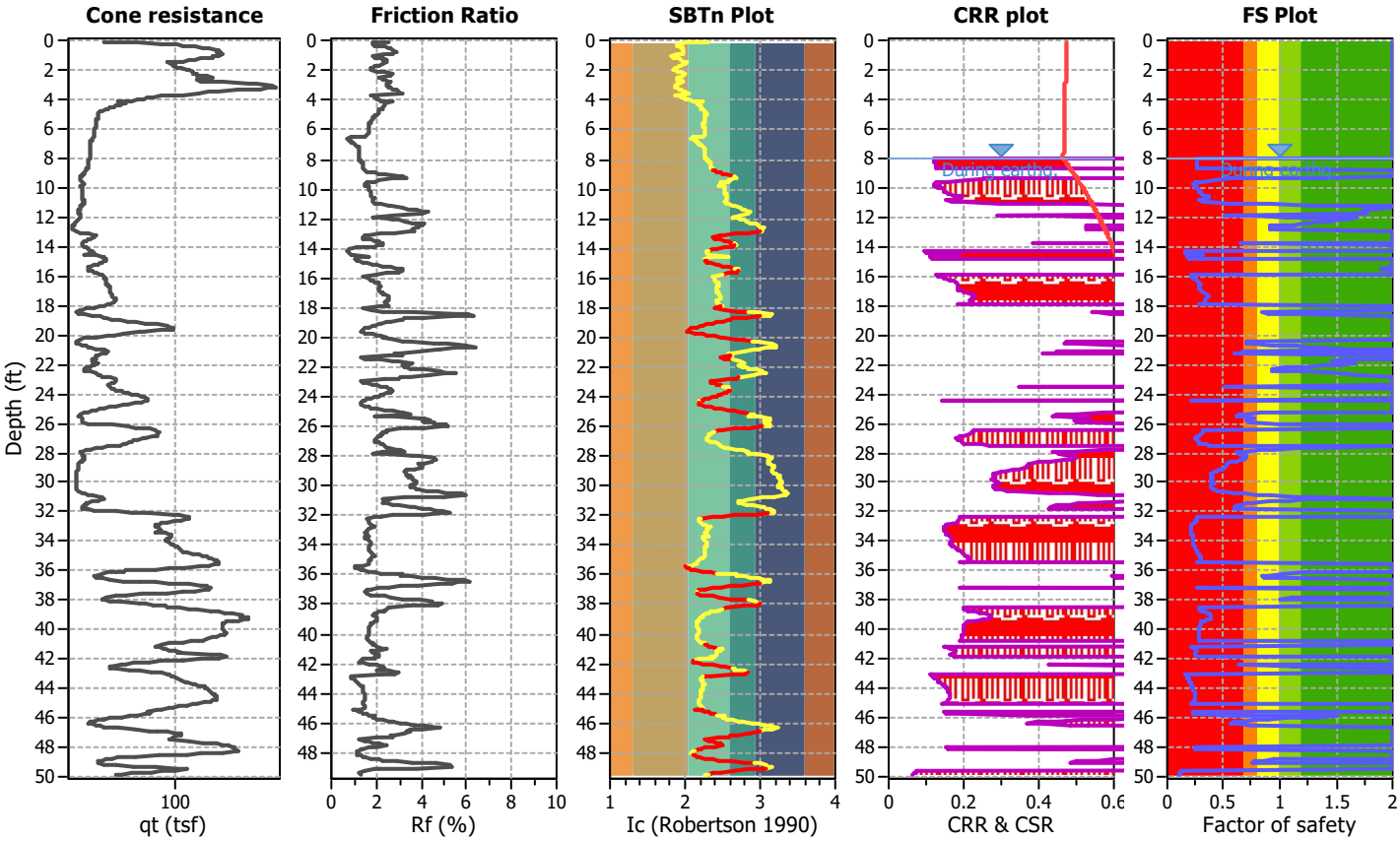
LIQUEFACTION ANALYSIS REPORT

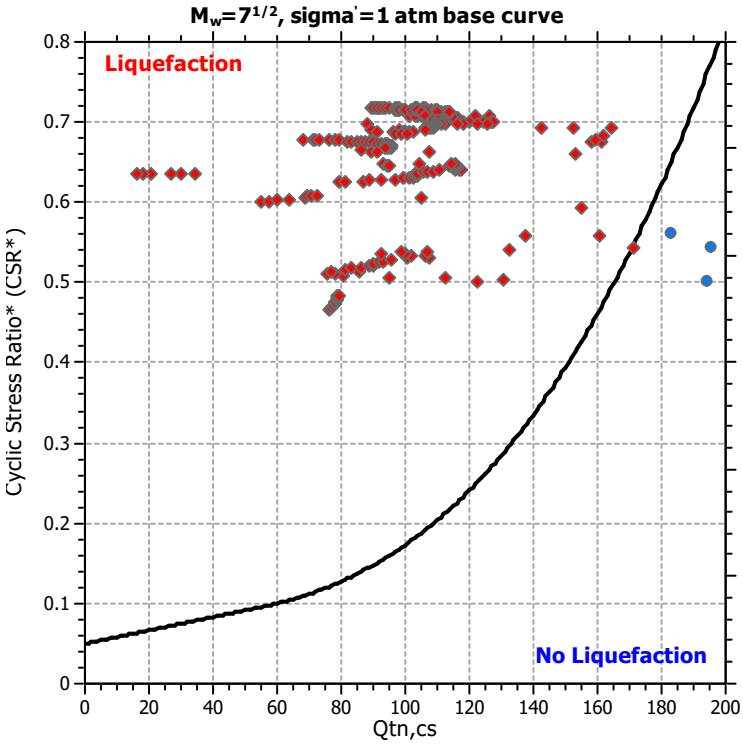
Project title : GPI / Lynwood High School **Location : 4050 E. Imperial Highway, Lynwood, CA**

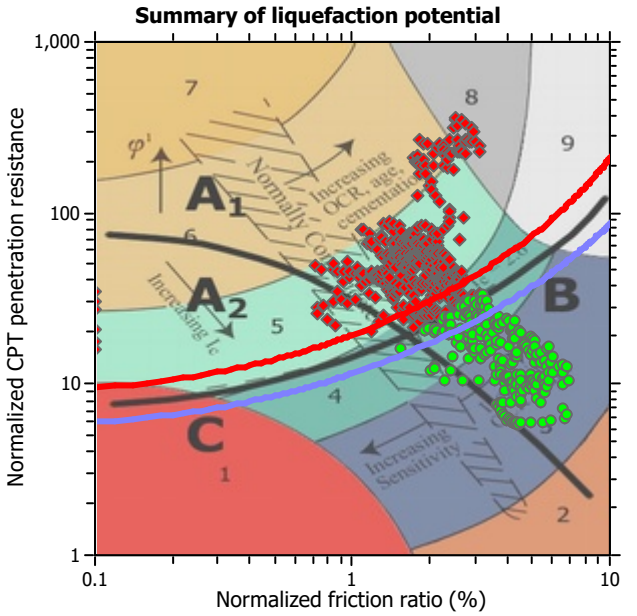
CPT file : C-5

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	45.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	8.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth:	60.00 ft
Earthquake magnitude M_w:	7.30	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.78	Unit weight calculation:	Based on SBT	K_σ applied:	No		

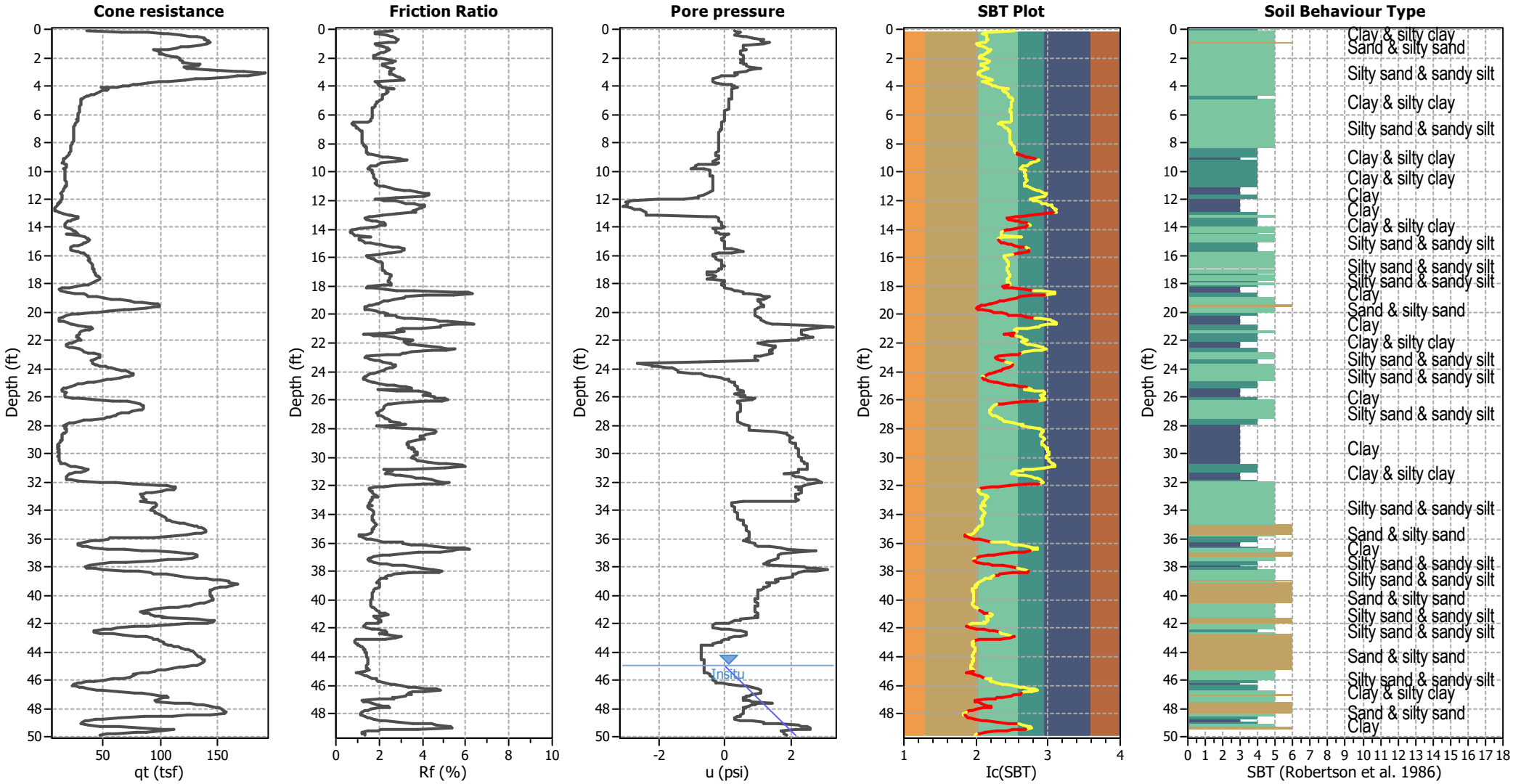






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots



Input parameters and analysis data

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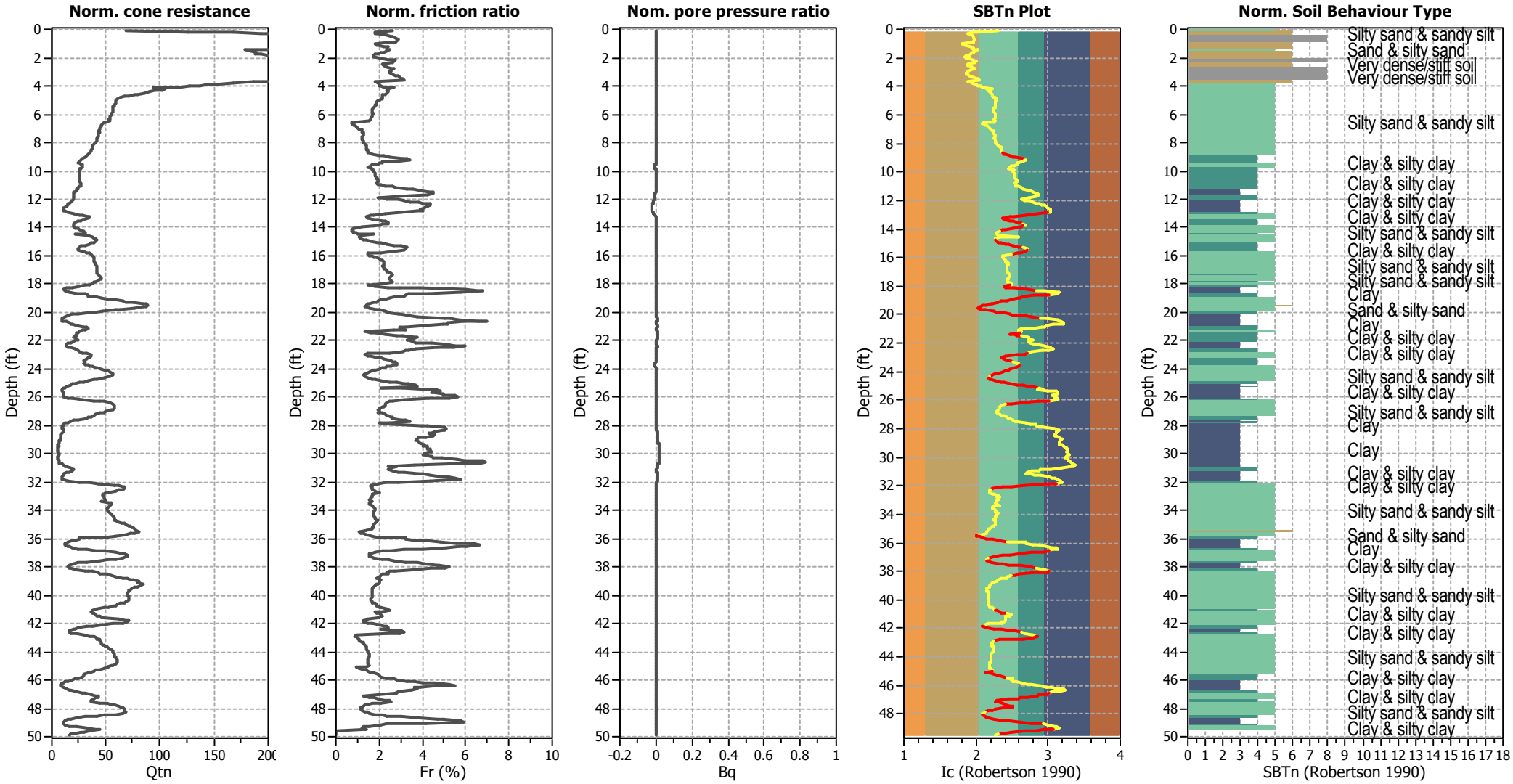
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.78	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

SBT legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

CPT basic interpretation plots (normalized)



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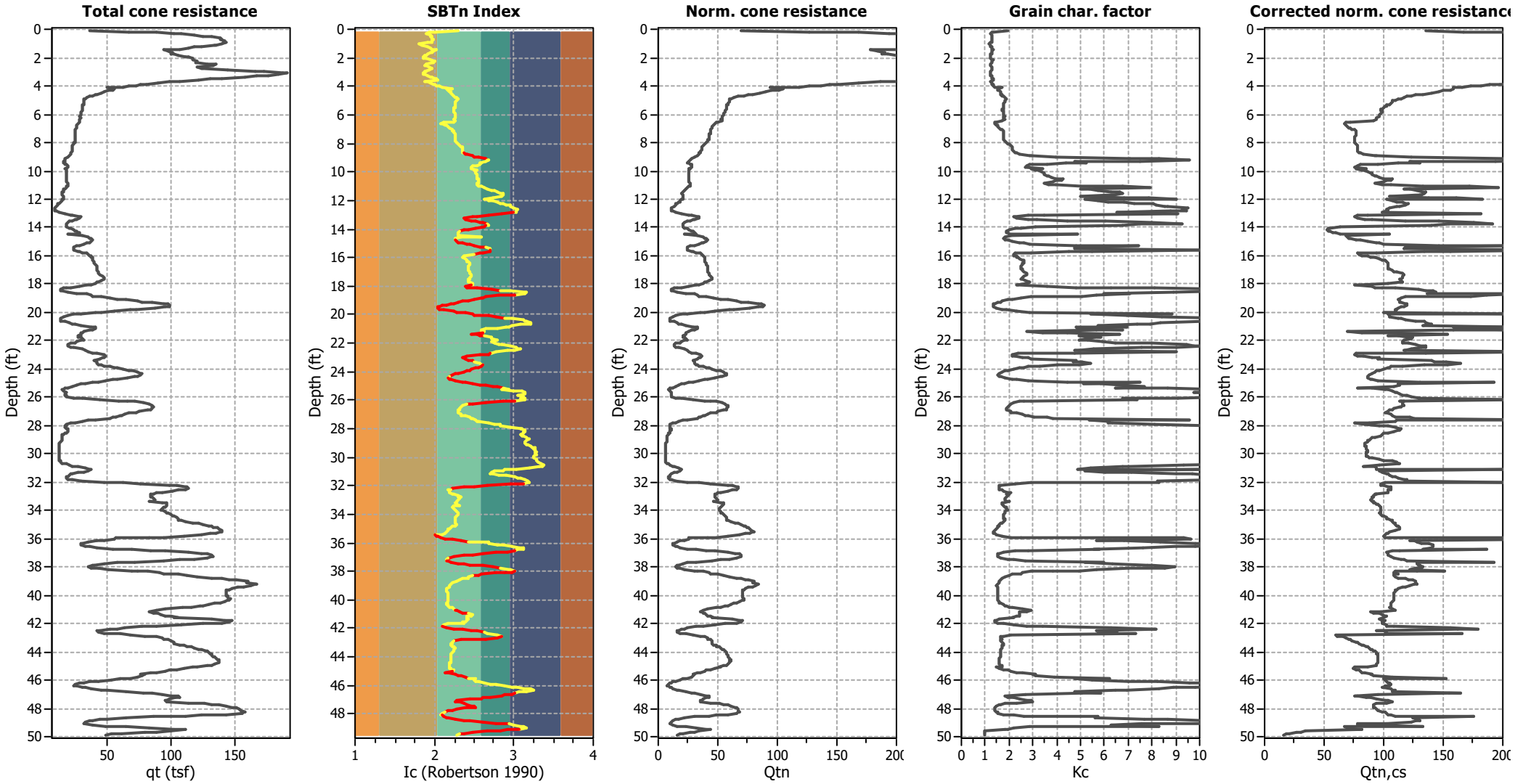
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.78	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

SBTn legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

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Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
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Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.78	Use fill:	No	Limit depth applied:	Yes
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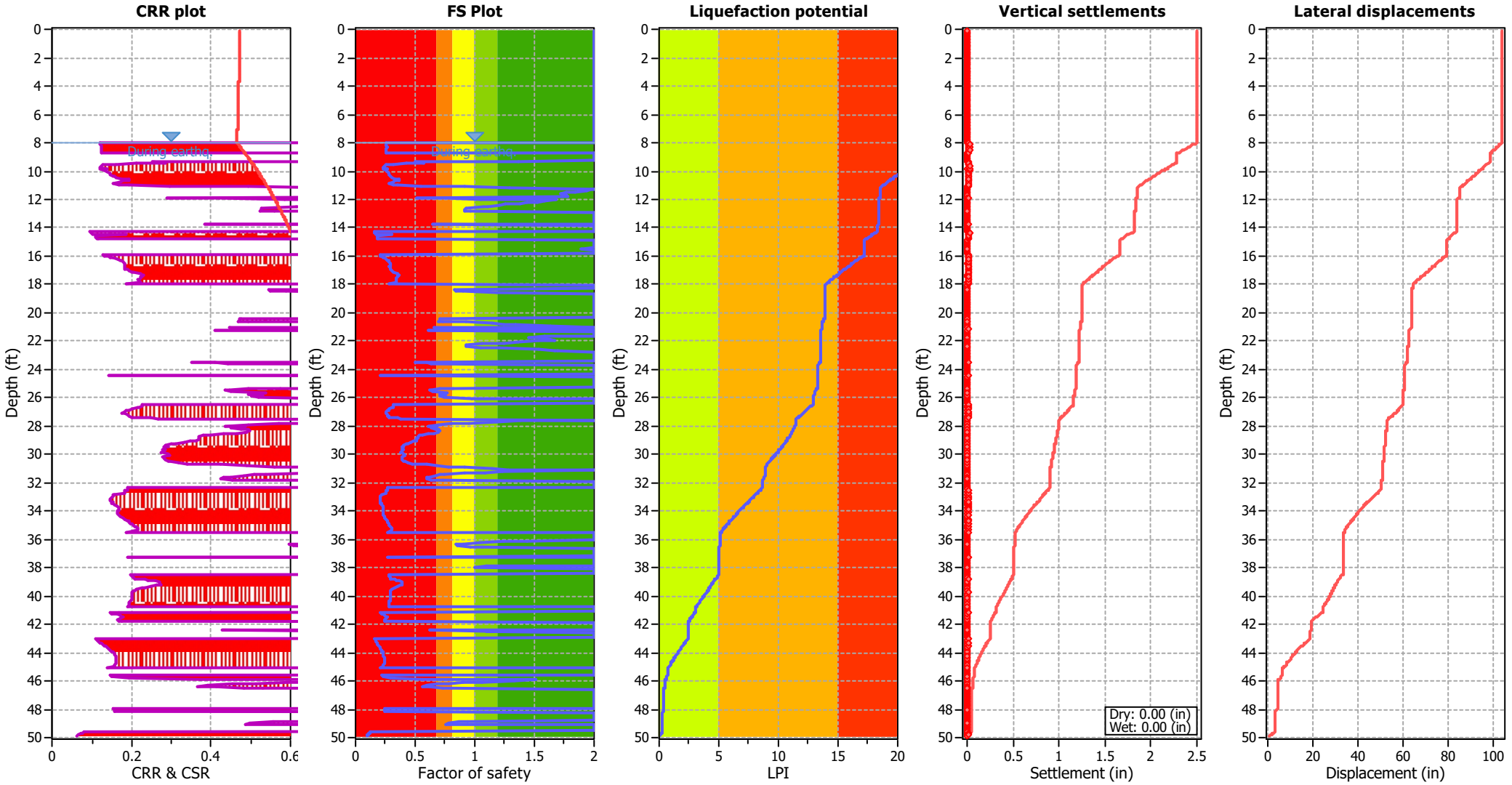
Liquefaction analysis overall plots



Input parameters and analysis data

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Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.78	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

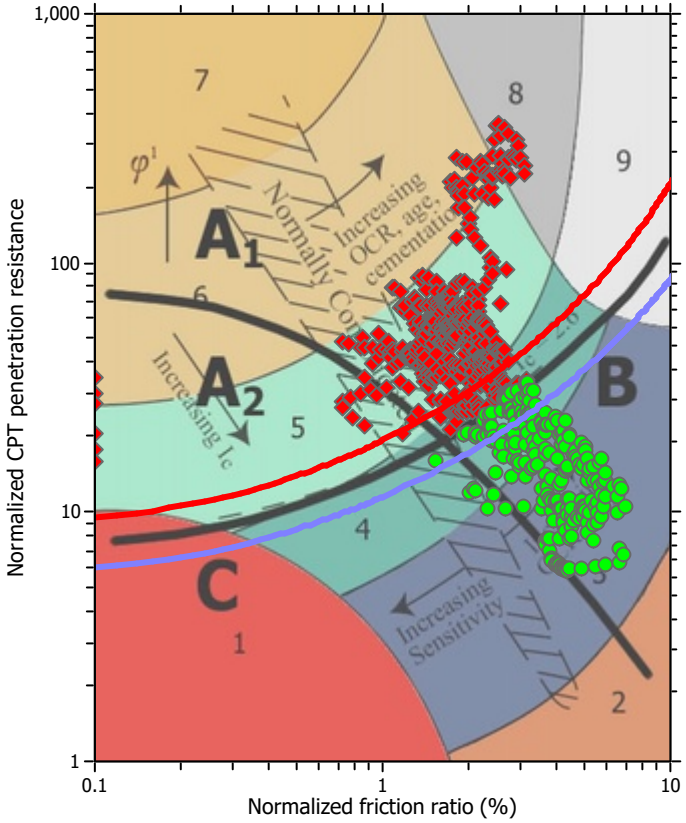
F.S. color scheme

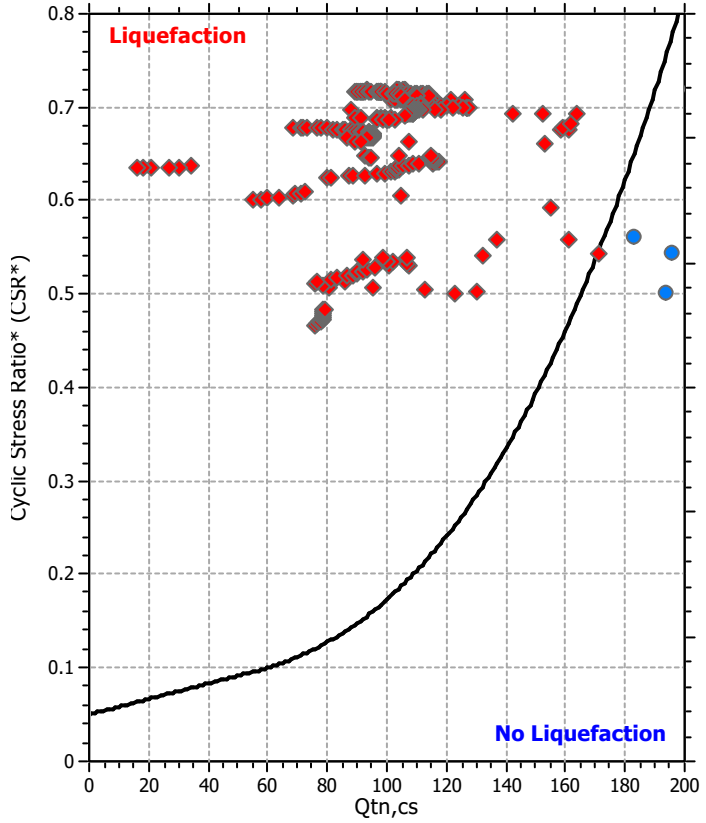
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

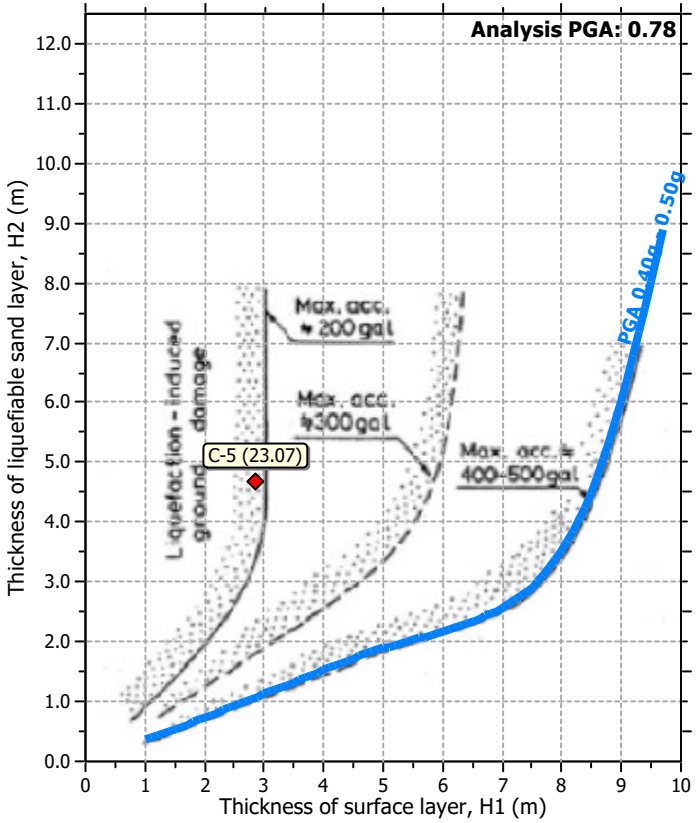
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots







Input parameters and analysis data

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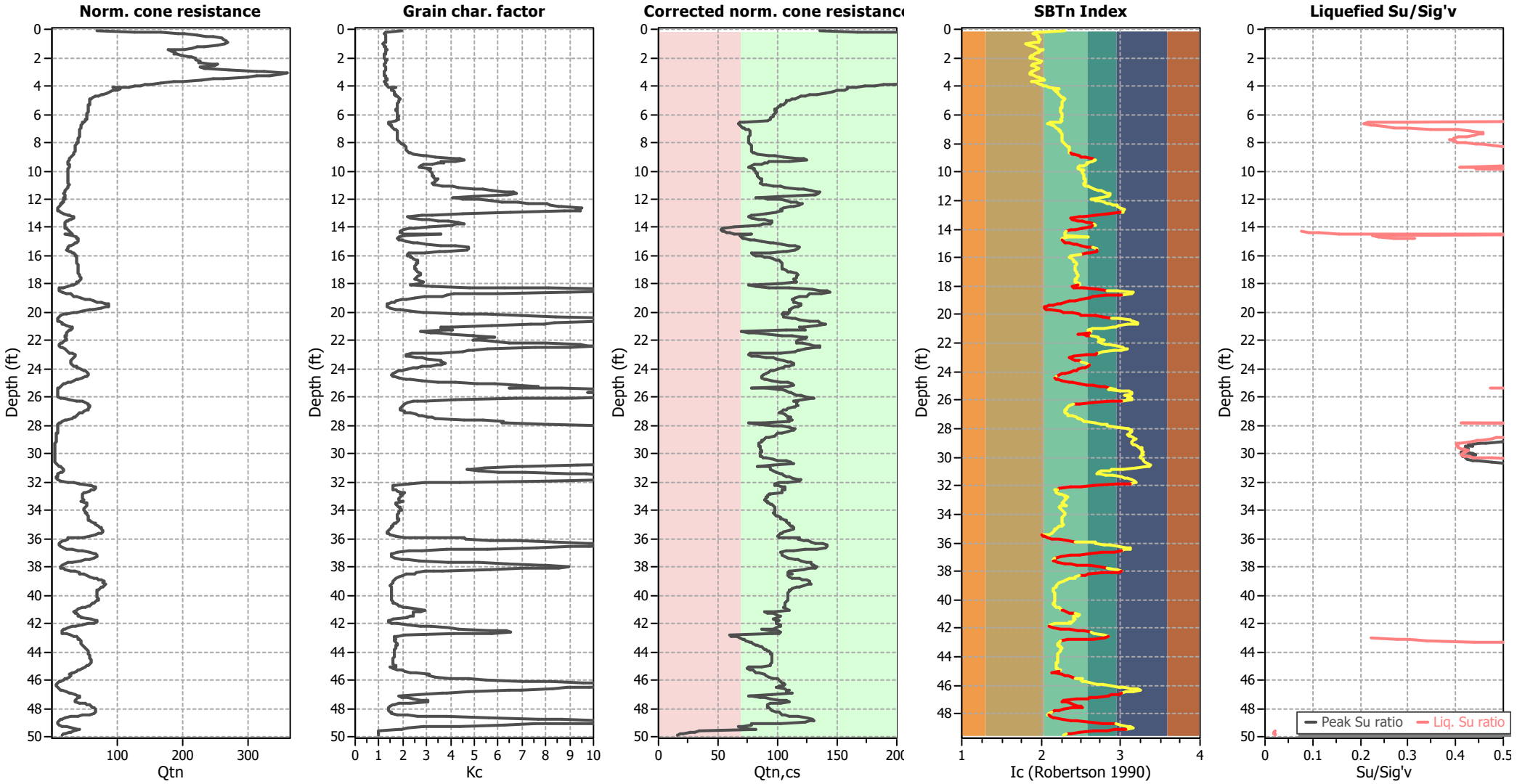
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.78	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

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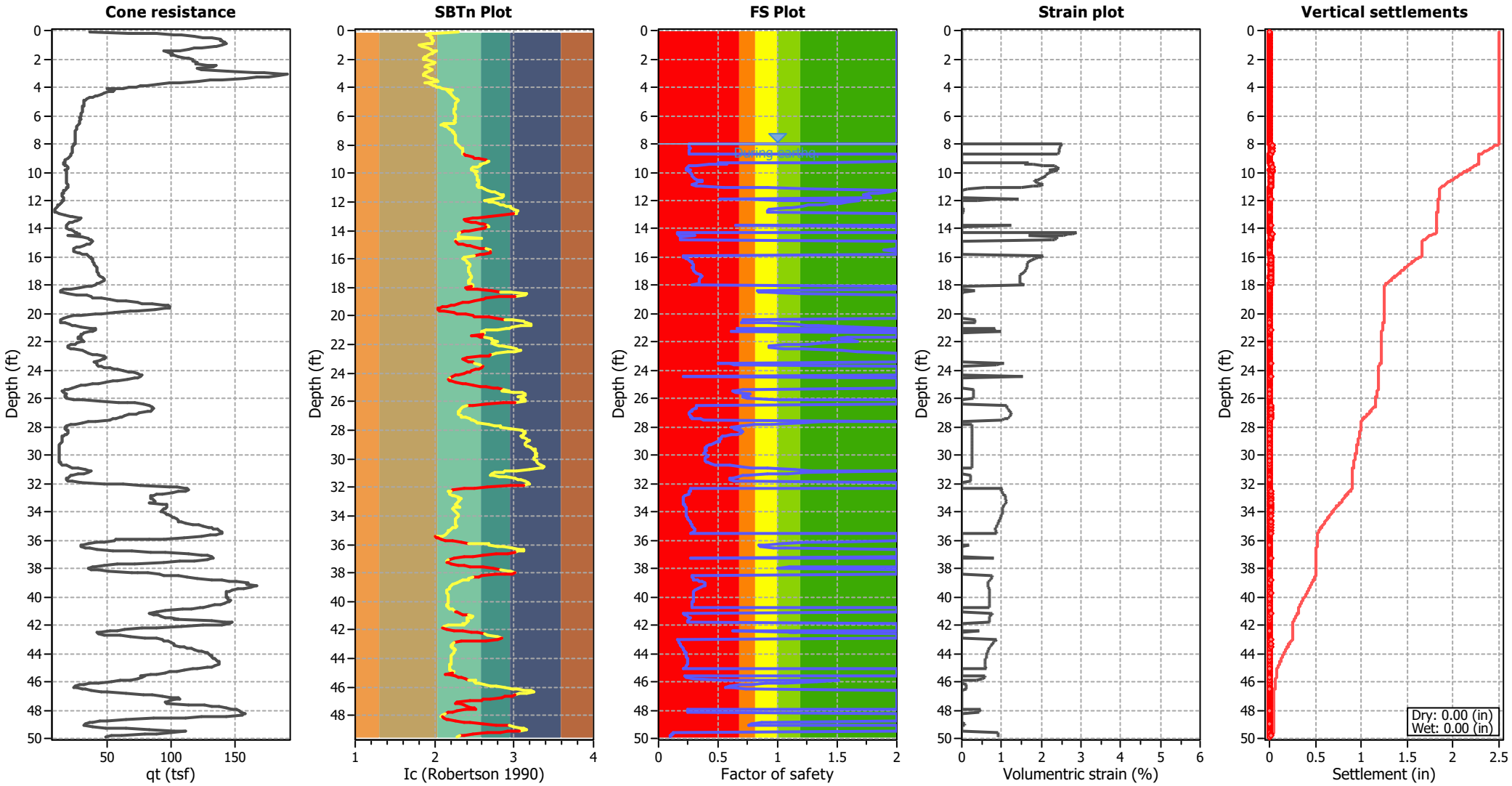
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.78	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

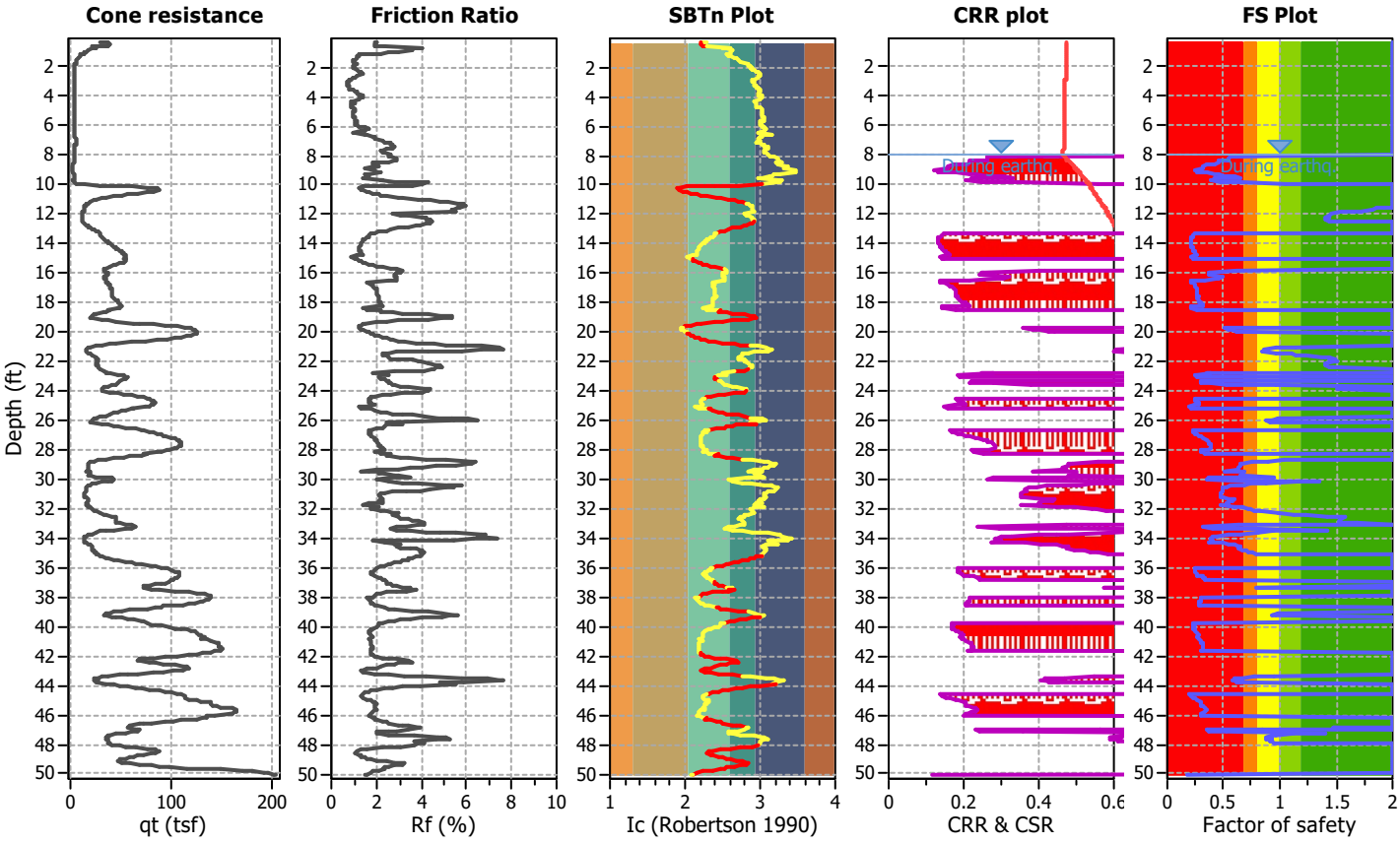
LIQUEFACTION ANALYSIS REPORT

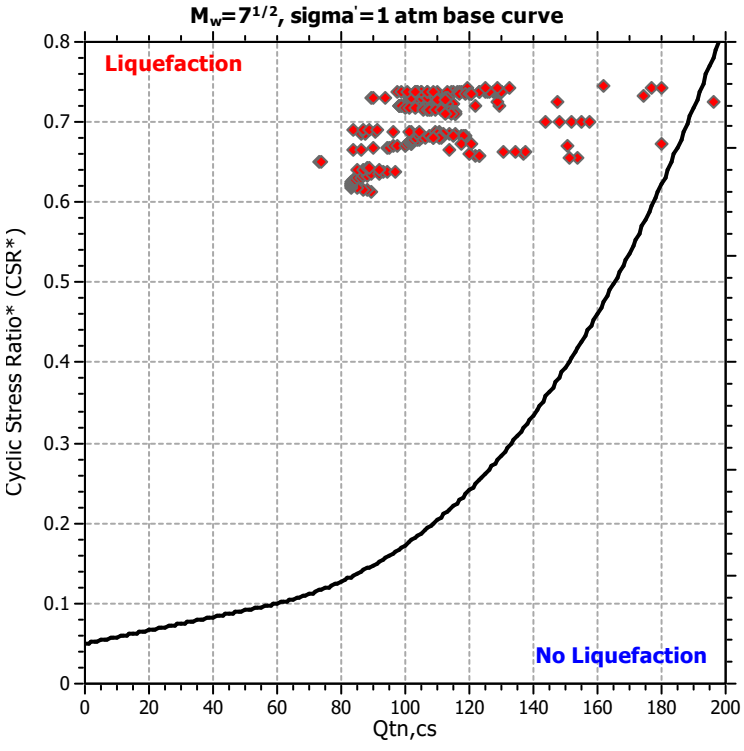
Project title : GPI / Lynwood High School **Location : 4050 E. Imperial Highway, Lynwood, CA**

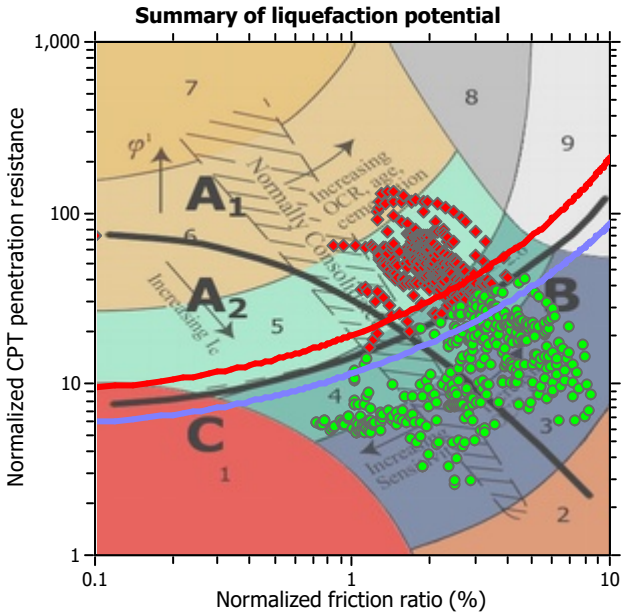
CPT file : C-6

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	45.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	8.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth:	60.00 ft
Earthquake magnitude M_w:	7.30	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.78	Unit weight calculation:	Based on SBT	K_σ applied:	No		

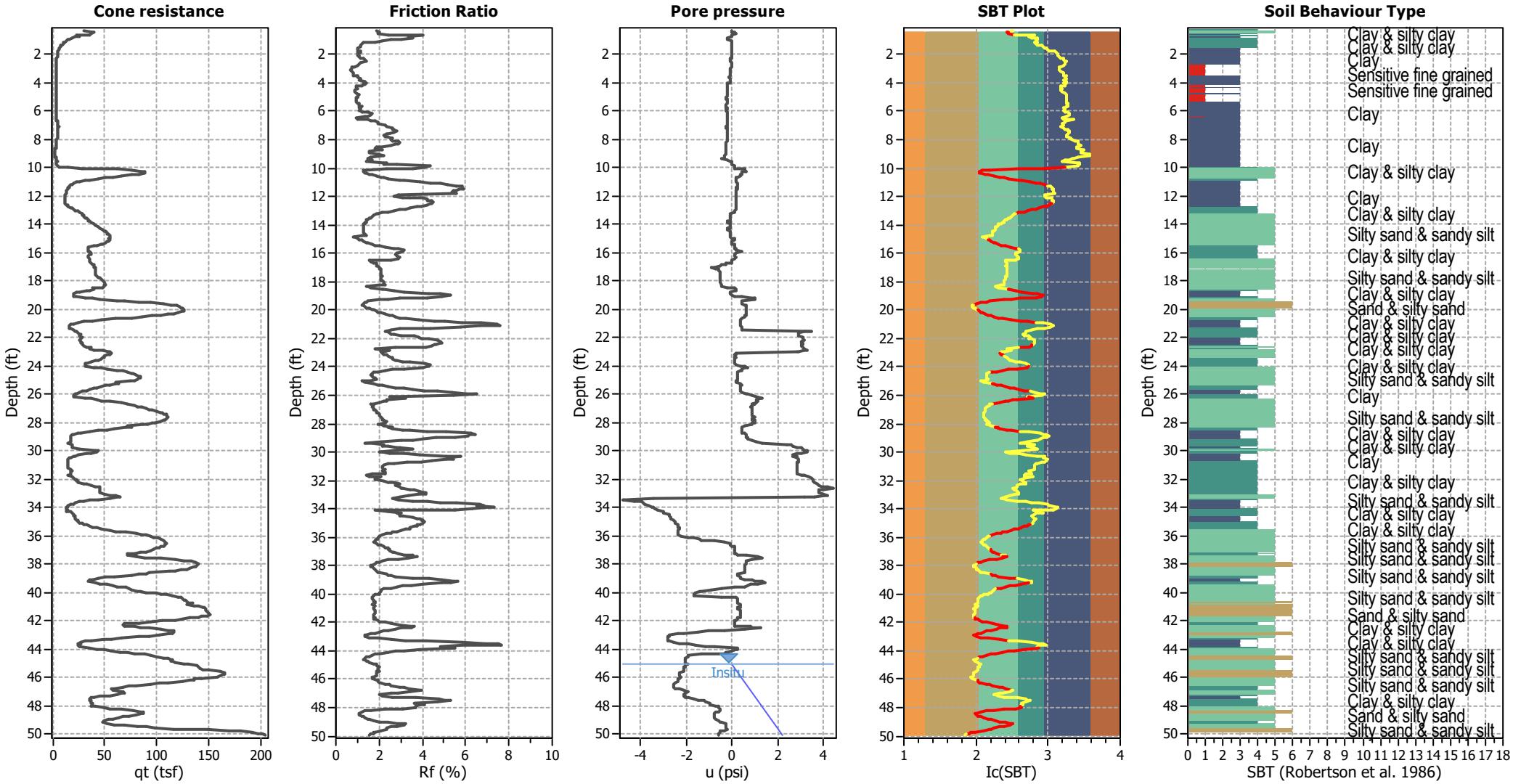






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots



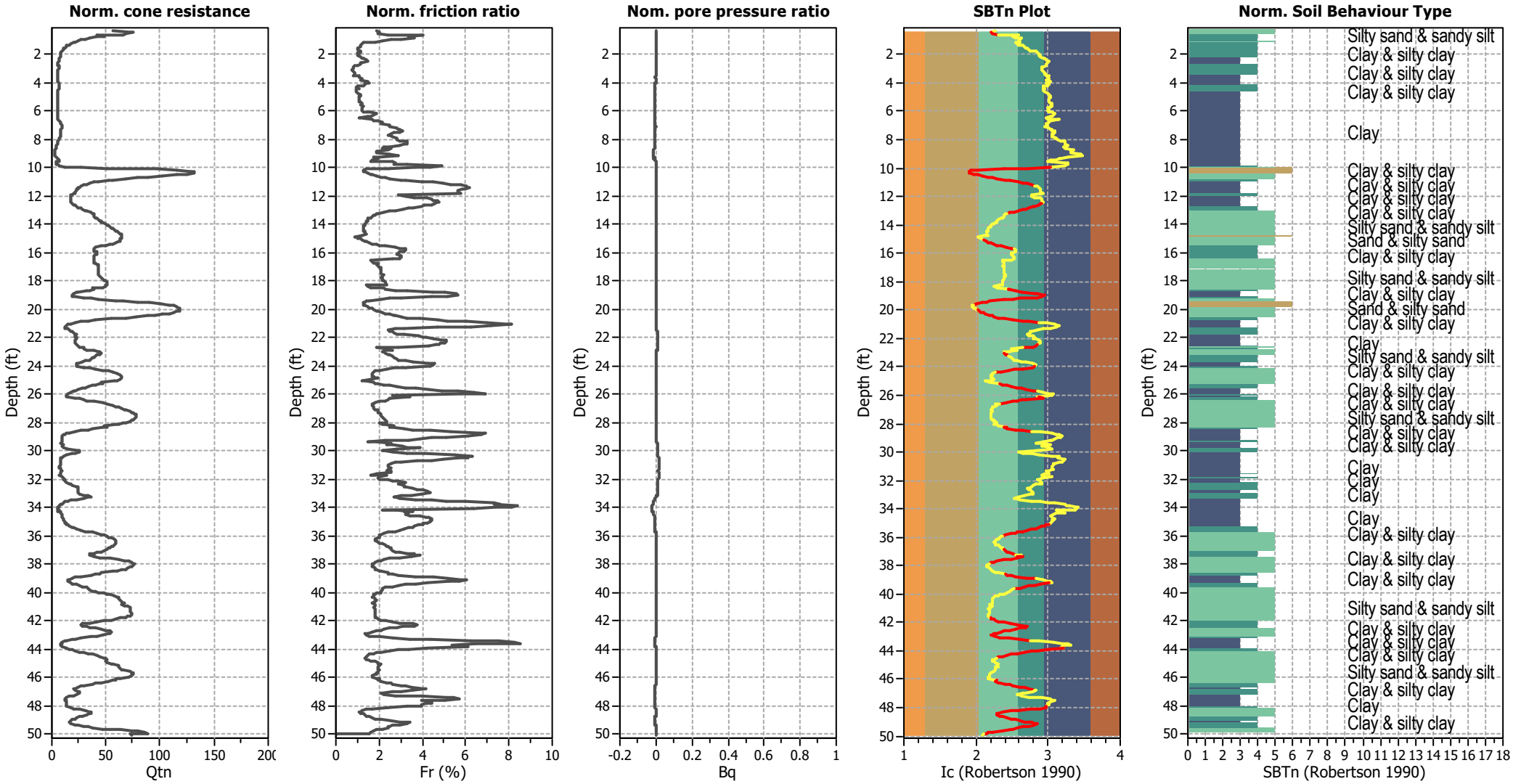
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.78	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

SBT legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

CPT basic interpretation plots (normalized)



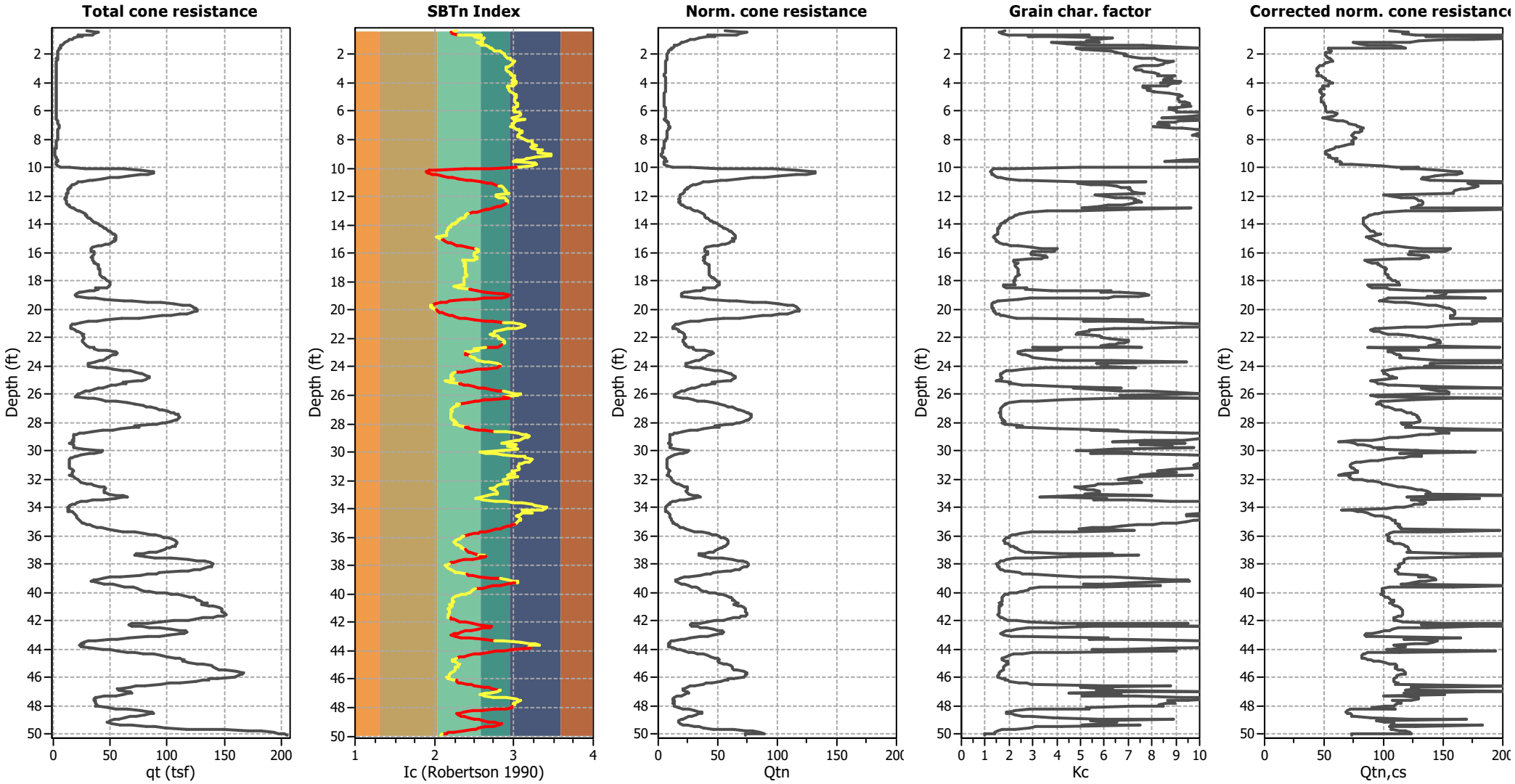
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.78	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

SBTn legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

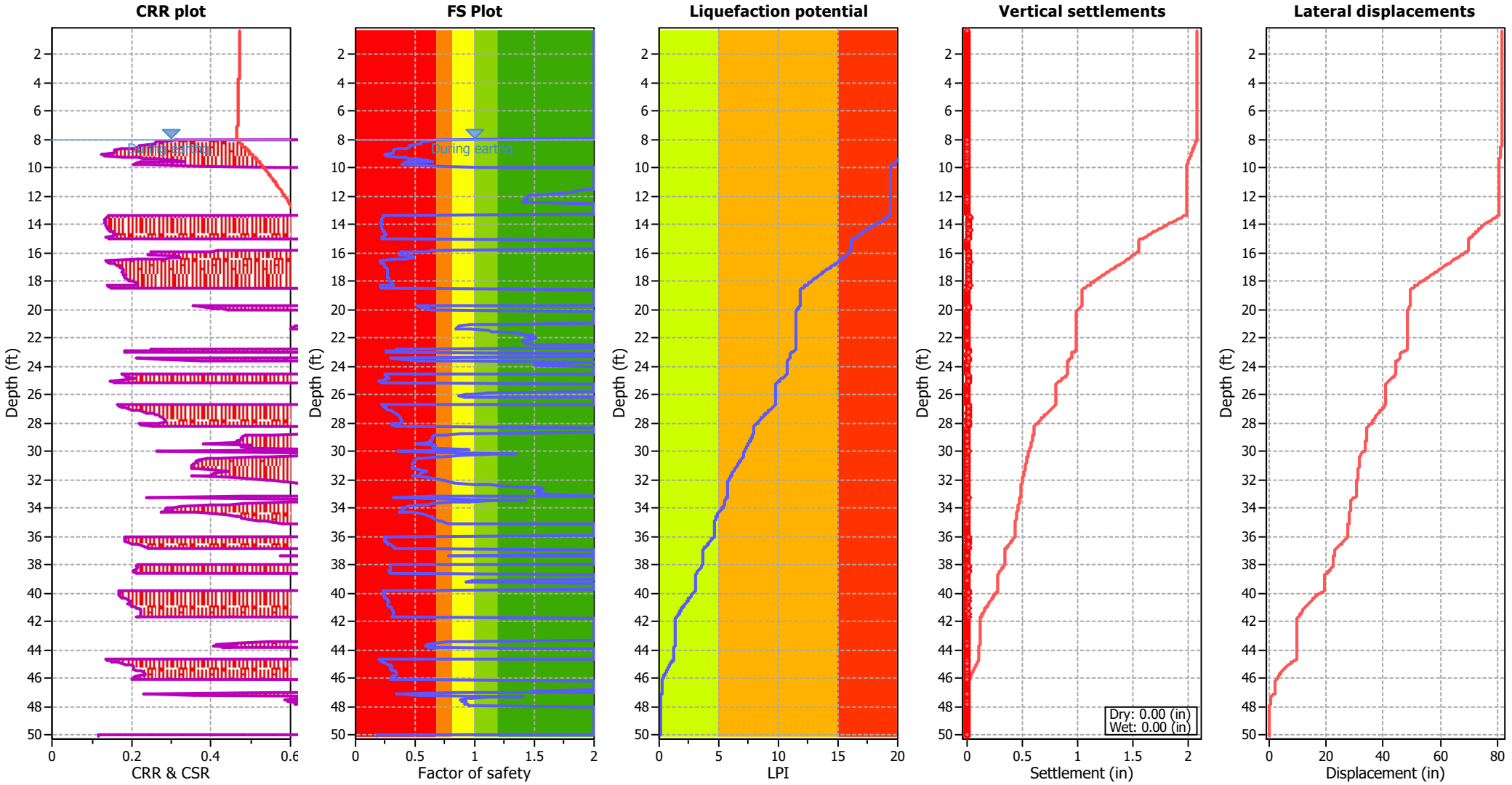
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.78	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.78	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

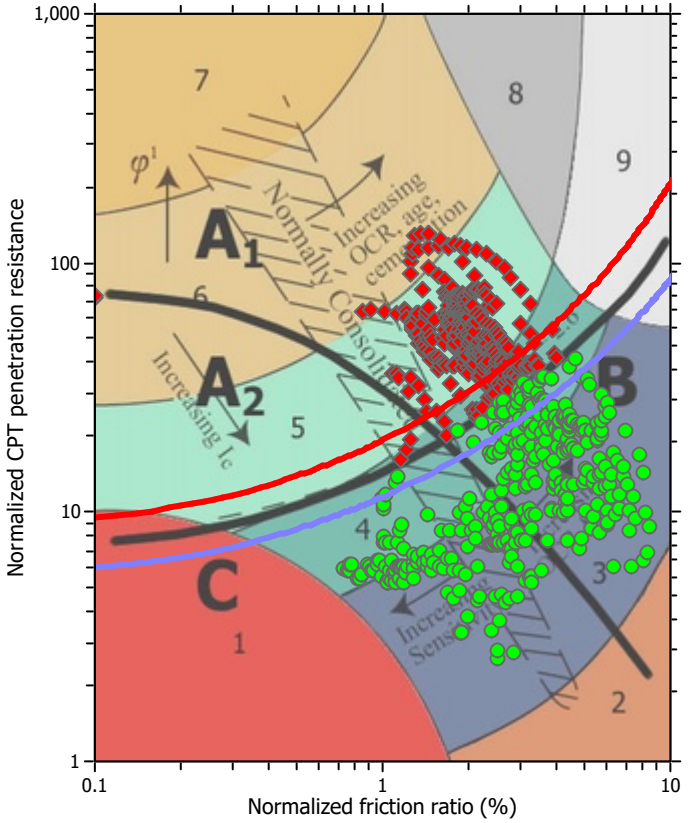
F.S. color scheme

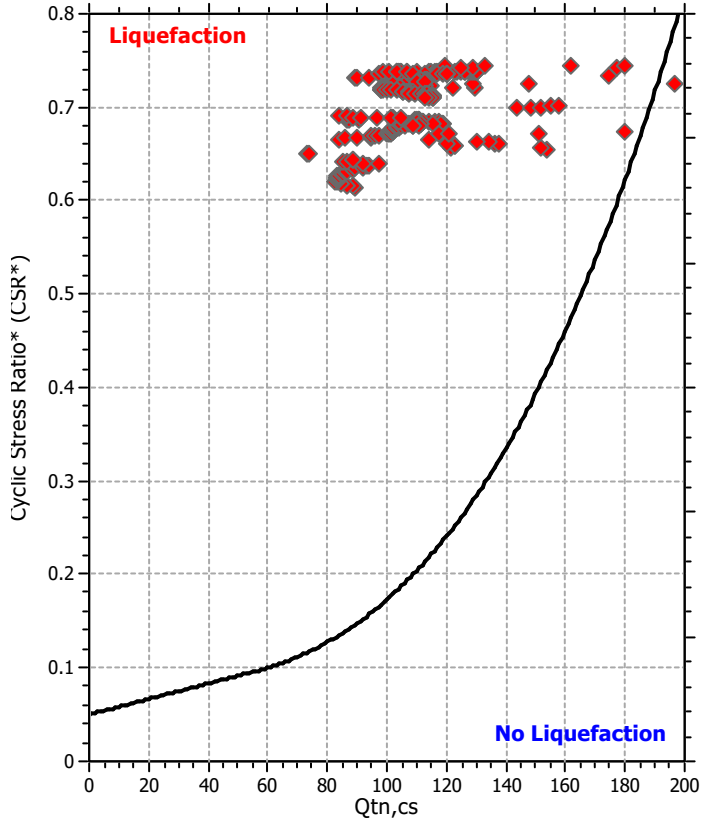
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

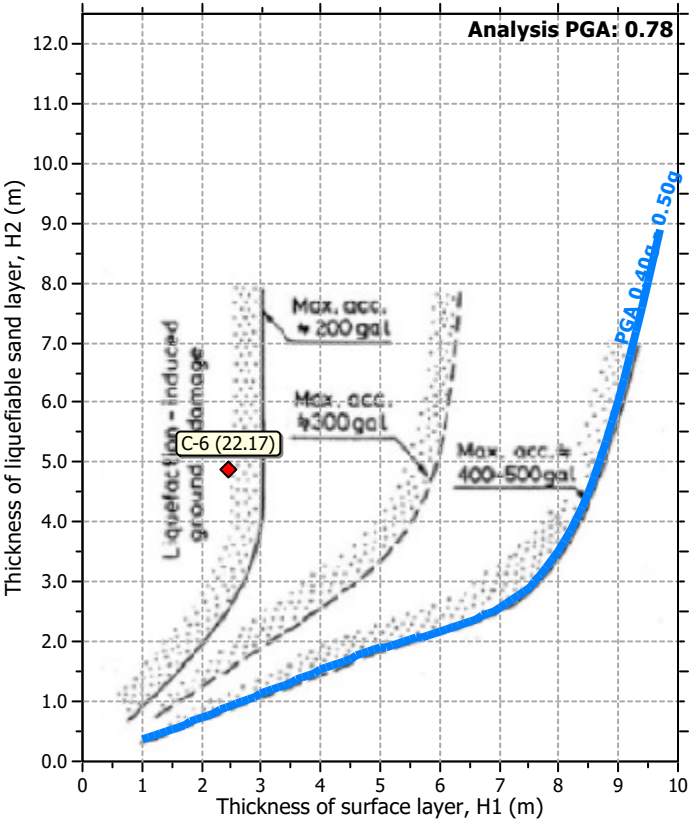
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots



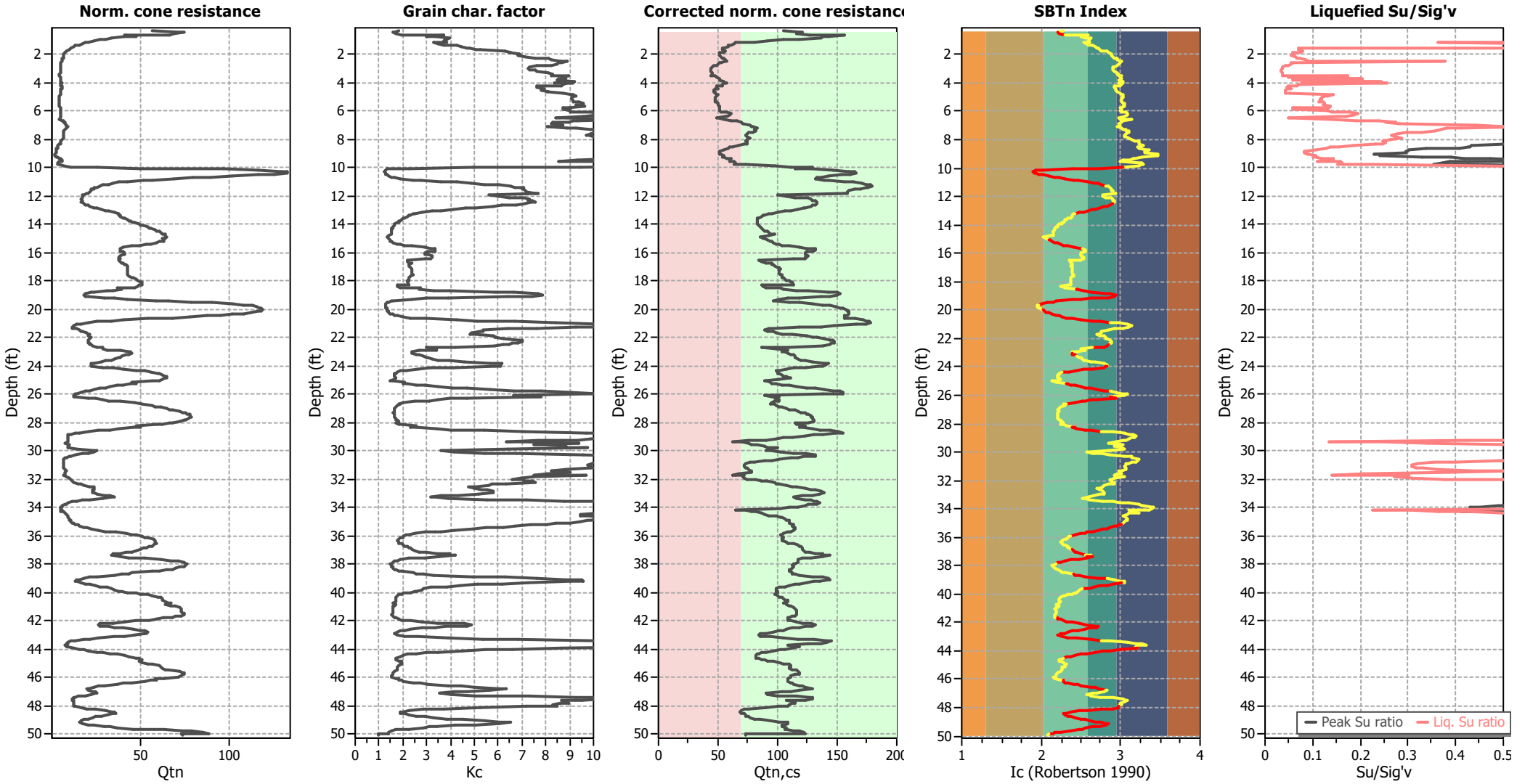




Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.78	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

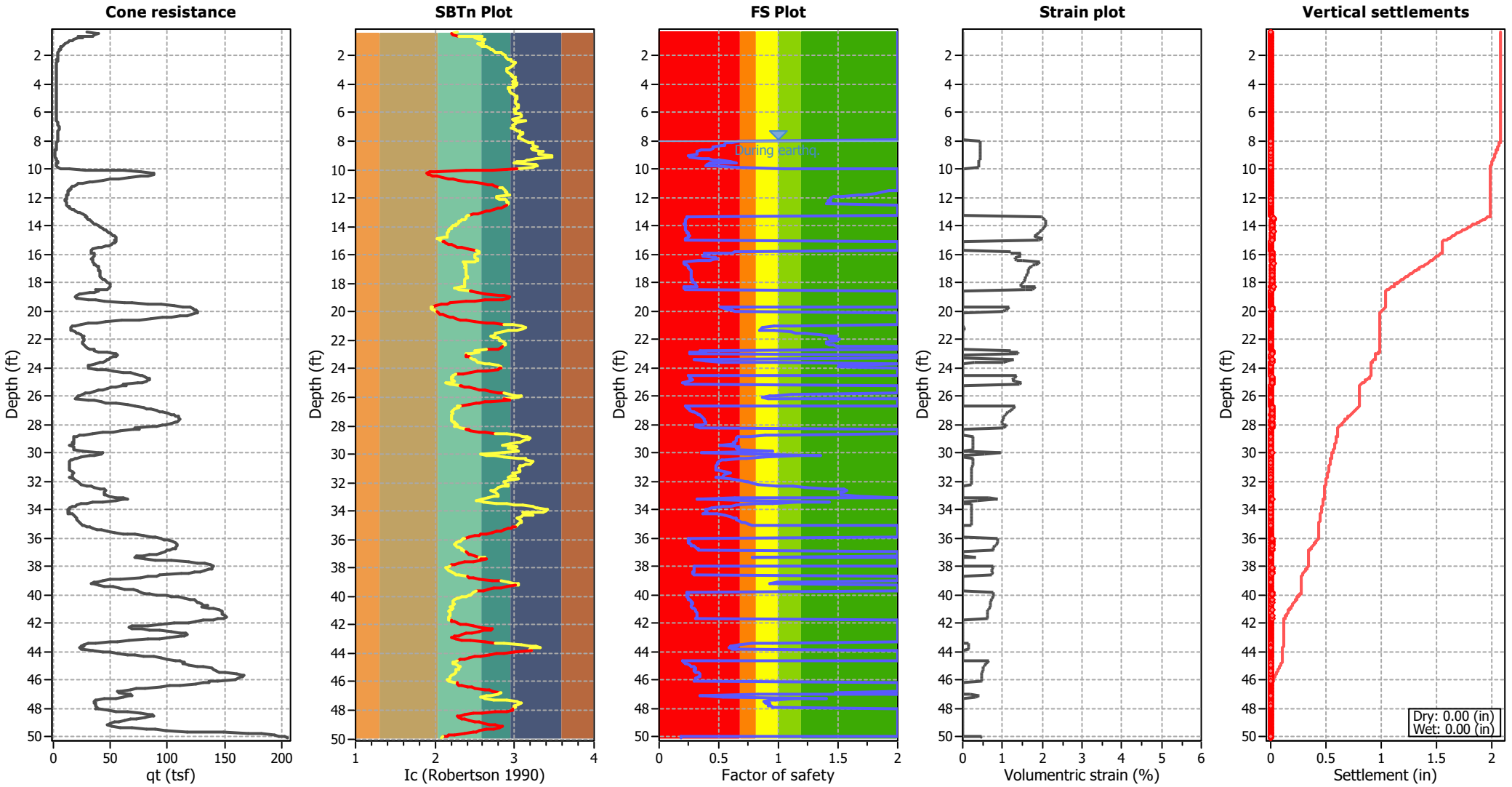
Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_o applied:	No
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.78	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

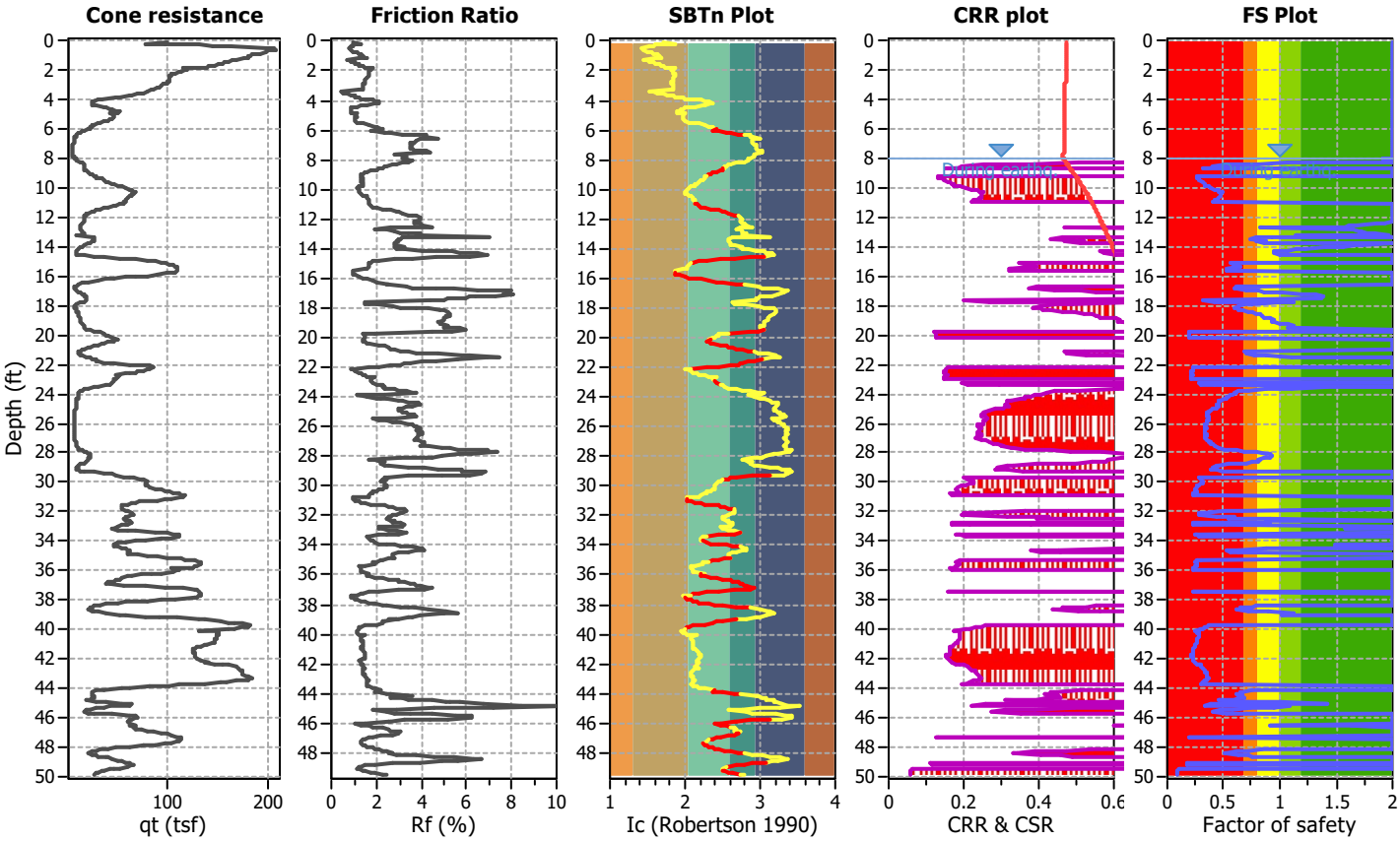
LIQUEFACTION ANALYSIS REPORT

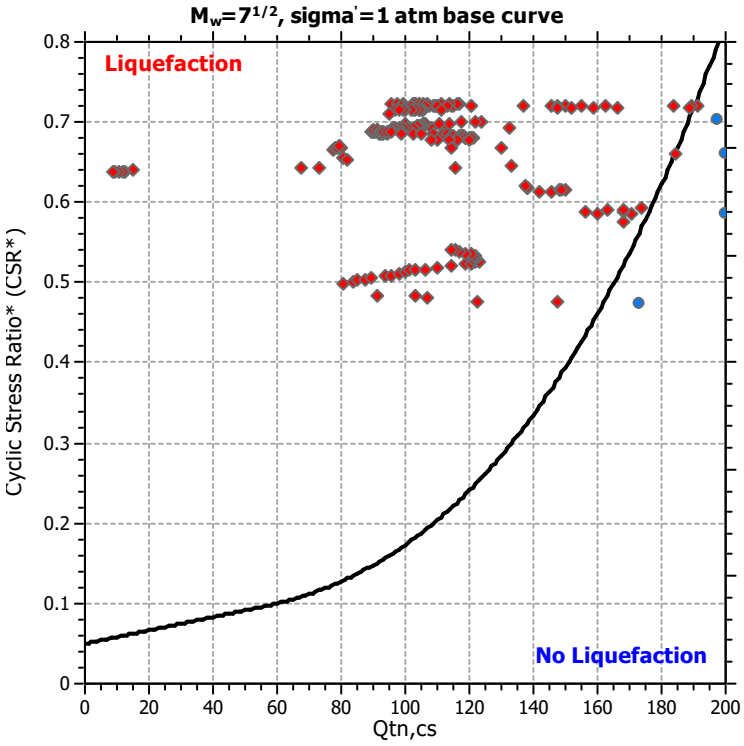
Project title : GPI / Lynwood High School **Location : 4050 E. Imperial Highway, Lynwood, CA**

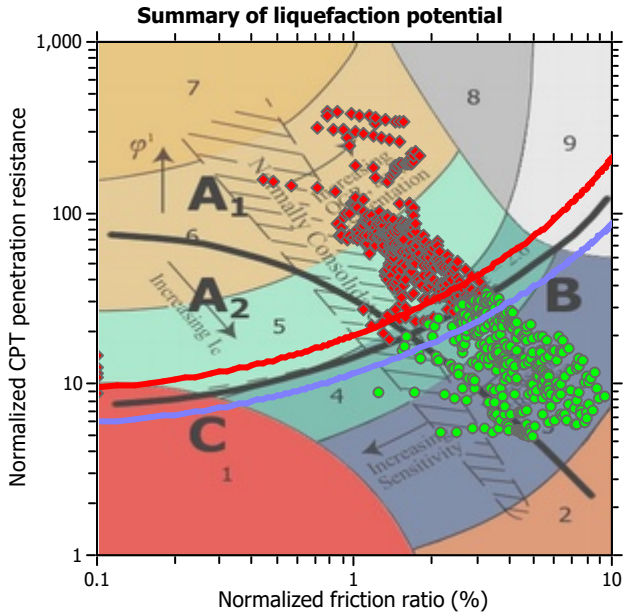
CPT file : C-7

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	45.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	8.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth:	60.00 ft
Earthquake magnitude M_w:	7.30	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.78	Unit weight calculation:	Based on SBT	K_σ applied:	No		

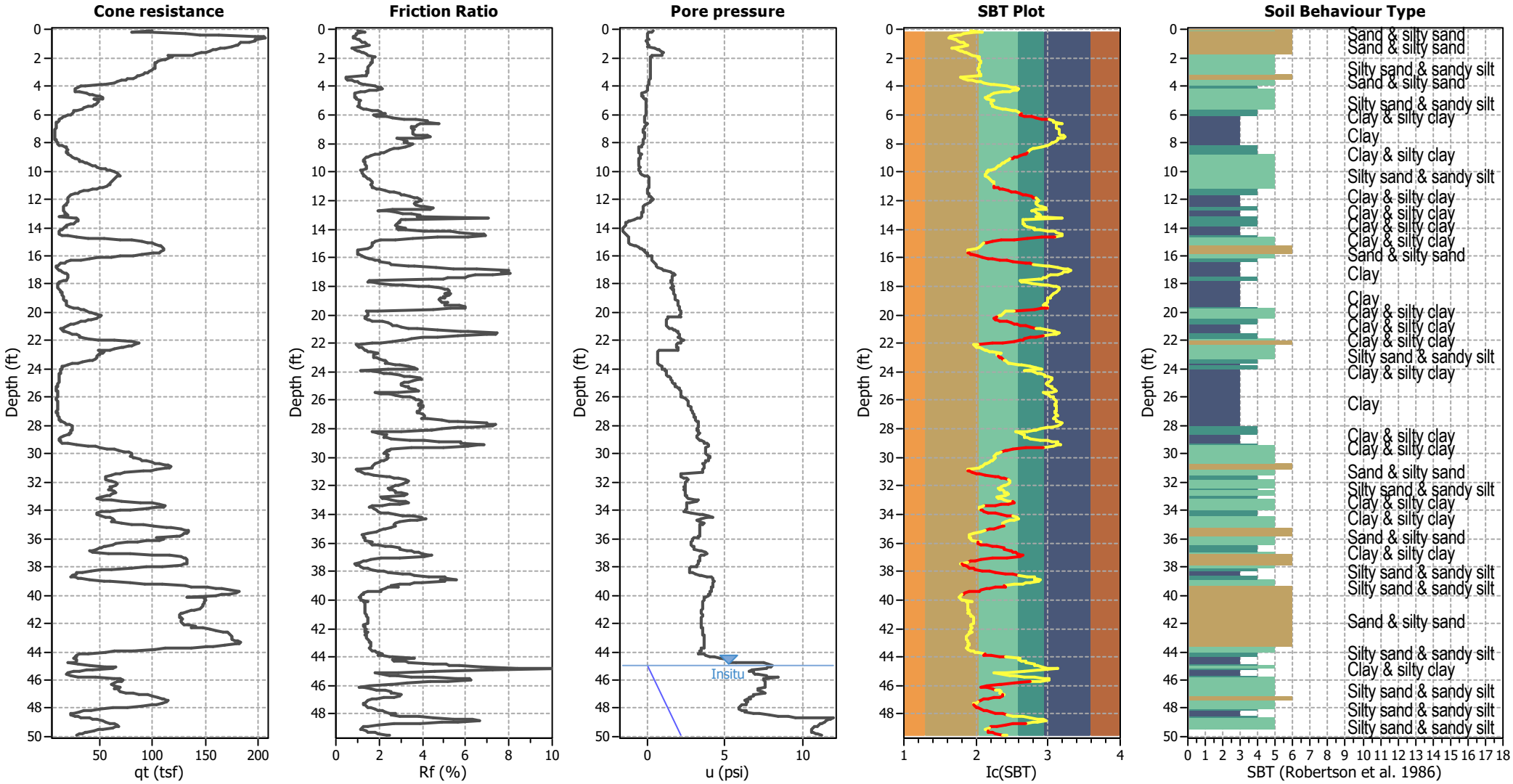






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots



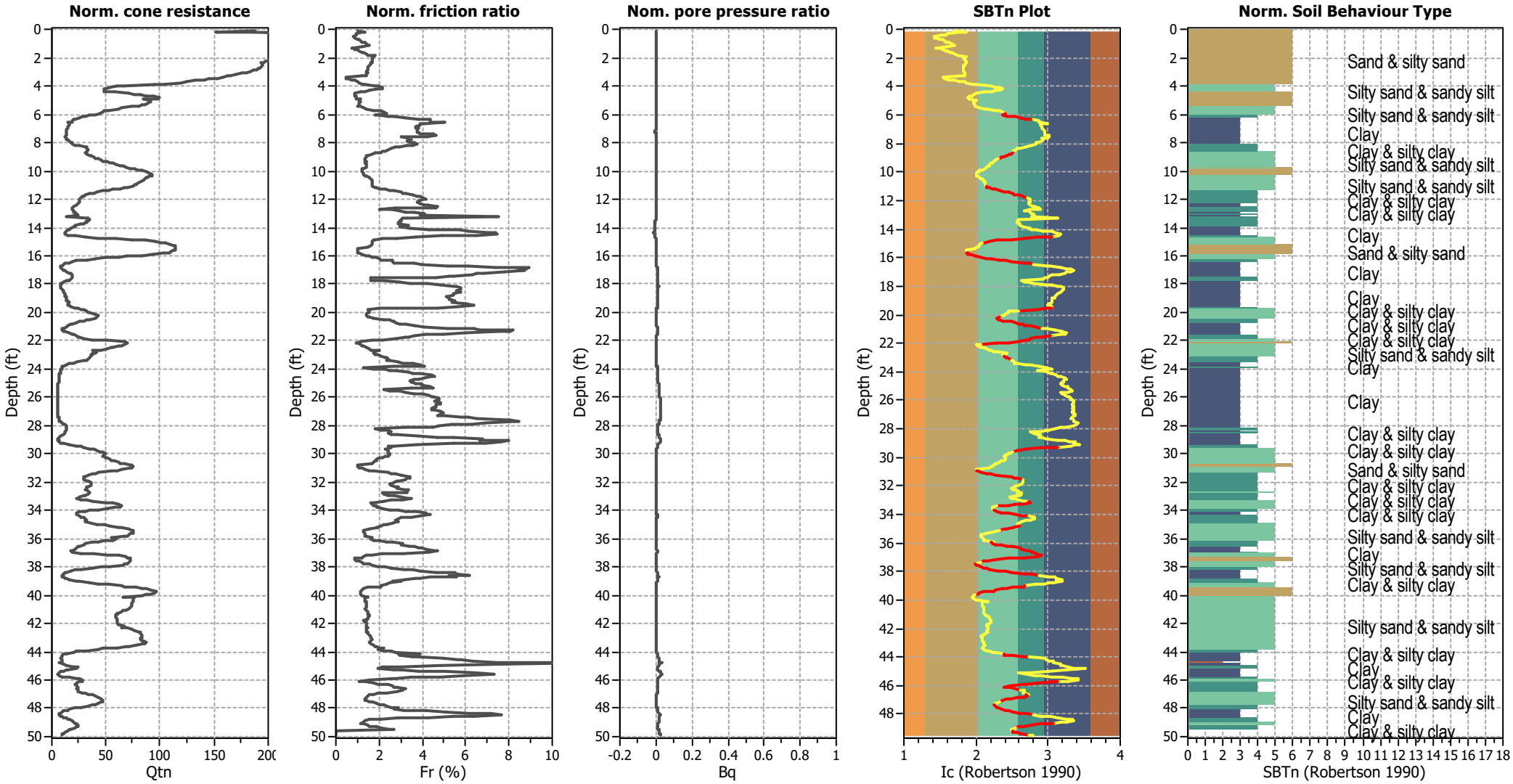
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_o applied:	No
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.78	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

SBT legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

CPT basic interpretation plots (normalized)



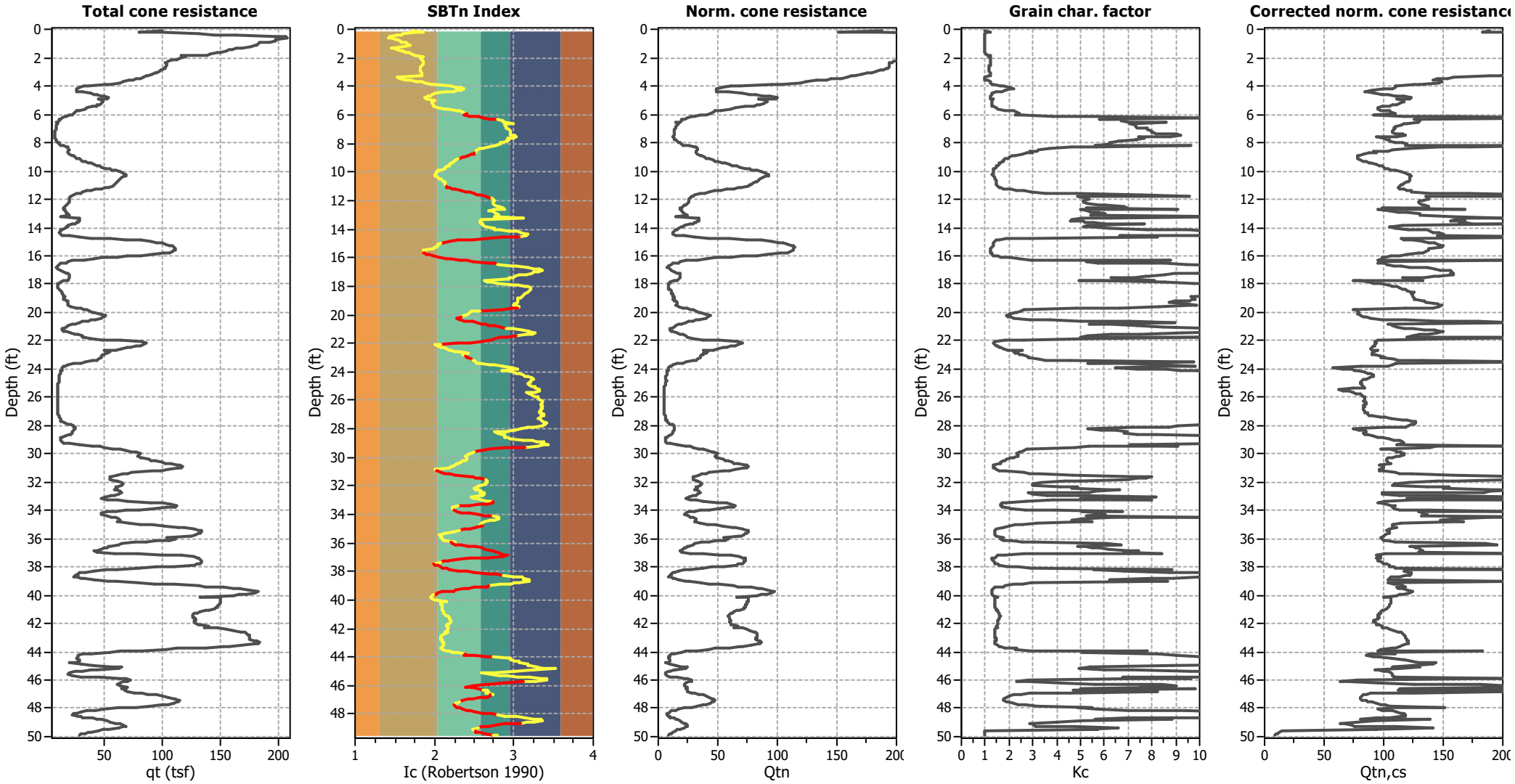
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.78	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

SBTn legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

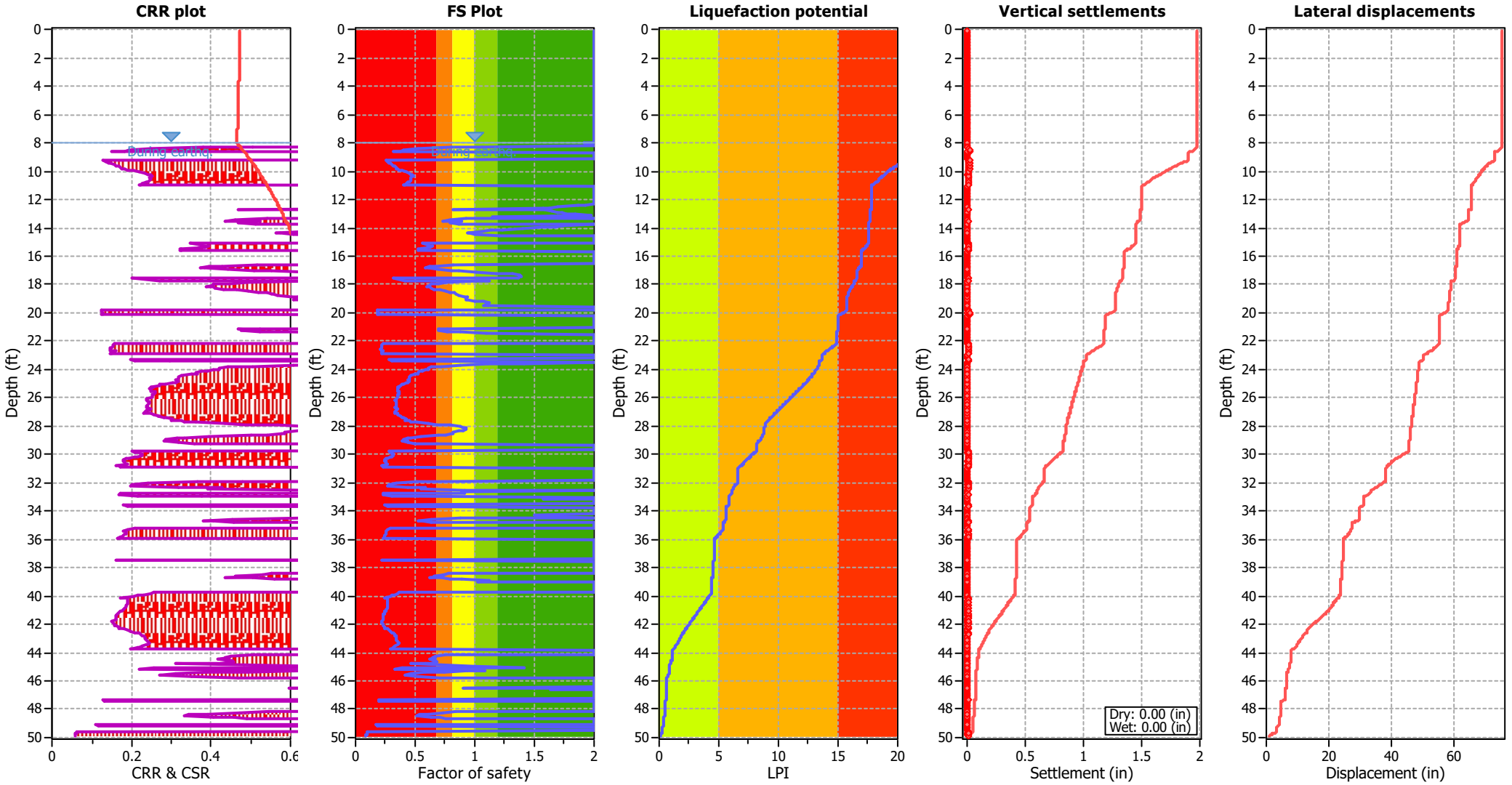
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_o applied:	No
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.78	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.78	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

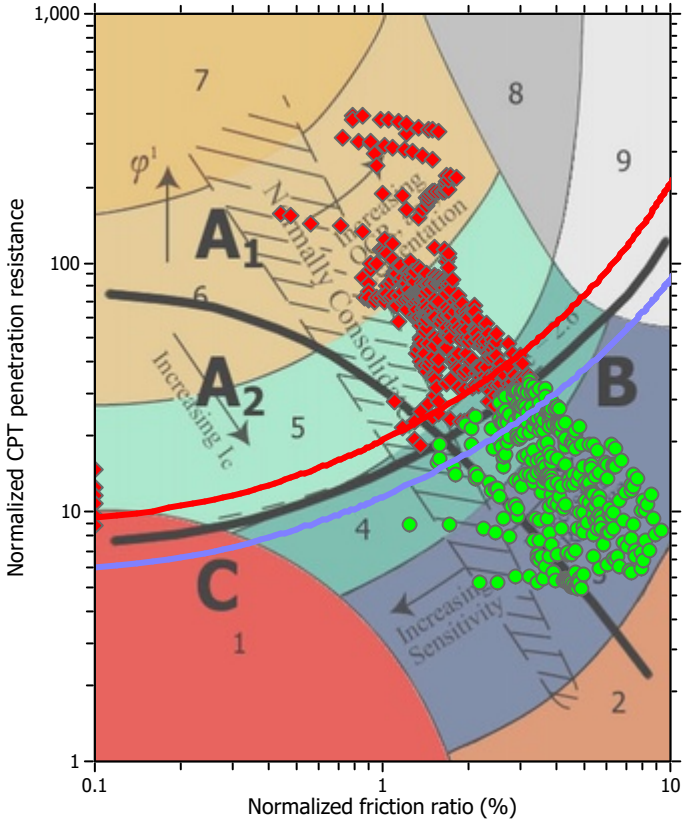
F.S. color scheme

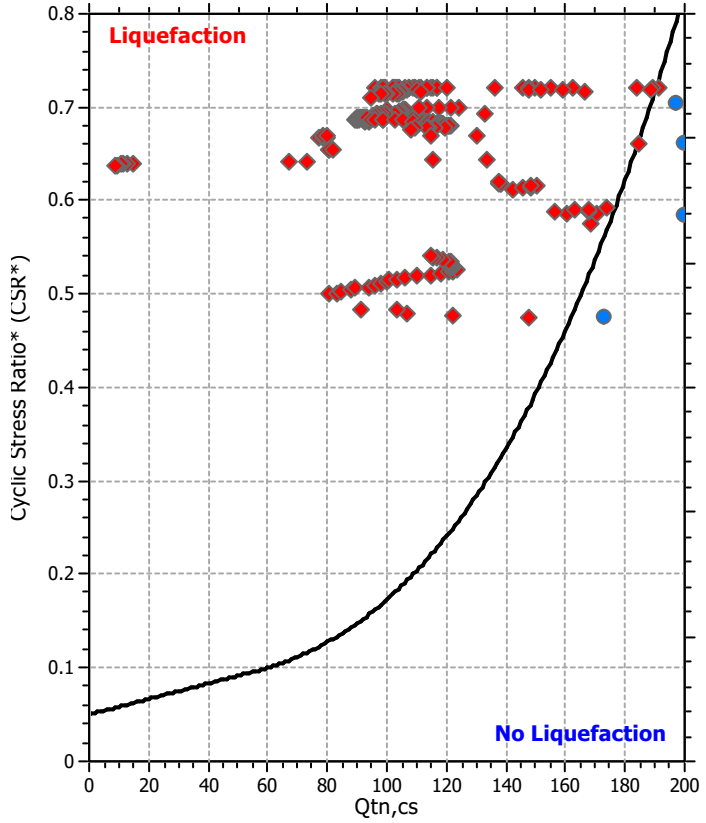
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

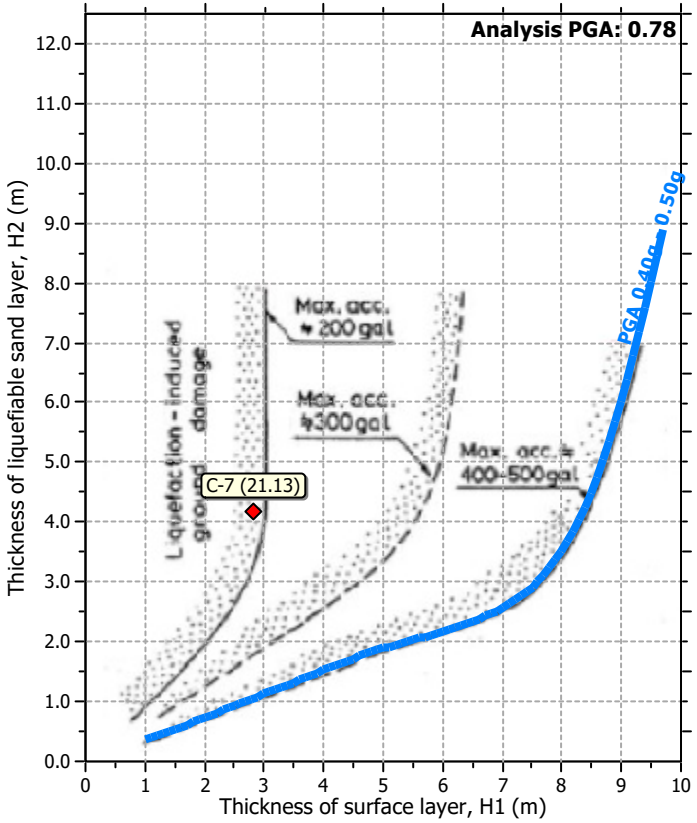
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots



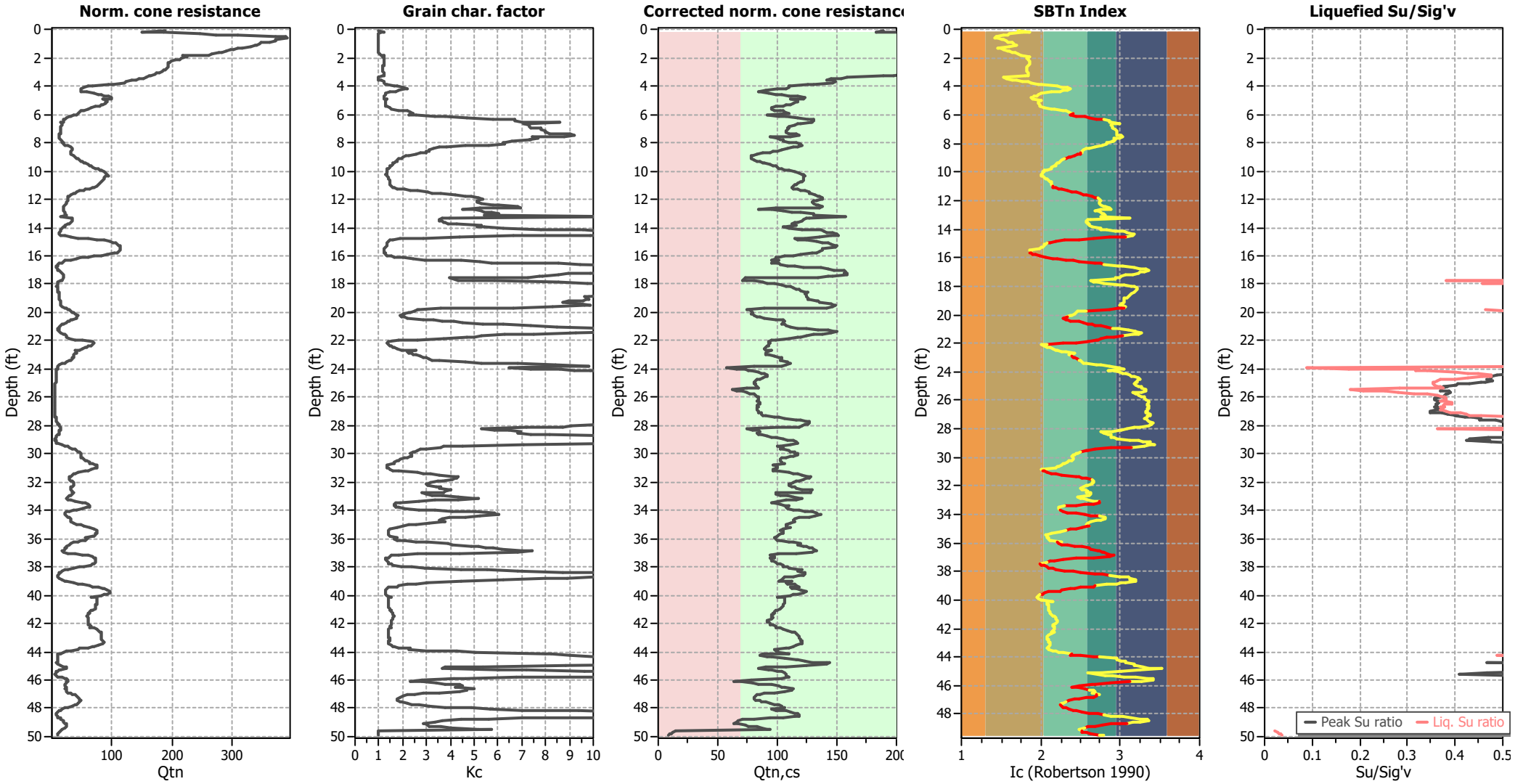




Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.78	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

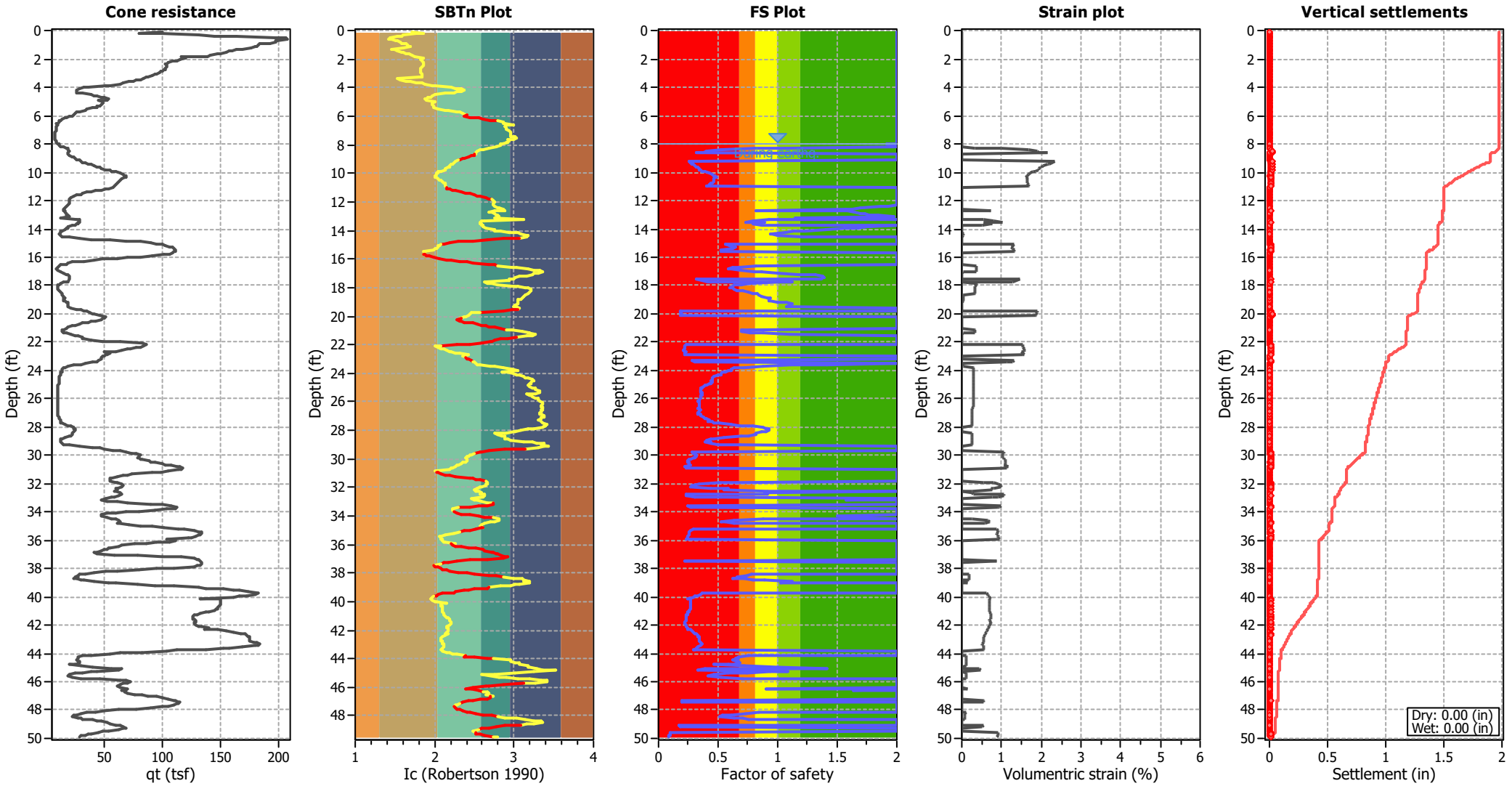
Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_o applied:	No
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.78	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

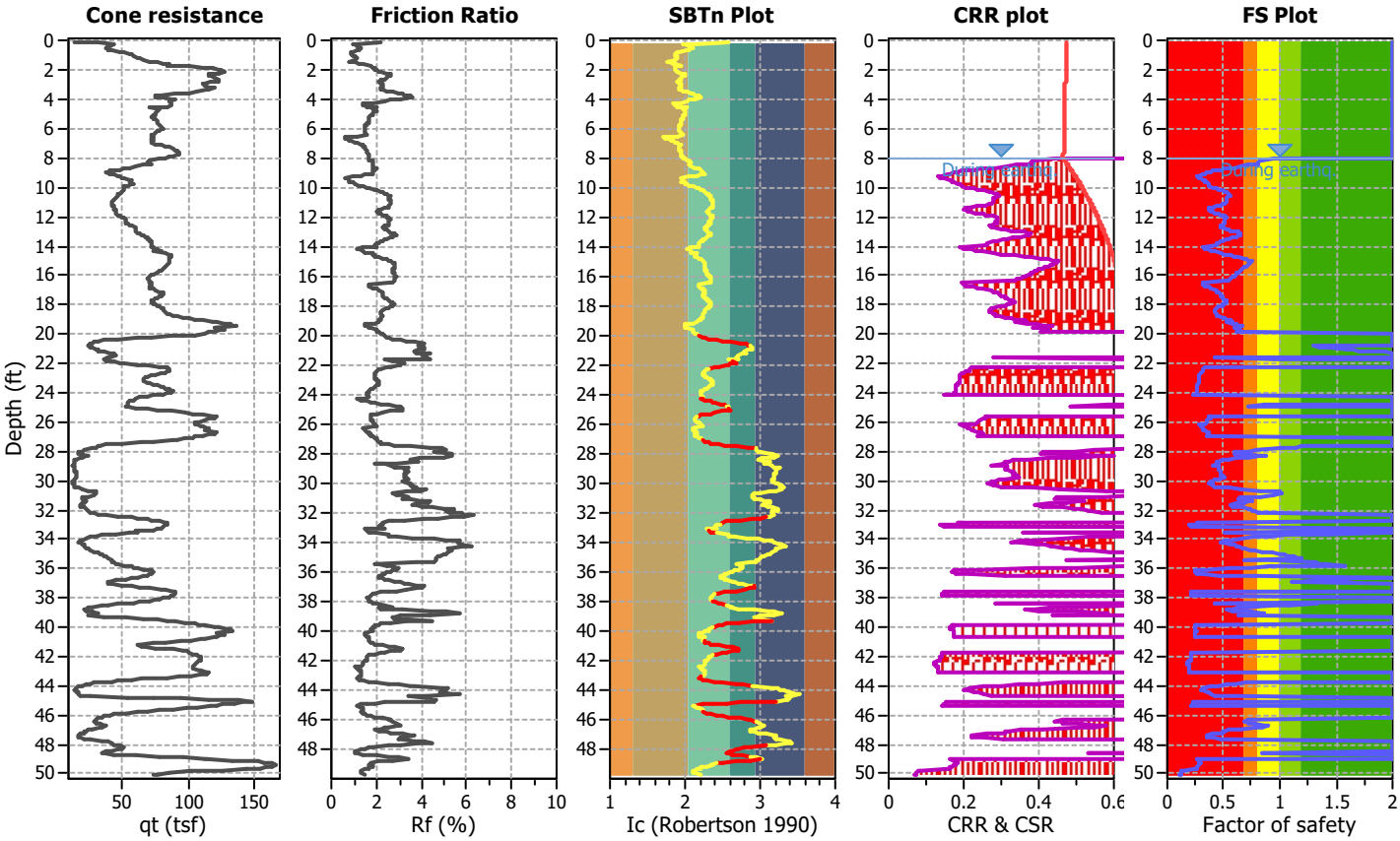
LIQUEFACTION ANALYSIS REPORT

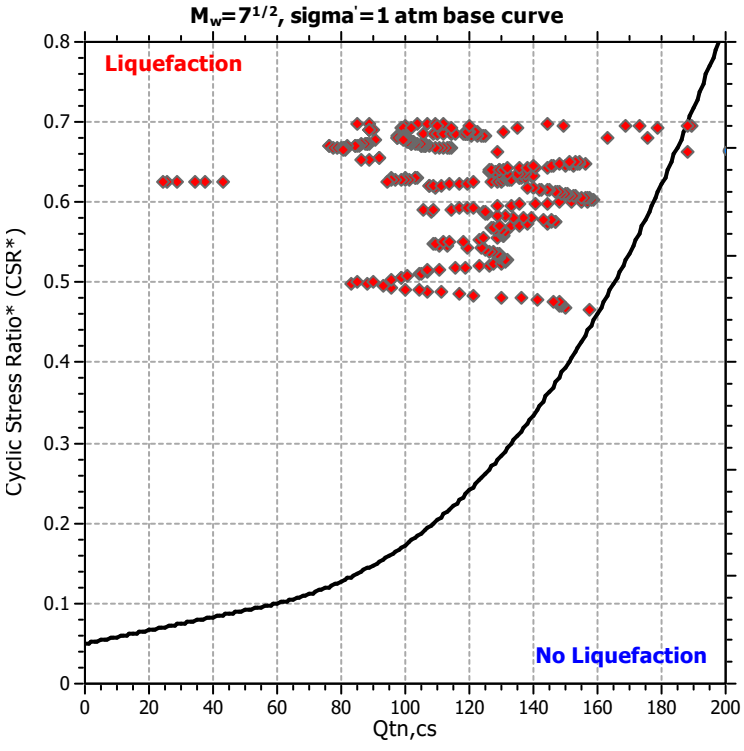
Project title : GPI / Lynwood High School **Location : 4050 E. Imperial Highway, Lynwood, CA**

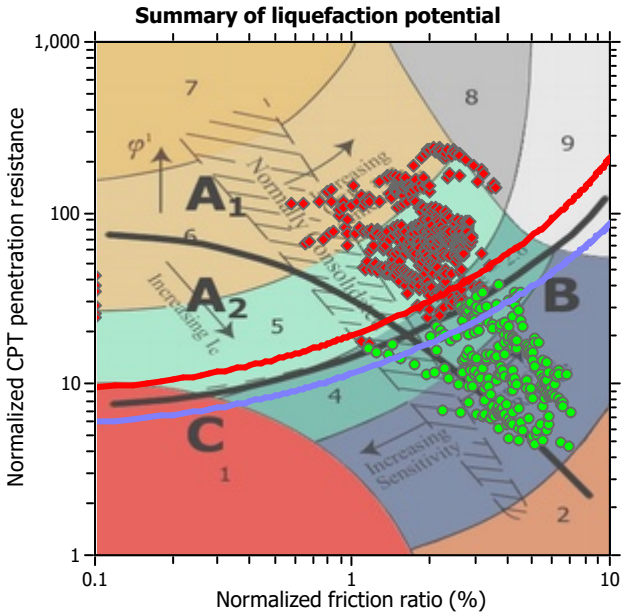
CPT file : C-8

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	45.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	8.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth:	60.00 ft
Earthquake magnitude M_w:	7.30	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.78	Unit weight calculation:	Based on SBT	K_σ applied:	No		

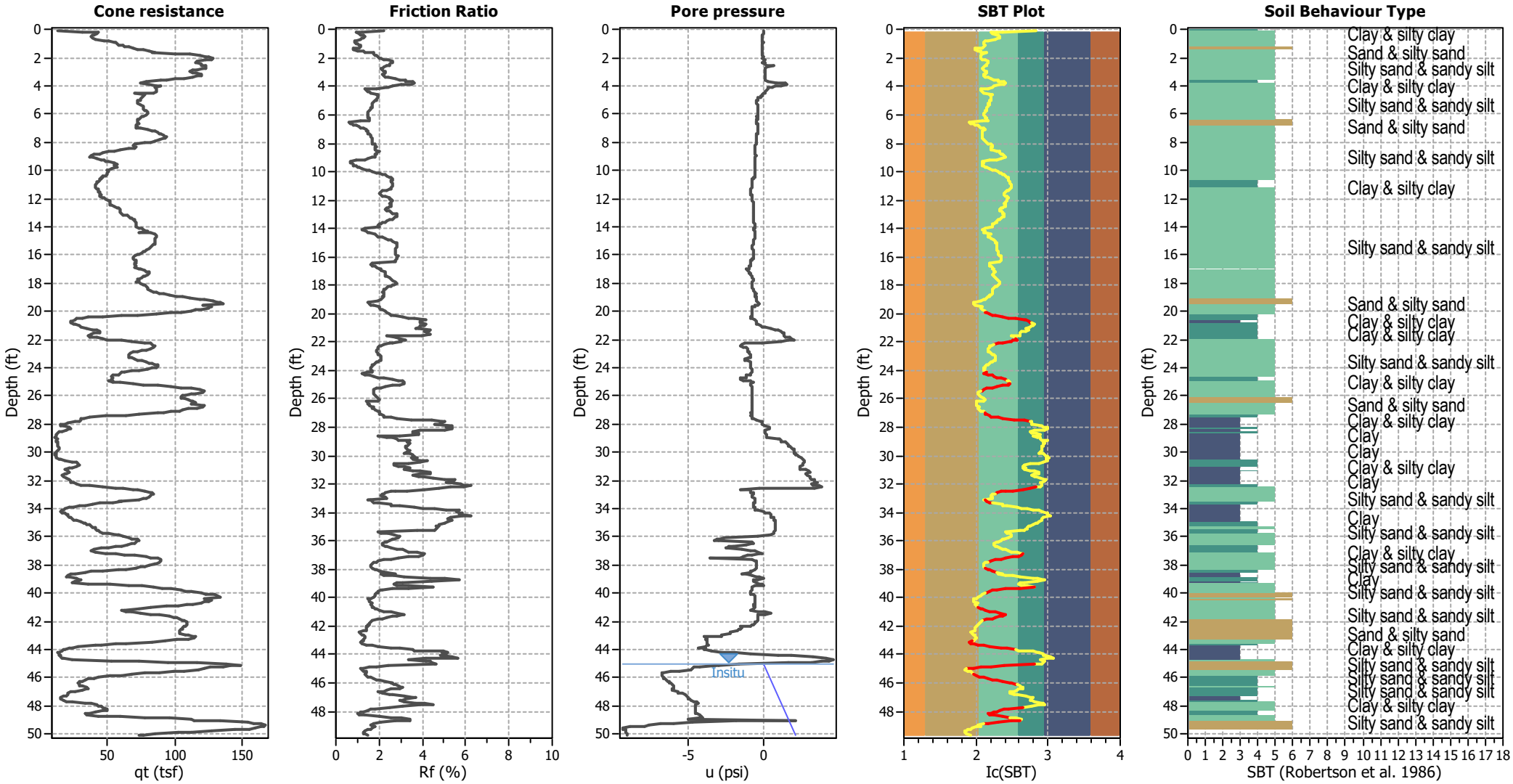






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots



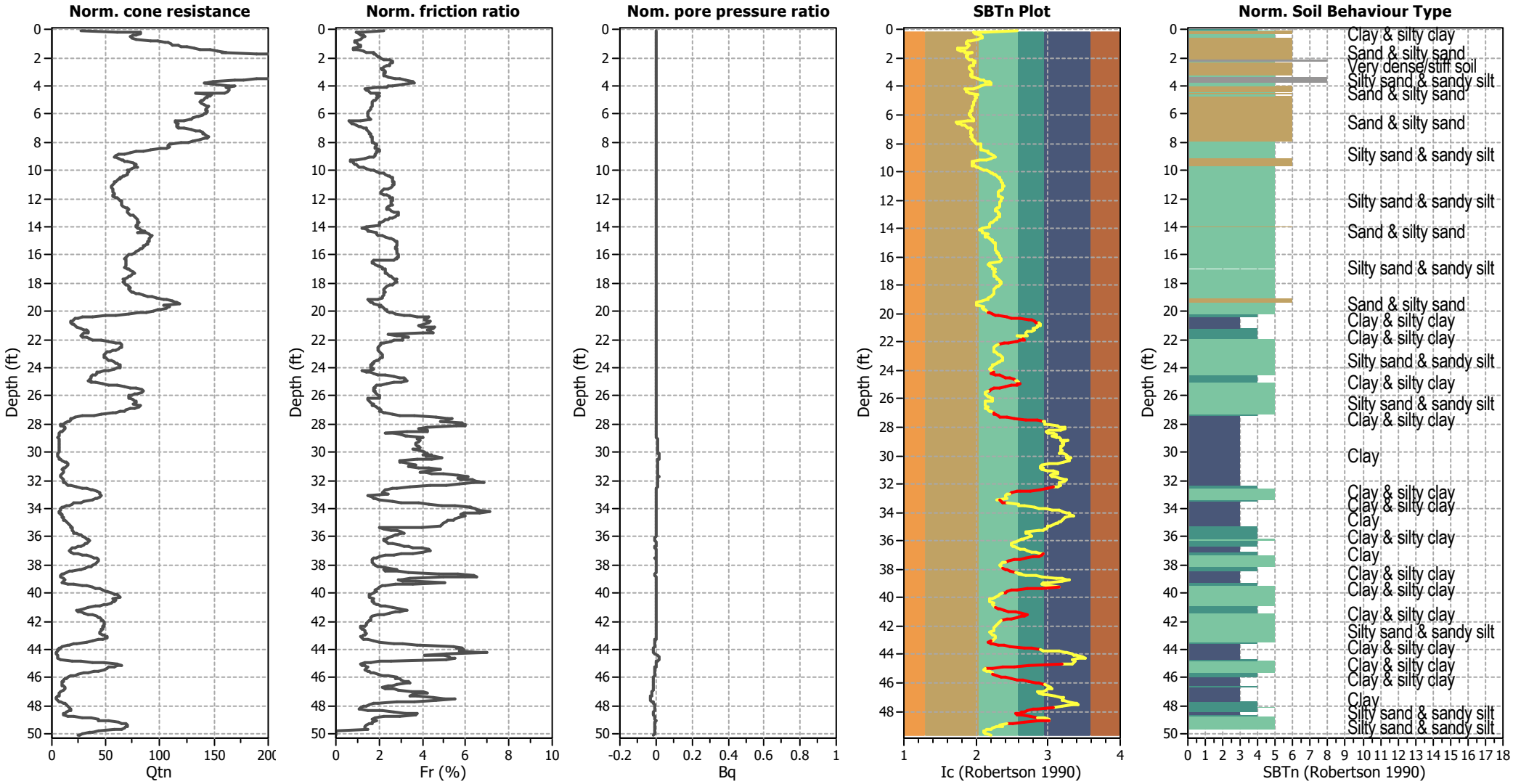
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.78	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

SBT legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

CPT basic interpretation plots (normalized)



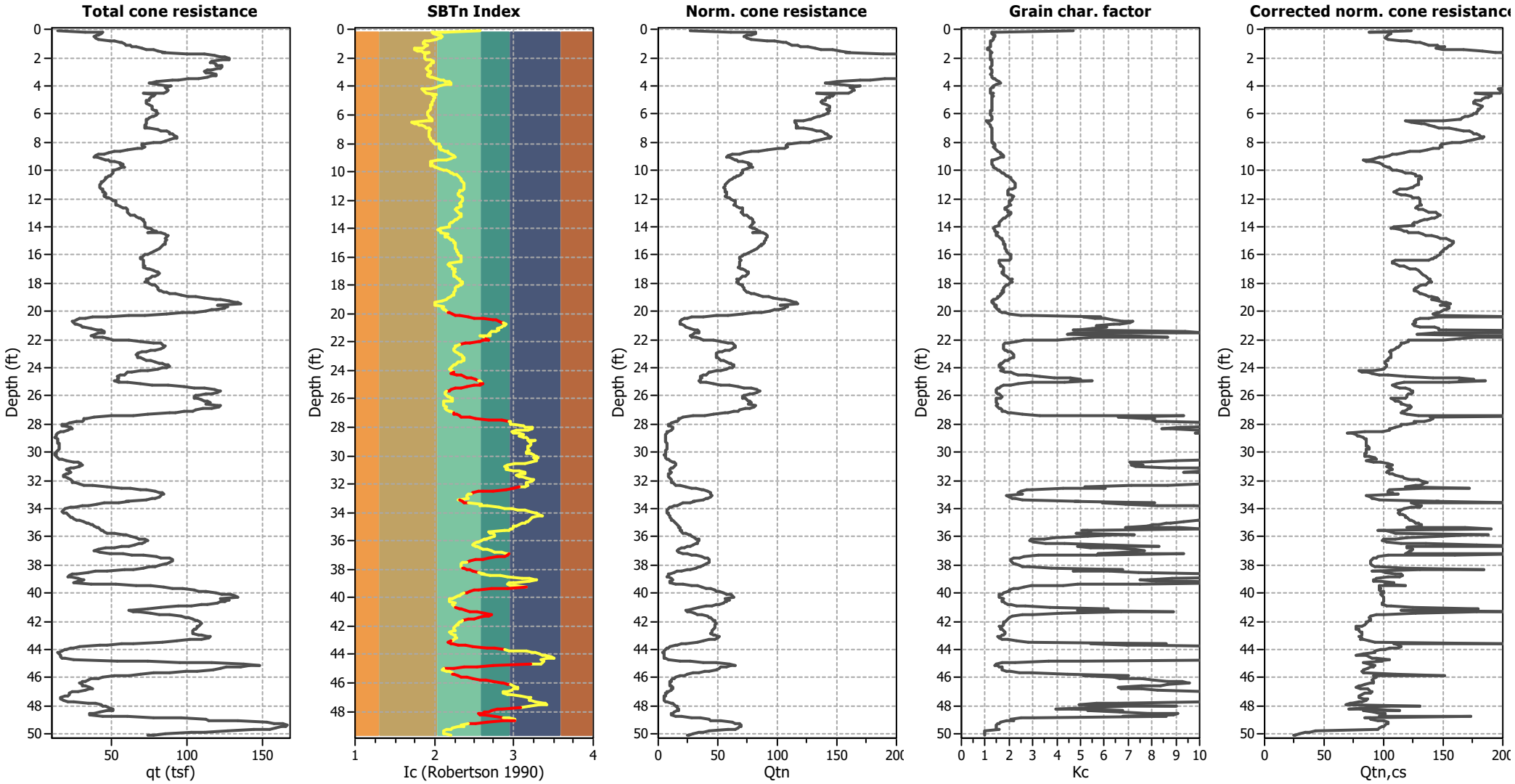
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.78	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

SBTn legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

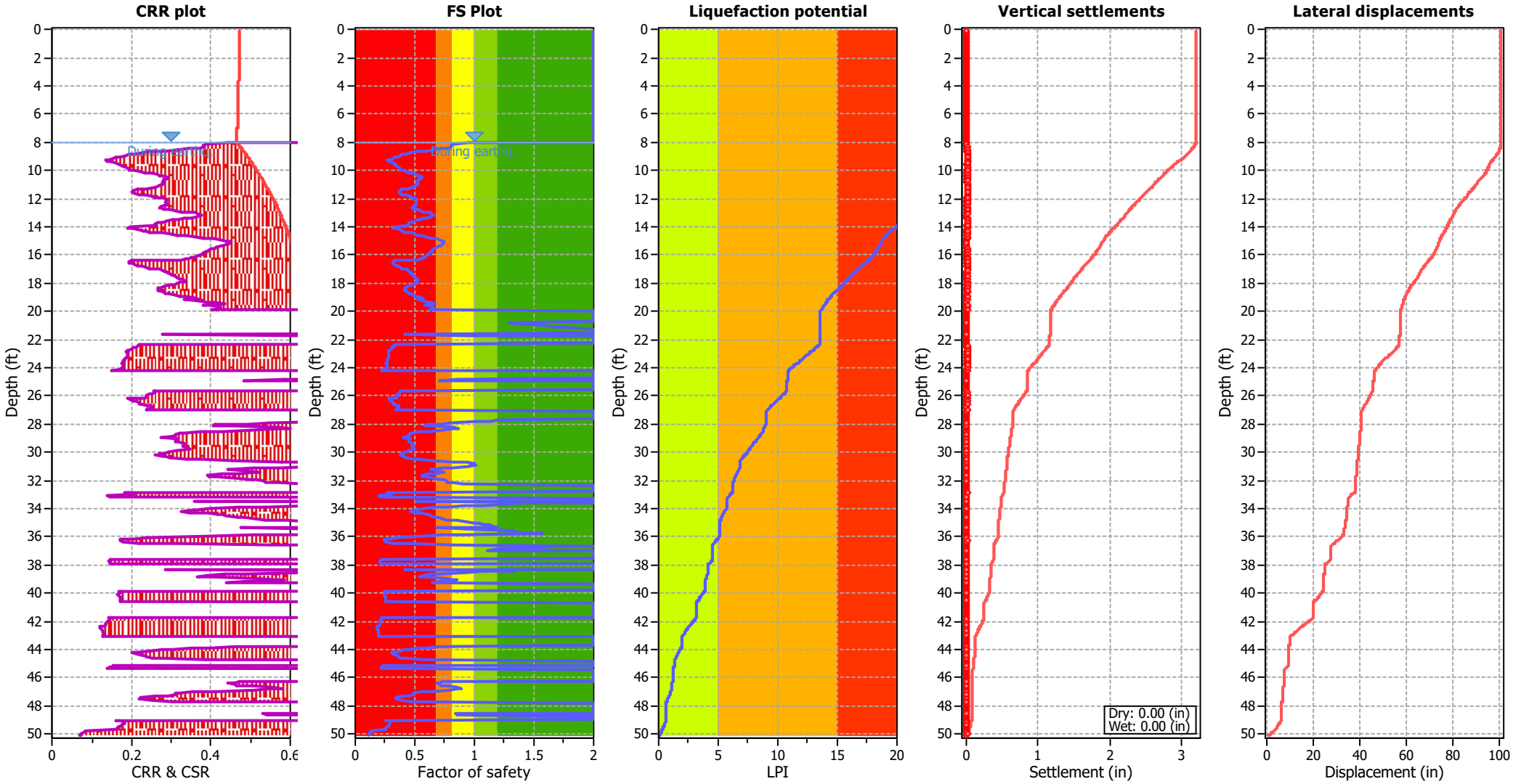
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.78	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
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Peak ground acceleration:	0.78	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

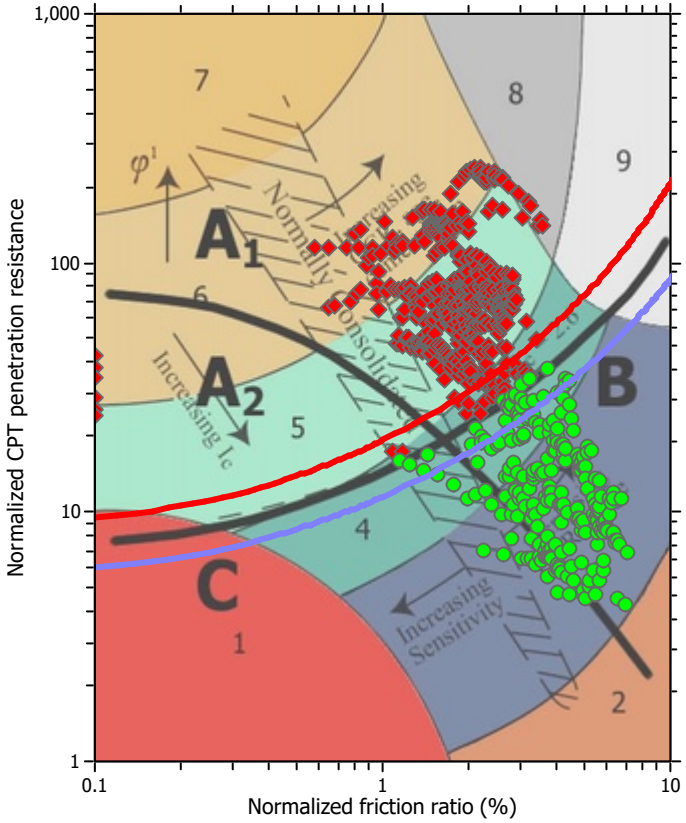
F.S. color scheme

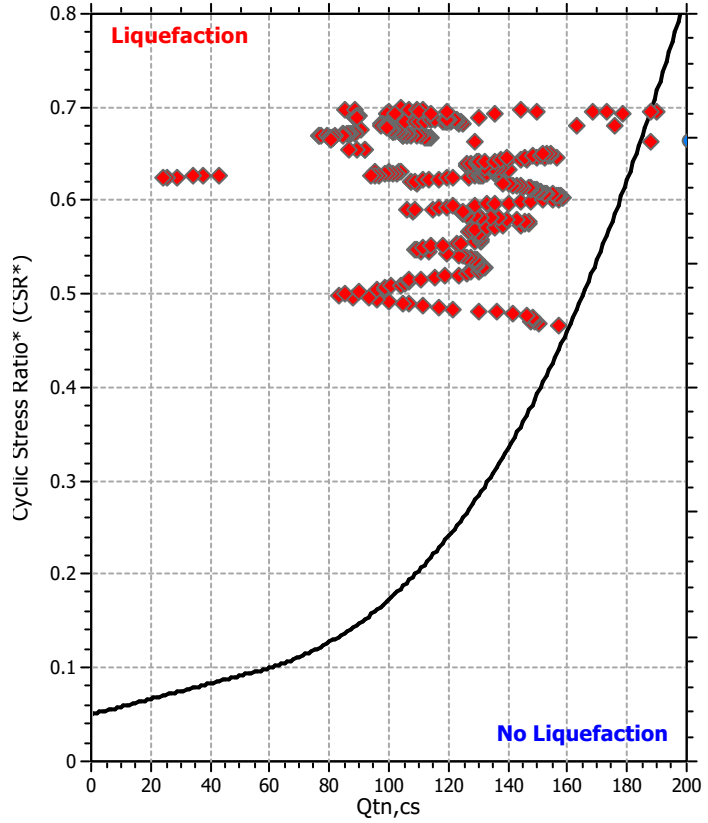
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

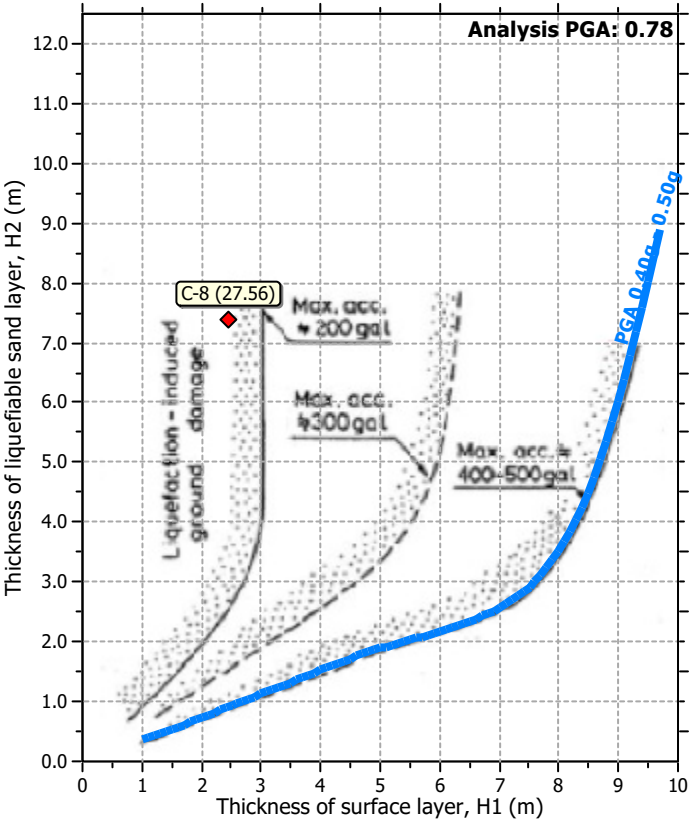
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots



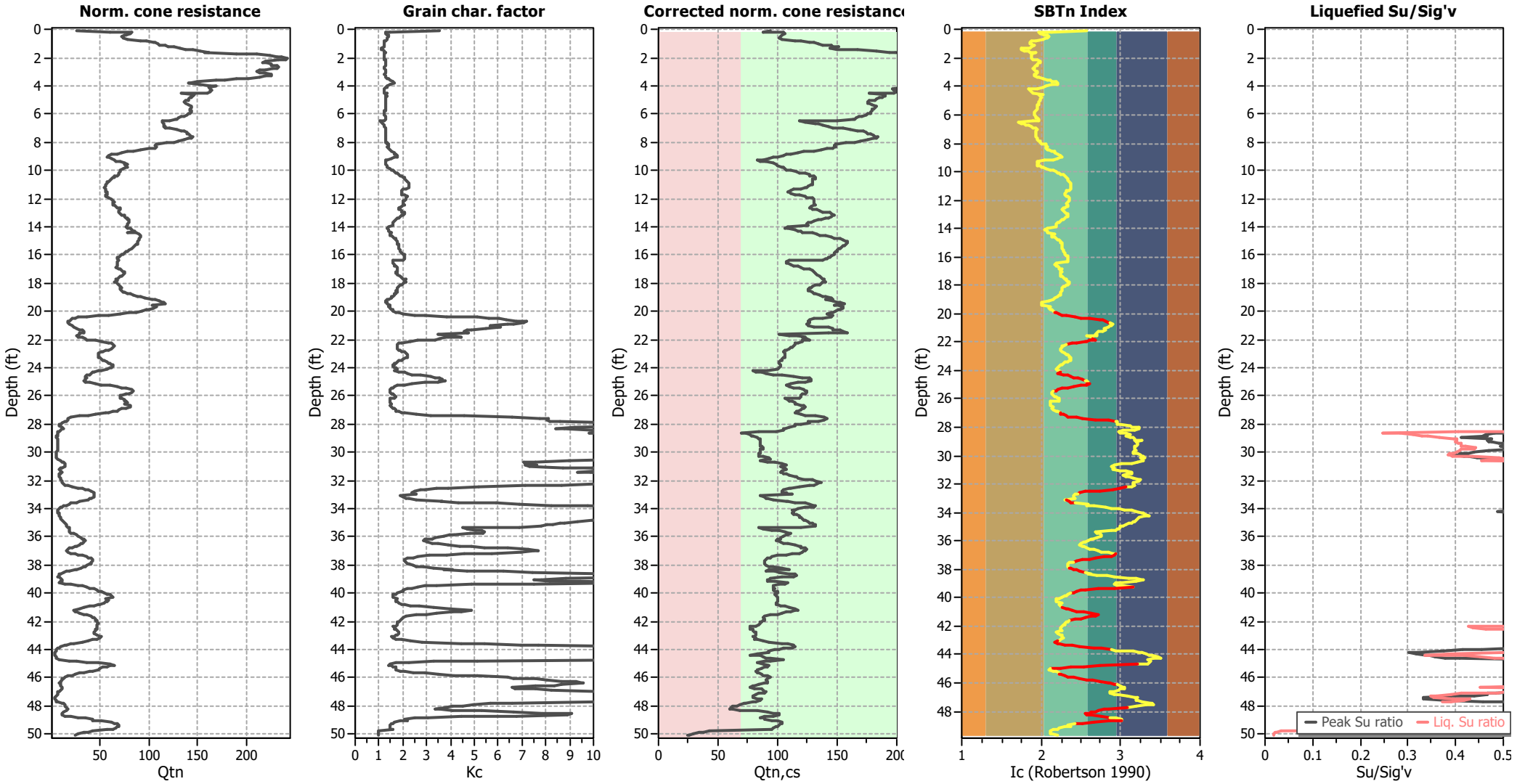




Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_o applied:	No
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.78	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

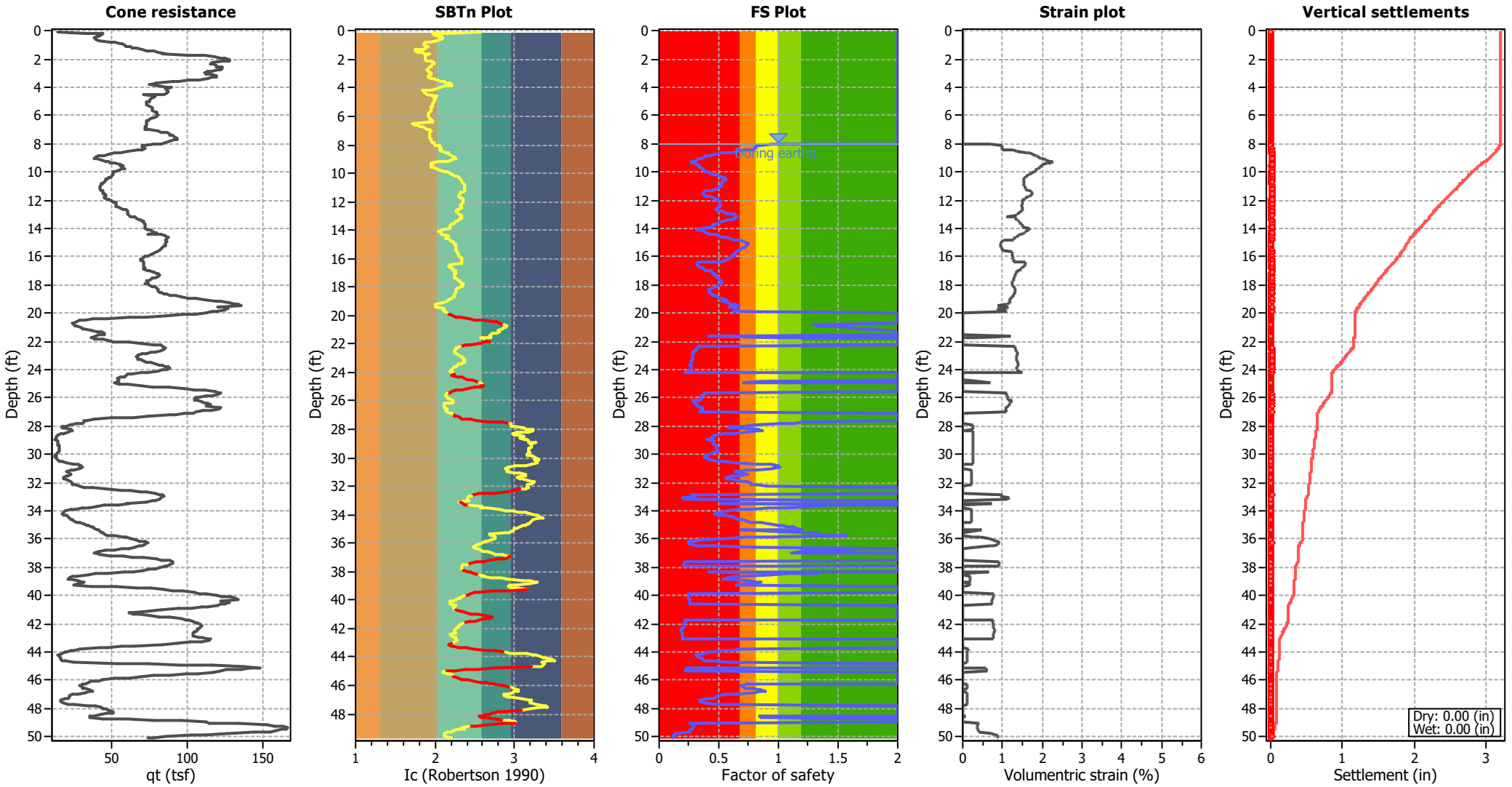
Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_o applied:	No
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.78	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	45.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain