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November 27, 2019
File Number 21855

Century Housing
1000 Corporate Pointe
Culver City, California 90230

Attention: Christopher Larkin

Subject: Geotechnical Engineering Investigation
Proposed Residential Complex
Century Village at Cabrillo (CVC) Phase VI
2221 W. Williams Street, Long Beach, California

Dear Mr. Larkin:

This letter transmits the Geotechnical Engineering Investigation for the subject site prepared by Geotechnologies, Inc. This report provides geotechnical recommendations for the proposed development, including earthwork, seismic design, excavations, retaining walls and foundation design. Engineering for the proposed project should not begin until approval of the geotechnical investigation is granted by the local building official. Significant changes in the geotechnical recommendations may result due to the building department review process.

The validity of the recommendations presented herein is dependent upon review of the geotechnical aspects of the project during construction by this firm. The subsurface conditions described herein have been projected from limited subsurface exploration and laboratory testing. The exploration and testing presented in this report should in no way be construed to reflect any variations which may occur between the exploration locations or which may result from changes in subsurface conditions.

Should you have any questions please contact this office.

Respectfully submitted,
GEOTECHNOLOGIES, INC.

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GEOTECHNICAL ENGINEERING INVESTIGATION
PROPOSED RESIDENTIAL COMPLEX
CENTURY VILLAGE AT CABRILLO (CVC) PHASE VI
2221 WEST WILLIAMS STREET
LONG BEACH, CALIFORNIA

INTRODUCTION

This report presents the results of the geotechnical engineering investigation performed on the subject site. The purpose of this investigation was to identify the distribution and engineering properties of the geologic materials underlying the site, and to provide geotechnical recommendations for the design of the proposed development.

This investigation included three exploratory borings, four cone penetration test soundings, collection of representative samples, laboratory testing, engineering analysis, review of published geologic data, review of available geotechnical engineering information, and the preparation of this report. The exploration locations are shown on the enclosed Plot Plan. The results of the exploration and the laboratory testing are provided in the Appendix of this report.

PROPOSED DEVELOPMENT

Information concerning the proposed development was furnished by the client with supplemental details provided by The Architects Collective. The proposed project consists of the construction a 4-story residential complex built near the existing site grade. The structure will provide an at-grade parking garage, offices, and community room on the ground floor level, with residential units on the upper three floors. Other site improvements including outdoor walkways, driveways, and landscaping are anticipated. As part of the proposed development, two adjacent residential buildings located northwest of the site will be demolished to provide space for the new residential complex.

Column loads are estimated to be between 300 and 600 kips. Wall loads are estimated to be between 3 and 5 kips per lineal foot. Grading is expected to consist of ground improvement preparations as well as removal and recompaction of existing unsuitable soils. The location of the proposed structure relative to surrounding developments is shown on the enclosed Plot Plan.

Any changes in the design of the project or location of any structure, as outlined in this report, should be reviewed by this office. The recommendations contained in this report should not be considered valid until reviewed and modified or reaffirmed, in writing, subsequent to such review.

SITE CONDITIONS

The subject site is located at 2221 West Williams Street, in the City of Long Beach, California. The site is bounded by a 4-story residential structure to the north, West Willard Street to the east, West Williams Street to the south, and residential structures along with a recreational area to the west. The site is shown relative to nearby topographic features on the enclosed Vicinity Map.

At the time of exploration, the site was occupied by a residential development consisting of a two-story apartment building with an associated parking lot. The overall topography of the site is relatively level with minor elevation changes as it gently descends towards the southwest. Drainage occurs by sheetflow along existing topographic contours towards the adjacent city streets. Vegetation on the site is contained within landscaped areas consisting of grasses, bushes, and trees. The surrounding developments predominantly consist of residential and commercial structures.

GEOTECHNICAL EXPLORATION

FIELD EXPLORATION

The site was explored between August 21, 2019, and August 29, 2019 by excavating three exploratory borings and performing four Cone Penetration Test soundings (CPT). The borings were excavated with the aid of a 5-inch diameter mud rotary drilling machine and were advanced to a depth of 70 feet below the existing grade. Soil samples were collected from the borings at regular intervals for laboratory testing. The CPT's were each conducted to a depth of 80 feet below the existing grade. The boring and CPT locations are shown on the enclosed Plot Plan. The geologic materials encountered within the borings are logged on Plates A-1 through A-3 enclosed in the Appendix of this report. CPT data logs and analyses have also been provided in the Appendix of this report.

Geologic Materials

The geologic materials underlying the site consist of existing fill and natural alluvium. Fill was encountered in the exploratory borings to depths of 2.5 to 3 feet below the existing grade consisting of sandy silt, silty sand and sand. The fill soils are yellowish to dark brown, moist, firm, medium dense, and fine grained.

The fill is underlain by native alluvial soils consisting sands, silty sands, sandy silts, and silty clays. The alluvium ranged from yellowish to dark brown or gray in color, moist to wet, loose to very dense, and primarily fine grained. At depths of 50 feet and below, the alluvium exhibited greater density indicated by the increased penetration resistance. More detailed descriptions of the geologic materials encountered may be obtained from the individual boring logs provided in the Appendix of this report.

Groundwater

Groundwater was encountered during exploration at depths of 4.5 to 8 feet below the existing grade. Review of the Seismic Hazard Zone Report of the Long Beach 7.5 Minute Quadrangle (CDMG, 1998, Revised 2006) indicates that the historically highest groundwater level within the vicinity of the site is approximately 15 feet below the ground surface.

Fluctuations in the level of groundwater may occur due to variations in rainfall, temperature, and other factors not evident at the time of the measurements reported herein. Fluctuations may also occur across the site. Higher groundwater levels can result in changed conditions.

Caving

Caving did not occur within the mud rotary borings. However, it should be noted that the boreholes were filled with drilling fluid during excavation in order to prevent caving. During the Cone Penetration Tests (CPT), as the cone penetrometer was withdrawn from the CPT soundings, caving occurred at depths below the groundwater level. Based on the experience of this firm, large diameter excavations, excavations that encounter granular, cohesionless soils and excavations below the groundwater table will most likely experience caving.

SEISMIC EVALUATION

REGIONAL GEOLOGIC SETTING

The subject site is located within the Los Angeles Basin and Peninsular Ranges Geomorphic Province. The Peninsular Ranges are characterized by northwest-trending blocks of mountain ridges and sediment-floored valleys. The dominant geologic structural features are northwest trending fault zones that either die out to the northwest or terminate at east-west trending reverse faults that form the southern margin of the Transverse Ranges (Yerkes, 1965).

The Los Angeles Basin is located at the northern end of the Peninsular Ranges Geomorphic Province. The basin is bounded by the east and southeast by the Santa Ana Mountains and San Joaquin Hills, to the northwest by the Santa Monica Mountains. Over 22 million years ago the Los Angeles basin was a deep marine basin formed by tectonic forces between the North American and Pacific plates. Since that time, over 5 miles of marine and non-marine sedimentary rock as well as intrusive and extrusive igneous rocks have filled the basin. During the last 2 million years, defined by the Pleistocene and Holocene epochs, the Los Angeles basin and surrounding mountain ranges have been uplifted to form the present day landscape. Erosion of the surrounding mountains, has resulted in deposition of unconsolidated sediments in low-lying areas by rivers such as the Los Angeles River. Areas that have experienced subtle uplift have been eroded with gullies.

REGIONAL FAULTING

Based on criteria established by the California Division of Mines and Geology (CDMG) now called California Geologic Survey (CGS), faults may be categorized as active, potentially active, or inactive. Active faults are those which show evidence of surface displacement within the last 11,000 years (Holocene-age). Potentially-active faults are those that show evidence of most recent surface displacement within the last 1.6 million years (Quaternary-age). Faults showing no evidence of surface displacement within the last 1.6 million years are considered inactive for most purposes, with the exception of design of some critical structures.

Buried thrust faults are faults without a surface expression but are a significant source of seismic activity. They are typically broadly defined based on the analysis of seismic wave recordings of hundreds of small and large earthquakes in the southern California area. Due to the buried nature of these thrust faults, their existence is usually not known until they produce an earthquake. The risk for surface rupture potential of these buried thrust faults is inferred to be low (Leighton, 1990). However, the seismic risk of these buried structures in terms of recurrence and maximum potential

magnitude, is not well established. Therefore, the potential for surface rupture on these surface-verging splays at magnitudes higher than 6.0 cannot be precluded.

SEISMIC DESIGN CONSIDERATIONS

The primary geologic hazard at the site is moderate to strong ground motion (acceleration) caused by an earthquake on any of the local or regional faults. Design of the proposed development in accordance with the provisions of the most current California Building Code (CBC) is intended to minimize the potential effects of ground shaking. The potential for other earthquake-induced hazards was also evaluated including surface rupture, liquefaction, dynamic settlement, inundation and landsliding.

Surface Rupture

In 1972, the Alquist-Priolo Special Studies Zones Act (now known as the Alquist-Priolo Earthquake Fault Zoning Act) was passed into law. The Act defines "active" and "potentially active" faults utilizing the same aging criteria as that used by California Geological Survey (CGS). However, established state policy has been to zone only those faults which have direct evidence of movement within the last 11,000 years. It is this recency of fault movement that the CGS considers as a characteristic for faults that have a relatively high potential for ground rupture in the future.

CGS policy is to delineate a boundary from 200 to 500 feet wide on each side of the known fault trace based on the location precision, the complexity, or the regional significance of the fault. If a site lies within an Earthquake Fault Zone, a geologic fault rupture investigation must be performed that demonstrates that the proposed building site is not threatened by surface displacement from the fault before development permits may be issued.

Based on research of available literature and results of site reconnaissance, no known active faults or potentially active faults underlie the subject site. Review of the Earthquake Zones of Required Investigation for the Long Beach Quadrangle indicates that the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. Based on these conditions, the potential for surface rupture at the subject site is considered low.

Liquefaction

Liquefaction is a phenomenon in which saturated silty to cohesionless soils below the groundwater table are subject to a temporary loss of strength due to the buildup of excess pore pressure during cyclic loading conditions such as those induced by an earthquake. Liquefaction-related effects include loss of bearing strength, settlement, and lateral spreading.

Liquefaction typically occurs in areas where groundwater is less than 50 feet from the surface, and where the soils are composed of poorly consolidated, fine to medium-grained sand. In addition to the necessary soil conditions, the ground acceleration and duration of the earthquake must also be of a sufficient level to initiate liquefaction.

The Seismic Hazards Zone Map of the Long Beach Quadrangle (CDMG, 1999), indicates that the subject site is located within an area designated as "Liquefiable." This determination is based on groundwater depth records, soil type and distance to a fault capable of producing a substantial earthquake. In addition, the Seismic Safety Element of the City of Long Beach General Plan (1988) indicates the site is located in an area where liquefaction potential is significant.

Site-specific liquefaction analyses were performed utilizing data obtained from the Standard Penetration Test (SPT) in Borings B1, B2, and B3, as well as data obtained from the Cone Penetration Test (CPT) conducted in CPT 1 through 4. CPT 1, 3, and 4 were performed adjacent to Boings B1, B2, and B3 (respectively), for the purpose of comparison and correlation of soil data.

Groundwater was encountered in the borings at depths of 4.5 to 8 feet below the ground surface. According to the Seismic Hazard Zone Report of the Long Beach 7.5-Minute Quadrangle (CDMG, 1998, Revised 2006), the historic high groundwater level for the subject site is approximately 15 feet below the ground surface. A groundwater level of 4.5 feet below the ground surface was utilized for the enclosed liquefaction analyses.

Section 11.8.3 of ASCE 7-16 indicates that the potential for liquefaction shall be evaluated utilizing a site-modified peak ground acceleration (PGA_M) corresponding to the Maximum Considered Earthquake (MCE_G). The OSHPD Seismic Design Maps Tool yielded a site class modified peak ground acceleration (PGA_M) of 0.893g. The USGS Probabilistic Seismic Hazard Analysis Deaggregation program (USGS, 2014) was utilized to determine the magnitude of the Maximum Considered Earthquake (MCE_G). The deaggregation program yielded a modal magnitude (M_w) of 7.3 for the site. Therefore, the liquefaction potential evaluations were performed by using a magnitude 7.3 earthquake and a peak ground acceleration of 0.893g.

Standard Penetration Test (SPT) – Liquefaction Analyses

Site-specific liquefaction analyses were performed following the Recommended Procedures for Implementation of CDMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California (Martin and Lew, 1999). Recommendations provided in CGS Special Publication 117A (CGS, 2008) and in EERI Monograph (MNO-12, 2008 by Idriss and Boulanger) were also incorporated into the analyses.

The enclosed "Liquefaction Potential" analyses are based on Borings B1, B2, and B3. Standard Penetration Test (SPT) data were collected at 5-foot intervals. Samples of the collected materials were conveyed to the laboratory for testing and analysis. The percent passing a Number 200 sieve, Atterberg Limits, and the plasticity index (PI) of representative samples of the soils encountered in the exploratory borings are presented on the enclosed E-Plates and F-Plates.

Based on criteria set forth in CGS Special Publication SP117A, a factor of safety against the occurrence of liquefaction greater than about 1.3 can be considered an acceptable level of risk where high-quality, site-specific penetration resistance and geotechnical laboratory data is collected. The SPT liquefaction analyses indicate soil layers susceptible to liquefaction underlying the site at various depths below the ground surface. The SPT liquefaction calculations have been enclosed in the Appendix of this report.

Cone Penetration Test (CPT) – Liquefaction Analyses

Liquefaction analyses were also performed using data from four CPT soundings. One of the advantages associated with the Cone Penetration Test is its ability to provide relatively continuous profiling of the underlying soils, making it very useful for highly stratified subsurface conditions.

CPT data were analyzed utilizing the liquefaction assessment software CLiq v.3.0.2.4 (Geologismiki, 2006). The analyses are based on published articles by (Robertson and Wride, 1998) and (Youd et al. 2001). The program estimates the soil characteristics directly from the CPT data and incorporates the interpreted results into an evaluation of their resistance to cyclic loading. It should be noted that the evaluations for liquefaction potential were limited to a depth of 60 feet below the ground surface.

The CPT liquefaction analyses indicate that soil layers at various depths below the ground surface would be susceptible to liquefaction. These potentially liquefiable layers occur throughout the subsurface profile with thicknesses ranging from approximately 6 inches to 11 feet.

In nearly all of the CPT analyses, potentially liquefiable layers began to be identified right at the depth of the existing groundwater levels. Therefore, the surficial soils which currently reside above the existing groundwater levels may also be susceptible to liquefaction if submerged.

Dynamic Settlement

Seismically-induced settlements have been calculated utilizing the results of the liquefaction analyses based on SPT blow count data and the CPT soundings. The results of the liquefaction analyses are provided in the following table.

SPT & CPT LIQUEFACTION ANALYSES		
Boring / CPT Sounding	**Potentially Liquefiable Layers (Depth Below Ground Surface, feet)**	**Total Liquefaction Settlement (inches)**
B1	10-14, 15-22, 30-34	5.58
B2	8-19, 25-34, 38-49	6.14
B3	5-12, 20-22, 28-37, 43-47	7.34
CPT-1	9 – 18.5 (stratified) 35 - 60 (stratified)	5.47
CPT-2	7.5 – 18.5 (stratified) 35-37 43 – 60 (stratified)	5.95
CPT-3	7.5 – 24 (stratified) 35 – 60 (stratified)	6.16
CPT-4	4.5 – 19 (stratified) 26 – 60 (stratified)	7.18

Based on the enclosed SPT liquefaction analyses, the total seismically-induced settlement due to liquefaction could range from 5.58 to 7.34 inches. The CPT analyses indicated potential liquefaction settlements ranging from 5.47 to 7.18 inches.

According to (Martin and Lew, 1999), the differential settlement used in foundation design should be up to two-thirds of the maximum total settlement, or 4.92 inches. However, multiple site-specific liquefaction analyses have been performed on the site. Therefore, it is the opinion of this

firm that the seismically-induced differential settlement may be estimated as one-half of the maximum total settlement. This would correspond to a differential settlement of approximately 3.67 inches.

Surface Manifestation

It has been shown in studies by O'Rourke and Pease (1997) and Youd and Garris (1995), building upon work by Ishihara (1985), that the visible effects of liquefaction on the ground surface are only manifested if the relative and absolute thicknesses of liquefiable soils to overlying non-liquefiable surface material fall within a certain range. Surface manifestations of liquefaction include phenomena such as sand boils.

The liquefaction analyses indicate relative thicknesses of liquefiable to non-liquefiable soils that are within the bounds where surface manifestations have been observed during past earthquakes. According to (Boulanger and Idriss, 2008), "damage from liquefaction is seldom, however, due to sand boils themselves, but rather due to the loss of strength and stiffness in the soils that have liquefied and the associated ground deformations that ensue."

Liquefaction of the upper site soils could result in a temporary loss of strength and reduction of bearing capacity. Recommendations have been provided herein for implementation during the design and construction of the proposed residential complex to mitigate the potential for surface manifestations of liquefaction and a loss in bearing strength.

Lateral Spreading

Lateral spreading is the most pervasive type of liquefaction-induced ground failure. During lateral spread, blocks of mostly intact surficial soil displace down gradient or towards a free face along a shear zone that has formed within the liquefied sediment. According to the procedure provided by Bartlett, Hansen, and Youd, "Revised Multilinear Regression Equations for Prediction of Lateral

Spread Displacement", ASCE, Journal of Geotechnical Engineering, Vol. 128, No. 12, December 2002, significant displacement is not likely for M < 8 earthquakes, when saturated cohesionless sediments have corrected (N1)60 values greater than 15.

The surficial sediments encountered within borings B1, B2, and B3 have corrected (N1)60 values of less than 15 within various layers to depths of approximately 15 to 35 feet. Based on the liquefaction evaluations, some of these soil layers would be susceptible to liquefaction. All corrected (N1)60 values below 35 feet were greater than 15. Based on these results, the upper site soils could be considered susceptible to lateral spreading during a liquefaction event. However, due to the relatively level nature of the subject site and surrounding areas, and the lack of significant nearby free faces, the potential for substantial lateral spreading is considered to be low.

Tsunamis, Seiches and Flooding

Tsunamis are large ocean waves generated by sudden water displacement caused by a submarine earthquake, landslide, or volcanic eruption. Review of the County of Los Angeles Flood and Inundation Hazards Map, (Leighton, 1990), indicates the site lies within the mapped tsunami inundation boundaries. However, the Seismic Safety Element of the City of Long Beach General Plan (1988) indicates the subject site is not located within the Tsunami inundation zone. A determination of whether a higher site elevation would remove the site from the potential inundation zone is beyond the scope of this investigation.

Seiches are oscillations generated in enclosed bodies of water which can be caused by ground shaking associated with an earthquake. Review of the County of Los Angeles Flood and Inundation Hazards Map, (Leighton, 1990) and the Seismic Safety Element of the City of Long Beach General Plan (1988), indicates the site lies within the mapped inundation boundaries of the Hansen Dam. A determination of whether a higher site elevation would remove the site from the potential inundation zones is beyond the scope of this investigation.

Landsliding

The topography onsite as well as in the immediate vicinity of the site is relatively level with very minor elevation changes. The County of Los Angeles Landslide Inventory Map (Leighton, 1990) and the Seismic Safety Element of the City of Long Beach General Plan indicate that the site is not located in an area highly susceptible to landslides. Additionally, the Seismic Hazards Zone Map of the Long Beach Quadrangle shows that the site is not located within an Earthquake-Induced Landslide Zone (CDMG, 1999). Based on these considerations, the potential of landslides negatively affecting the proposed development is considered to be low.

CONCLUSIONS AND RECOMMENDATIONS

Based upon the exploration, laboratory testing, and research, it is the finding of this firm the proposed residential complex is considered feasible from a geotechnical engineering standpoint, provided the advice and recommendations presented herein are followed and implemented during construction.

Fill materials were encountered during exploration at depths of 2.5 to 3 feet below the existing grade. The fill is underlain by native alluvial soils comprised of interlayered mixture of sand, silt, and clay. Groundwater was encountered during exploration at depths ranging from 4.5 to 8 feet below the ground surface.

The liquefaction analyses provided herein indicate that the underlying soil layers could be susceptible to liquefaction during a seismic event. These liquefiable soil layers occur at various depths between 4.5 and 60 feet below the ground surface. Total seismically-induced settlement on the order of 5.47 to 7.34 inches could occur as a result of liquefaction. Seismically-induced differential settlement of up to 3.67 inches could be expected. The analyses also indicate relative thicknesses of liquefiable soils to overlying non-liquefiable materials fall within the bounds where surface manifestations of liquefaction have been observed during past earthquakes.

It is recommended that ground improvement techniques be implemented at the subject site. The ground improvement should be designed to mitigate the potential of liquefaction in the upper site soils. It is recommended the ground improvement should extend from the ground surface to a minimum depth of 30 feet. This would be expected to reduce the potential for surface manifestations, loss of bearing strength, and liquefaction induced settlements at the site. Based on the CPT analyses, the recommended ground improvement would be expected to reduce the total and differential liquefaction induced settlement to approximately 3 and 2 inches, respectively. Additional recommendations are provided in the "Ground Improvement" section of this report.

Subsequent to the implementation of ground improvement, the proposed structure may be supported on a mat foundation. Support of the proposed structure in this manner would reduce the potential for damage should liquefaction occur but would not completely eliminate the potential for damage. The client should be aware that these design recommendations are not intended to prevent all structural damage to the proposed structure in the event of an earthquake, but to maintain the safety of the building occupants.

The upper site soils consist of consist of existing fill and native soils that should be expected to be soft and well above their optimum moisture level. The subgrade for the proposed mat foundation must be firm and unyielding. Subsequent to ground improvement, it should be anticipated that some preparation of the subgrade that will support the mat foundation will be necessary. This should be expected to include removal and recompaction of the upper 1 to 3 feet of soils underlying the mat foundation. Pumping and rutting of the soft and wet subgrade may occur under compaction efforts and/or construction related traffic. Should this occur, it will be necessary to stabilize the subgrade, as discussed in the "Wet Soils" section below. In either case, a representative of this office should observe the subgrade for the mat foundation as it becomes exposed so that recommendations may be provided as necessary.

Foundations for small outlying structures, such as planter walls or trash enclosures, which will not be tied-in to the proposed structure may be supported on conventional foundations bearing in native soils or properly placed compacted fill. The recommendations provided herein for these miscellaneous structures are not intended to mitigate the effects of liquefaction. The client should be aware that a liquefaction event on the subject site could result in severe and/or irreparable damage to the miscellaneous structures. Depending on the extent of the proposed ground improvement, these miscellaneous structures could also bear upon improved soils, which could substantially reduce liquefaction related damage to these structures.

The validity of the conclusions and design recommendations presented herein is dependent upon review of the geotechnical aspects of the proposed construction by this firm. The subsurface conditions described herein have been projected from excavations on the site as indicated and should in no way be construed to reflect any variations which may occur between these excavations or which may result from changes in subsurface conditions. Any changes in the design, as outlined in this report, should be reviewed by this office. The recommendations contained herein should not be considered valid until reviewed and modified or reaffirmed subsequent to such review.

GROUND IMPROVEMENT

The recommended ground improvement should be designed to mitigate the potential for liquefaction of the upper site soils, thus reducing the potential for surface manifestations, loss of bearing strength, and liquefaction induced settlements at the site. The applied ground improvement should extend to a minimum depth of 30 feet below the ground surface.

A qualified ground improvement contractor should be retained to aid in the selection and implementation of an appropriate ground improvement method. At this time, it is anticipated the most feasible ground improvement technique would consist of the installation of rammed aggregate piers.

Geotechnologies, Inc. must be consulted during the ground improvement method selection, design, and installation process. In addition, it may be necessary for the local building official to review and approve the proposed design. This office shall review and approve any ground improvement plans prior to implementation.

It will be necessary during construction to demonstrate through subsurface exploration, testing, and analysis that the applied ground improvement successfully achieved the recommended level of mitigation. It is recommended the construction schedule incorporate this requirement. Exploration, testing, and analysis of the improved subgrade soils will take several weeks to complete, and it may be necessary for the local building official to review and approve the analysis prior to foundation construction. In order to minimize delays to construction, it is suggested the exploration and testing be completed as soon as possible following the completion of ground improvement.

FILL SOILS

Fill was encountered during exploration to a maximum depth of 3 feet below the existing grade. This material and any fill generated during demolition should be removed and recompacted as controlled fill prior to foundation excavation or placement of floor slabs or additional fill.

EXPANSIVE SOILS

The onsite materials are in the very low expansion range. The Expansion Index was found to be 10 for a representative bulk sample of the upper site materials remolded to 90 percent of the laboratory maximum density. Recommended reinforcement for foundations and slabs-on-grade are provided in the subsequent sections of this report.

SOIL CORROSION POTENTIAL

Geotechnologies, Inc. does not practice in the field of corrosion engineering. Therefore, site samples were transported to the office of HDR, Engineering Inc. so that a corrosivity study could be performed.

Testing was performed on representative samples of the onsite soils at various depths. The electrical resistivity indicates that the soil is in the moderately corrosive category with respect to the field (in-situ) moisture condition. When saturated, the soil was in the severely corrosive category. Soil pH values varied from 7.8 to 8.9, indicating a mildly to strongly alkaline condition. Soil pH values exceeding 8.5 may be aggressive to aluminum. Nitrate was detected at low concentrations. The soluble salt content of the samples ranged from low to moderate.

Overall, the site soil is classified as severely corrosive to ferrous metals and aggressive to aluminum. Discussion of results and recommended mitigating measures are provided in the report from HDR Engineering, Inc. which has been enclosed in the Appendix of this report.

GRADING GUIDELINES

The following guidelines may be used in preparation of the grading plan and job specifications for any areas where fill or recompaction may be required, such as the building subgrade area, or driveway and sidewalk areas.

Site Preparation

All vegetation, existing fill, and soft or disturbed earth materials should be removed from the areas to receive controlled fill. The excavated area shall be observed by a representative of the geotechnical engineer prior to placing compacted fill.

It is very important that the position of the area to be graded is accurately located so that the grading operation proceeds efficiently.

Any vegetation or associated root system located within the area to be graded should be removed during grading. Any existing or abandoned utilities located within the area to be graded should be removed or relocated as appropriate. All fill materials and disturbed earth materials resulting from grading operations should be removed and properly recompacted.

Subsequent to the indicated removals, the exposed grade shall be scarified to a depth of 6 inches, moistened to optimum moisture content, and recompacted in excess of the minimum required comparative density.

Compaction

Fill, consisting of soil approved by a representative of this firm shall be placed in loose lifts not more than 8 inches in thickness. The loose materials shall be compacted with suitable compaction equipment. Once a layer has been adequately compacted, the next loose lift may be placed.

Fill materials shall be moisture conditioned to within 3 percent of optimum moisture content and sufficiently blended prior to placement as controlled fill. Materials larger than 6 inches in maximum dimension shall not be used in the fill.

All fill shall be compacted to at least 90 percent of the maximum laboratory density. The maximum density shall be determined by the laboratory operated by Geotechnologies, Inc., using test method ASTM D 1557 or equivalent.

Field observation and testing shall be performed by a representative of the geotechnical engineer during grading to assist the contractor in obtaining the required degree of compaction and the proper moisture content. Where compaction is less than required, additional compactive effort

shall be made with adjustment of the moisture content, as necessary, until a minimum of 90 percent compaction is obtained.

Acceptable Materials

The excavated onsite materials are considered satisfactory for reuse in the controlled fills as long as any debris and/or organic matter is removed. Any imported materials shall be observed and tested by the representative of the geotechnical engineer prior to use in fill areas. Imported materials should contain sufficient fines so as to be relatively impermeable and result in a stable subgrade when compacted. Any required import materials should consist of geologic materials with an expansion index of less than 20. The water-soluble sulfate content of the import materials should be less than 0.1% percentage by weight.

Imported materials should be free from chemical or organic substances which could affect the proposed development. A competent professional should be retained in order to test imported materials and address environmental issues and organic substances which might affect the proposed development.

Wet Soils

Most of the soils encountered during exploration of the site were well above their optimum moisture levels. Some drying, aeration, and processing of the onsite soils should be anticipated prior to their use and placement as compacted fill. Pumping and rutting of the subgrade may occur under compaction efforts and/or construction related traffic.

Pumping and rutting of the subgrade will disturb the subgrade soils, resulting in an unstable subgrade that is not suitable for placement of compacted fill or support of overlying improvements. It is recommended that care be exercised as the subgrade becomes exposed in order to establish if mitigative action will be required.

Exposed subgrades must be suitable for placement of compacted fill and support of overlying improvements. If pumping, rutting, or other disturbance of the subgrade occurs during construction, it is recommended a minimum 1-foot thick layer of angular 1 to 3-inch gravel be placed and vibrated to a dense state. The gravel will function as a stabilization material upon which heavy equipment may operate and provide a firm bottom to compact soil against. It is not recommended that rubber tire construction equipment attempt to operate directly on the subgrade soils prior to placing the gravel. Direct operation of rubber tire equipment on the soft subgrade soils will likely result in excessive disturbance to the soils, which in turn will result in a delay to the construction schedule since those disturbed soils would then have to be removed and properly recompacted. Extreme care should be utilized to place gravel as the subgrade becomes exposed.

The client should be aware that subgrade stabilization with gravel is a trial and error process. The amount of gravel required for stabilization is determined in the field during placement. However, the thickness of the gravel layer could possibly be reduced by incorporating a geotextile material, such as Tensar Geogrid.

Utility Trench Backfill

Utility trenches should be backfilled with controlled fill. The utility should be bedded with clean sands at least one foot over the crown. The remainder of the backfill may be onsite soil compacted to 90 percent of the laboratory maximum density. Utility trench backfill should be tested by representatives of this firm in general accordance with the most recent revision of ASTM D 1557.

Shrinkage

Shrinkage results when a volume of soil removed at one density is compacted to a higher density. A shrinkage factor between 10 and 15 percent should be anticipated when excavating and

recompacting the existing fill and underlying native geologic materials on the site to an average comparative compaction of 92 percent.

Weather Related Grading Considerations

When rain is forecast all fill that has been spread and awaits compaction shall be properly compacted prior to stopping work for the day or prior to stopping due to inclement weather. These fills, once compacted, shall have the surface sloped to drain to an area where water can be removed.

Temporary drainage devices should be installed to collect and transfer excess water to the street in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. Drainage should not be allowed to flow uncontrolled over any descending slope.

Work may start again, after a period of rainfall, once the site has been reviewed by a representative of this office. Any soils saturated by the rain shall be removed and aerated so that the moisture content will fall within three percent of the optimum moisture content.

Surface materials previously compacted before the rain shall be scarified, brought to the proper moisture content and recompacted prior to placing additional fill, if considered necessary by a representative of this firm.

Geotechnical Observations and Testing During Grading

Geotechnical observations and testing during grading are considered to be a continuation of the geotechnical investigation. It is critical that the geotechnical aspects of the project be reviewed by this firm during the construction process. Compliance with the design concepts, specifications or recommendations during construction requires review by this firm during the course of construction. Any fill which is placed should be observed, tested, and verified if used for

engineered purposes. Please advise this office at least twenty-four hours prior to any required site visit.

FOUNDATION DESIGN

Mat Foundation

Following completion of the recommended ground improvement and verification that the recommended level of mitigation has been achieved, the proposed structure may be supported on a mat foundation bearing upon a newly compacted fill pad. The bearing capacity and subgrade modulus provided herein should be reviewed, reconfirmed, and revised (if necessary) following exploration, testing, and analysis of the improved subgrade soils.

The average bearing pressure below the proposed mat foundation is anticipated to be on the order of 500 to 750 pounds per square foot, with locally higher concentrated loads below the core up to 1,500 pounds per square foot. For design purposes, an average allowable bearing capacity of 1,000 pounds per square foot may be utilized in the design of the proposed mat foundation. For initial design purposes, the mat foundation may be designed utilizing a modulus of subgrade reaction of 200 pounds per cubic inch. This value should be multiplied by a reduction factor of:

$$K_{\mathrm{R}} = K\left[\frac{B+1}{2B}\right]^2$$

Where:

K = Unit subgrade Modulus

K_R = Reduced Subgrade Modulus

B = Equivalent Foundation width

The bearing capacity indicated above is for the total of dead and frequently applied live loads and may be increased by one third for short duration loading, which includes the effects of wind or seismic forces. Since the recommended bearing value is a net value, the weight of concrete in the foundations may be taken as 50 pounds per cubic foot and the weight of the soil backfill may be neglected when determining the downward load on the foundations.

Hydrostatic Considerations

Groundwater was observed during exploration of the site at depths of 4.5 to 8 feet below the existing grade. These levels have the potential to fluctuate over time. The proposed residential complex will be an at-grade structure; however, if the bottom of the proposed foundation extends below the groundwater level, the project structural engineer shall evaluate and design the proposed foundation for hydrostatic uplift pressure. Hydrostatic uplift pressure would be equivalent to 62.4(H), where "H" is the vertical distance between the bottom of the mat foundation and the groundwater level. In any case, it is recommended that the proposed mat foundation be waterproofed.

Lateral Design

Resistance to lateral loading may be provided by friction acting at the base of foundations and by passive geologic pressure. An allowable coefficient of friction of 0.25 may be used with the dead load forces.

Passive geologic pressure for the sides of the proposed foundation poured against undisturbed or recompacted soil may be computed as an equivalent fluid having a density of 200 pounds per cubic foot with a maximum earth pressure of 2,000 pounds per square foot. The passive and friction components may be combined for lateral resistance without reduction. A one-third increase in the passive value may be used for short duration loading such as wind or seismic forces.

Foundation Settlement

The static settlement of the proposed mat foundation is expected to occur on initial application of loading. The maximum static settlement is not expected to exceed 1 inch. Differential settlement is not expected exceed ¼ inch.

In addition to static settlement, the proposed foundation may be subject to seismically-induced settlement as a result of liquefaction. Following the implementation of the recommended ground improvement measures discussed above, total and differential seismically-induced settlements would be anticipated to be approximately 3 and 2 inches, respectively. The degree of static and seismic settlements reported herein additive.

Miscellaneous Foundations

Conventional foundations may be used to support miscellaneous structures as privacy walls or trash enclosures which will not be rigidly connected to the proposed residential complex. These foundations should bear in native geologic materials or properly compacted fill. Continuous footings may be designed for a bearing capacity of 1,500 pounds per square foot and should be a minimum of 12 inches in width, 18 inches in depth below lowest adjacent grade, and 18 inches into the recommended bearing material. Bearing capacity increases are not recommended.

Foundation Observations

It is critical that all foundation excavations are observed by a representative of this firm to verify penetration into the recommended bearing materials. The observation should be performed prior to the placement of reinforcement. Foundations should be deepened to extend into satisfactory earth materials, if necessary. Foundation excavations should be cleaned of all loose soils prior to placing steel and concrete. Any required foundation backfill should be mechanically compacted, flooding is not permitted.

RETAINING WALL DESIGN

Cantilever Retaining Walls

The proposed structure is expected to be built at or near existing grade. However, miscellaneous site retaining walls on the order of 6 feet in height may be required as part of the proposed development. Retaining walls supporting a level backslope may be designed utilizing a triangular distribution of pressure. Cantilever retaining walls may be designed for 45 pounds per cubic foot for walls retaining up to 6 feet of earth.

For this equivalent fluid pressure to be valid, walls which are to be restrained at the top should be backfilled prior to the upper connection being made. Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures. The lateral earth pressure recommended above assumes that a permanent drainage system will be installed so that external water pressure will not be developed against the walls.

Dynamic (Seismic) Earth Pressure

Retaining walls greater than 6 feet in height shall be designed to resist the additional earth pressure caused by seismic ground shaking. This office should be notified if retaining walls greater than 6 feet in height become part of the proposed development, and additional recommendations will be provided.

Waterproofing

Moisture affecting retaining walls is one of the most common post- construction complaints. Poorly applied or omitted waterproofing can lead to effloresence or standing water. Effloresence is a process in which a powdery substance is produced on the surface of the concrete by the

evaporation of water. The white powder usually consists of soluble salts such as gypsum, calcite, or common salt.

It is recommended that retaining walls be waterproofed. Waterproofing design and inspection of its installation is not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained in order to recommend a product or method which would provide protection to below grade walls.

Retaining Wall Drainage

Retaining walls should be provided with a subdrain or weepholes covered with a minimum of 12 inches of gravel, and a compacted fill blanket or other seal at the surface. Certain types of subdrain pipe are not acceptable to the various municipal agencies. It is recommended that prior to purchasing subdrainage pipe, the type and brand is cleared with the proper municipal agencies. Subdrainage pipes should outlet to an acceptable location.

The earth pressures recommended herein for retaining walls assume that a permanent drainage system will be installed so that external water pressure will not be developed against the walls. If a drainage system is not provided, the walls should be designed to resist the indicated earth pressure plus hydrostatic pressure. The hydrostatic pressure based on a water level at the ground surface.

Retaining Wall Backfill

Any required backfill should be mechanically compacted in layers not more than 8 inches thick, to at least 90 percent of the maximum density obtainable by the ASTM Designation D 1557 method of compaction. Flooding should not be permitted. Proper compaction of the backfill will be necessary to reduce settlement of the backfill and to reduce settlement of overlying walks and

paving. Some settlement of required backfill should be anticipated, and any utilities supported therein should be designed to accept differential settlement.

TEMPORARY EXCAVATIONS

Excavations on the order of 5 feet in vertical height are anticipated for the proposed development. The excavations are expected to expose fill and native soils, which are suitable for vertical excavations up to 5 feet where not surcharged by adjacent traffic or structures. Open excavations should not be made below the groundwater level. Caving should be expected in any excavation or drilled shaft extending below the groundwater.

Where sufficient space is available, temporary unsurcharged embankments may be sloped back at a uniform 1:1 (h:v) gradient in its entirety. A uniform sloped excavation is sloped from bottom to top and does not contain a vertical component. Sloped excavations with vertical cuts at the toe of the slope are not recommended.

Where sloped embankments are utilized, the tops of the slopes should be barricaded to prevent vehicles and storage loads near the top of slope within a horizontal distance equal to the depth of the excavation. If the temporary construction embankments are to be maintained during the rainy season, berms are suggested along the tops of the slopes where necessary to prevent runoff from entering the excavation and eroding the slope faces. The soils exposed in the cut slopes should be inspected during excavation by personnel from this office so that modifications of the slopes can be made if variations in the soil conditions occur.

Excavations Observations

It is critical that the soils exposed in the cut slopes are observed by a representative of Geotechnologies, Inc. during excavation so that modifications of the slopes can be made if variations in the geologic material conditions occur. Many building officials require that temporary

excavations should be made during the continuous observations of the geotechnical engineer. All excavations should be stabilized within 30 days of initial excavation.

SLABS ON GRADE

If desired, interior concrete floor slabs may be cast above properly compacted fill materials placed above the proposed mat foundations. Interior floor slabs should be a minimum of 4 inches in thickness and reinforced with a minimum of #3 steel bars on 18-inch centers each way. All fill shall be compacted to a minimum of 90 percent of the maximum dry density.

Outdoor Concrete Flatwork

Outdoor concrete flatwork, such as sidewalks and patio areas, should be a minimum of 4 inches in thickness and may be cast over properly controlled fill materials. Any earth materials loosened or over-excavated should be wasted from the site or properly compacted to 90 percent of the maximum dry density. Exterior flatwork should be reinforced with a minimum of #3 steel bars on 18-inch centers each way.

Exterior Concrete Pavements

Exterior concrete pavement subject to passenger vehicle and truck traffic should be a minimum of 6 inches in thickness and reinforced with a minimum of #3 steel bars on 18-inch centers each way. The concrete pavement should be underlain by 8 inches of aggregate base. A subgrade modulus of 100 pounds per cubic inch may be assumed for design of concrete paving.

Aggregate base should be compacted to a minimum of 95 percent of the ASTM D 1557 laboratory maximum dry density. Base materials should conform with Sections 200-2.2 or 200-2.4 of the "Standard Specifications for Public Works Construction", (Green Book), Current Edition.

Design of Slabs That Receive Moisture-Sensitive Floor Coverings

Geotechnologies, Inc. does not practice in the field of moisture vapor transmission evaluation and mitigation. Therefore, it is recommended that a qualified consultant be engaged to evaluate the general and specific moisture vapor transmission paths and any impact on the proposed construction. The qualified consultant should provide recommendations for mitigation of potential adverse impacts of moisture vapor transmission on various components of the structure.

All concrete slabs-on-grade should be supported on vapor retarder. The design of the slab and the installation of the vapor retarder should comply with the most recent revisions of ASTM E 1643 and ASTM E 1745. The vapor retarder should comply with ASTM E 1745 Class A requirements. Vapor barrier is not required for exposed concrete slabs such as garage slab-on-grade.

Where a vapor retarder is used, a low-slump concrete should be used to minimize possible curling of the slabs. The barrier can be covered with a layer of trimmable, compactible, granular fill, where it is thought to be beneficial. See ACI 302.2R-32, Chapter 7 for information on the placement of vapor retarders and the use of a fill layer.

Where dampness would be objectionable, it is recommended that the floor slabs should be waterproofed. A qualified waterproofing consultant should be retained in order to recommend a product or method which would provide protection for concrete slabs-on-grade.

Concrete Crack Control

The recommendations presented in this report are intended to reduce the potential for cracking of concrete slabs-on-grade due to settlement. However even where these recommendations have been implemented, foundations, stucco walls and concrete slabs-on-grade may display some cracking due to minor soil movement and/or concrete shrinkage. The occurrence of concrete cracking may be reduced and/or controlled by limiting the slump of the concrete used, proper concrete placement

and curing, and by placement of crack control joints at reasonable intervals, in particular, where re-entrant slab corners occur.

For standard control of concrete cracking, a maximum crack control joint spacing of 12 feet should not be exceeded. Lesser spacing would provide greater crack control. Joints at curves and angle points are recommended. The crack control joints should be installed as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. Construction joints should be designed by a structural engineer.

Complete removal of the existing fill soils beneath outdoor flatwork such as walkways or patio areas, is not required, however, due to the rigid nature of concrete, some cracking, a shorter design life and increased maintenance costs should be anticipated. In order to provide uniform support beneath the flatwork it is recommended that a minimum of 12 inches of the exposed subgrade beneath the flatwork be scarified and recompacted to 90 percent relative compaction.

ASPHALT PAVEMENTS

It is recommended that the existing fill materials be removed and recompacted for the support of asphalt pavements that will be subjected to vehicle traffic. The client should be aware that removal of all existing fill in the area of new paving is not required, however, pavement constructed in this manner will most likely have a shorter design life and increased maintenance costs. As a minimum, it is recommended the upper 18 inches of the paving subgrade be removed and recompacted to a minimum of 90 percent of the maximum density as determined by ASTM D 1557.

Due to a wide variation which may occur during the grading process, it is recommended that R-value tests be performed near the end of grading in order to verify the subgrade condition prior to paving. The recommended paving section should be considered preliminary and is subject to revision. However, for preliminary design purposes, an R-value of 20 was assumed for the design of the proposed paving section. The following pavement sections are recommended:

Vehicular Service - Assumed Traffic Index (T.I.)	Asphalt Pavement Thickness (inches)	Thickness of Aggregate Base (inches)
Passenger Cars - 4.0	4	6
Moderate Trucks - 6.0	5	9

Aggregate base should be compacted to a minimum of 95 percent of the ASTM D 1557 laboratory maximum dry density. Base materials should conform with Sections 200-2.2 or 200-2.4 of the "Standard Specifications for Public Works Construction", (Green Book), current edition.

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

SITE DRAINAGE

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

All site drainage should be collected and transferred to an acceptable location in non-erosive drainage devices. Structures should be provided with roof drainage. Discharge from downspouts, roof drains and scuppers should not be permitted on unprotected soils within five feet of building perimeters. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. Drainage should not be allowed to flow uncontrolled over any descending slope. Planters which are located within retaining wall backfill should be sealed to prevent moisture intrusion into the backfill.

STORMWATER DISPOSAL

Regulatory agencies have been requiring the disposal of a certain amount of stormwater generated on a site by infiltration into the site soils. This requirement is not prudent engineering practice. Increasing the moisture content of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the designed engineering properties. This means that any overlying structure, including buildings, pavements and concrete flatwork, could sustain damage due to saturation of the subgrade soils. Structures serviced by subterranean levels, like the one proposed for this project, could be adversely impacted by stormwater disposal by increasing the design fluid pressures on retaining walls, hydrostatic uplift of the foundation, or causing leaks in the walls. Proper site drainage is critical to the performance of any structure in the built environment.

Groundwater was encountered during exploration at relatively shallow depths ranging from 4.5 to 8 feet below the ground surface. As a result, the vertical clearance between the existing groundwater level for potential stormwater infiltration is very limited. Additionally, the soils underlying the site are susceptible to liquefaction when saturated. Based on these considerations, stormwater infiltration is not recommended for the site.

Where infiltration of stormwater into the subgrade soils is not advisable, some Building Officials have allowed the stormwater to be filtered through soils in planter areas. Once the water has been filtered through a planter it may be released into a storm drain system. It is recommended that overflow pipes are incorporated into the design of the discharge system in the planters to prevent flooding. In addition, the planters shall be sealed and waterproofed to prevent leakage. Please be advised that adverse impact to landscaping and periodic maintenance may result due to excessive water and contaminates discharged into the planters.

DESIGN REVIEW

Engineering of the proposed project should not begin until approval of the geotechnical report by the Building Official is obtained in writing. Significant changes in the geotechnical recommendations may result during the building department review process.

It is recommended that the geotechnical aspects of the project be reviewed by this firm during the design process. This review provides assistance to the design team by providing specific recommendations for particular cases, as well as review of the proposed construction to evaluate whether the intent of the recommendations presented herein are satisfied.

CONSTRUCTION MONITORING

Geotechnical observations and testing during construction are considered to be a continuation of the geotechnical investigation. It is critical that this firm review the geotechnical aspects of the project during the construction process. Compliance with the design concepts, specifications or recommendations during construction requires review by this firm during the course of construction. All foundations should be observed by a representative of this firm prior to placing concrete or steel. Any fill which is placed should be observed, tested, and verified if used for engineered purposes. Please advise this office at least twenty-four hours prior to any required site visit.

If conditions encountered during construction appear to differ from those disclosed herein, notify this office immediately so the need for modifications may be considered in a timely manner.

It is the responsibility of the contractor to ensure that all excavations and trenches are properly sloped or shored. All temporary excavations should be cut and maintained in accordance with applicable OSHA rules and regulations.

EXCAVATION CHARACTERISTICS

The exploration performed for this investigation is limited to the geotechnical excavations described. Direct exploration of the entire site would not be economically feasible. The owner, design team and contractor must understand that differing excavation and drilling conditions may be encountered based on boulders, gravel, oversize materials, groundwater and many other conditions. Fill materials, especially when they were placed without benefit of modern grading codes, regularly contain materials which could impede efficient grading and drilling. Southern California sedimentary bedrock is known to contain variable layers which reflect differences in depositional environment. Such layers may include abundant gravel, cobbles and boulders. Similarly, bedrock can contain concretions. Concretions are typically lenticular and follow the bedding. They are formed by mineral deposits. Concretions can be very hard. Excavation and drilling in these areas may require full size equipment and coring capability. The contractor should be familiar with the site and the geologic materials in the vicinity.

CLOSURE AND LIMITATIONS

The purpose of this report is to aid in the design and completion of the described project. Implementation of the advice presented in this report is intended to reduce certain risks associated with construction projects. The professional opinions and geotechnical advice contained in this report are sought because of special skill in engineering and geology and were prepared in accordance with generally accepted geotechnical engineering practice. Geotechnologies, Inc. has a duty to exercise the ordinary skill and competence of members of the engineering profession. Those who hire Geotechnologies, Inc. are not justified in expecting infallibility, but can expect reasonable professional care and competence.

The scope of the geotechnical services provided did not include any environmental site assessment for the presence or absence of organic substances, hazardous/toxic materials in the soil, surface water, groundwater, or atmosphere, or the presence of wetlands.

Proper compaction is necessary to reduce settlement of overlying improvements. Some settlement of compacted fill should be anticipated. Any utilities supported therein should be designed to accept differential settlement. Differential settlement should also be considered at the points of entry to the structure.

GEOTECHNICAL TESTING

Classification and Sampling

The soil is continuously logged by a representative of this firm and classified by visual examination in accordance with the Unified Soil Classification System. The field classification is verified in the laboratory, also in accordance with the Unified Soil Classification System. Laboratory classification may include visual examination, Atterberg Limit Tests and grain size distribution. The final classification is shown on the boring logs.

Samples of the geologic materials encountered in the borings were collected and transported to the laboratory. Undisturbed samples of soil are obtained at frequent intervals. Unless noted on the boring logs as an SPT sample, samples acquired while utilizing a mud rotary drill rig are obtained by driving a thin walled, California Modified Sampler with successive 30-inch drops of a 140-pound automatic hammer. The soil is retained in brass rings of 2.50 inches inside diameter and 1.00 inches in height. The central portion of the samples are stored in close fitting, waterproof containers for transportation to the laboratory. Samples noted on the boring logs as SPT samples are obtained in accordance with ASTM D 1586. An automatic trip hammer is utilized to drive both SPT and ring samples. Samples are retained for 30 days after the date of the geotechnical report.

Moisture and Density Relationships

The field moisture content and dry unit weight are determined for each of the undisturbed soil samples, and the moisture content is determined for SPT samples by ASTM D 4959 or ASTM D

4643. This information is useful in providing a gross picture of the soil consistency between exploration locations and any local variations. The dry unit weight is determined in pounds per cubic foot and shown on the “Boring Logs”, A-Plates. The field moisture content is determined as a percentage of the dry unit weight.

Direct Shear Testing

Shear tests are performed by ASTM D 3080 with a strain controlled, direct shear machine manufactured by GeoMatic, Inc. The rate of deformation ranges between approximately 0.005 and 0.025 inches per minute, depending on the nature of the sample to be sheared. Each sample is sheared under varying confining pressures in order to determine the Mohr Coulomb shear strength parameters of the cohesion intercept and the angle of internal friction. Samples are generally tested in an artificially saturated condition. Depending upon the sample location and future site conditions, samples may be tested at field moisture content. The results are plotted on the "Shear Test Diagrams," B Plates.

Consolidation Testing

Settlement predictions of the soil's behavior under load are made on the basis of the consolidation tests ASTM D 2435. The consolidation apparatus is designed to receive a single one-inch high ring. Loads are applied in several increments in a geometric progression, and the resulting deformations are recorded at selected time intervals. Porous stones are placed in contact with the top and bottom of each specimen to permit addition and release of pore fluid. Samples are generally tested at increased moisture content to determine the effects of water on the bearing soil. The normal pressure at which the water is added is noted on the drawing. Results are plotted on the "Consolidation Test," C Plates.

Expansion Index Testing

The expansion tests performed on samples are in accordance with the Expansion Index testing procedures, as described in the ASTM D4829. The soil sample is compacted into a metal ring at a saturation degree of 50 percent. The ring sample is then placed in a consolidometer, under a vertical confining pressure of 1 lbf/square inch and inundated with distilled water. The deformation of the specimen is recorded for a period of 24 hours or until the rate of deformation becomes less than 0.0002 inches/hour, whichever occurs first. The expansion index, EI, is determined by dividing the difference between final and initial height of the ring sample by the initial height, and multiplied by 1,000. Expansion test results are shown on the enclosed Plate D.

Grain Size Distribution

These tests cover the quantitative determination of the distribution of particle sizes in soils. Sieve analysis is used to determine the grain size distribution of the soil larger than the Number 200 sieve. ASTM D 422-63 is used to determine particle sizes smaller than the Number 200 sieve. A hydrometer is used to determine the distribution of particle sizes by a sedimentation process. Hydrometer testing was not performed as part of this investigation. Particle size determination for this investigation utilized the Number 200 sieve. The results are plotted on the E Plates presented in the Appendix of this report.

Atterberg Limits

Depending on their moisture content, cohesive soils can be either solid, plastic, or liquid. The water contents corresponding to the transitions from solid to plastic or plastic to liquid are known as the Atterberg Limits. The transitions are called the plastic limit and liquid limit. The difference between the liquid and plastic limits is known as the plasticity index. ASTM D 4318 is utilized to determine the Atterberg Limits. The results are shown on the enclosed F-Plates.

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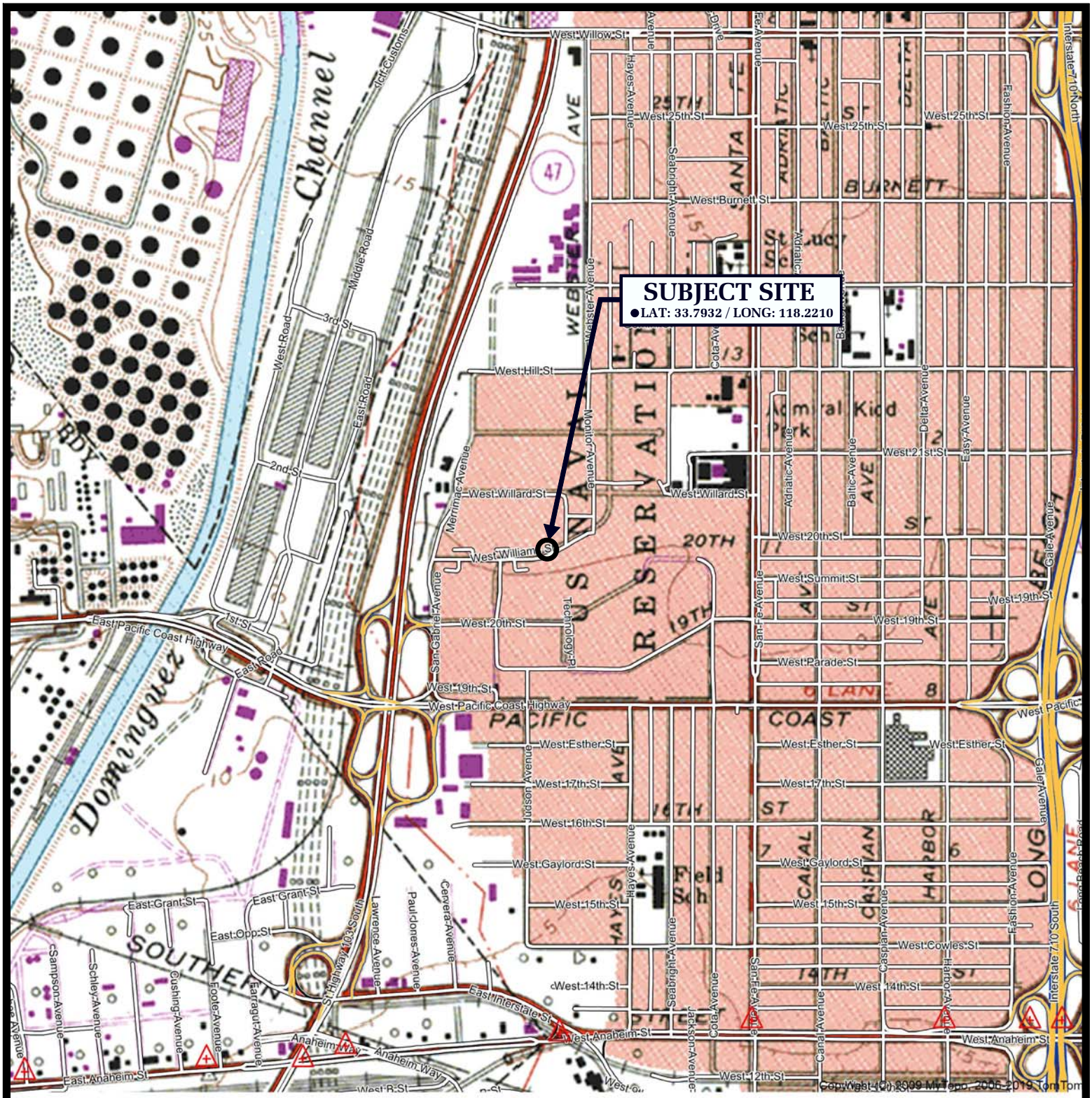
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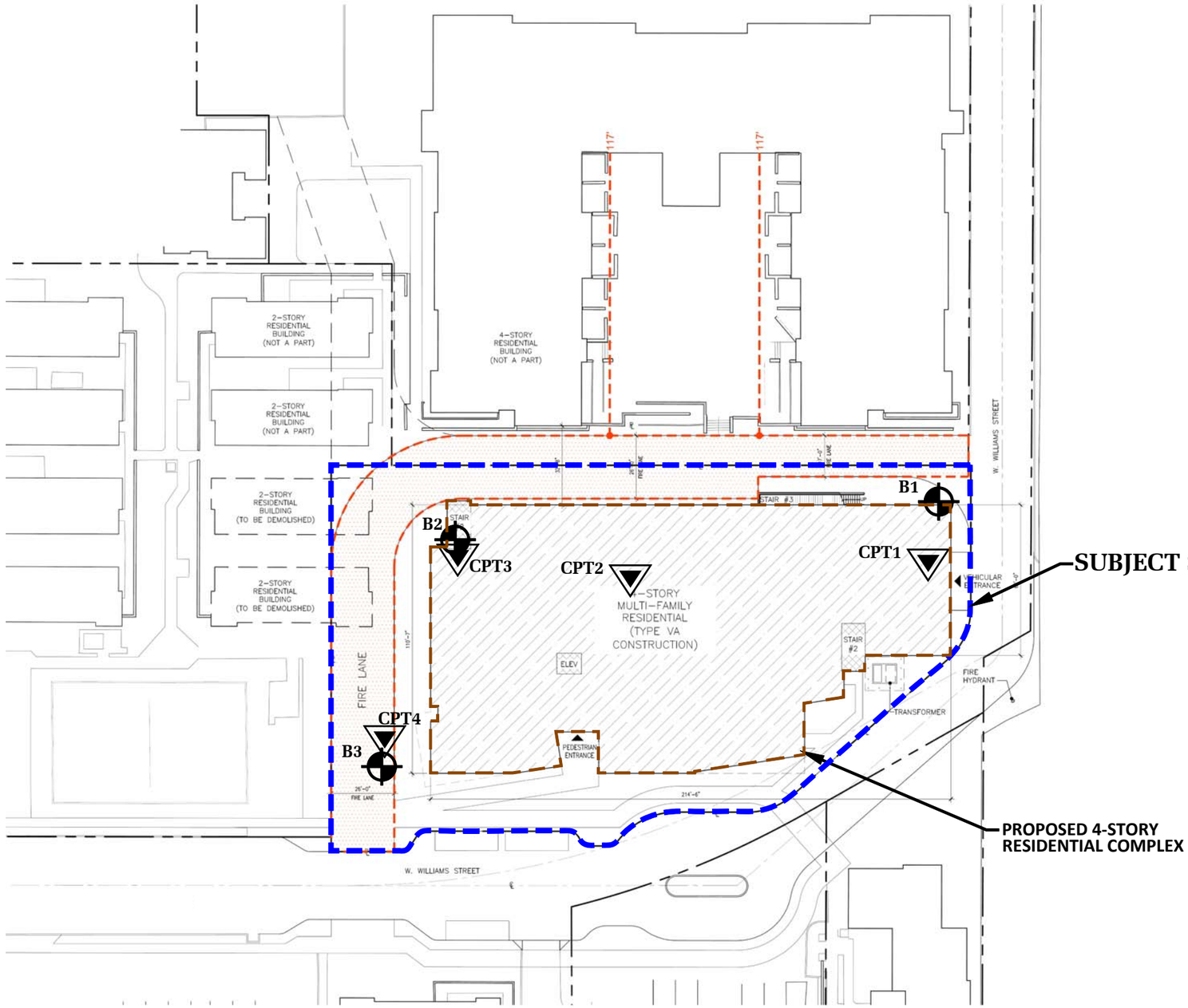
REFERENCE: U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, LONG BEACH, CA QUADRANGLE

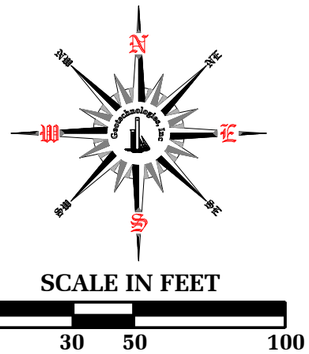
VICINITY MAP

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Consulting Geotechnical Engineers

CENTURY HOUSING
2221 W. WILLIAMS ST., LONG BEACH

FILE NO. 21855





LEGEND

B3 — LOCATION & NUMBER OF BORING

CPT4 — LOCATION & NUMBER OF CONE PENETROMETER TEST PERFORMED BY GEOTECHNOLOGIES, INC.

REFERENCE: SITE PLAN PROVIDED BY THE ARCHITECTS COLLECTIVE DATED JUNE 4, 2019

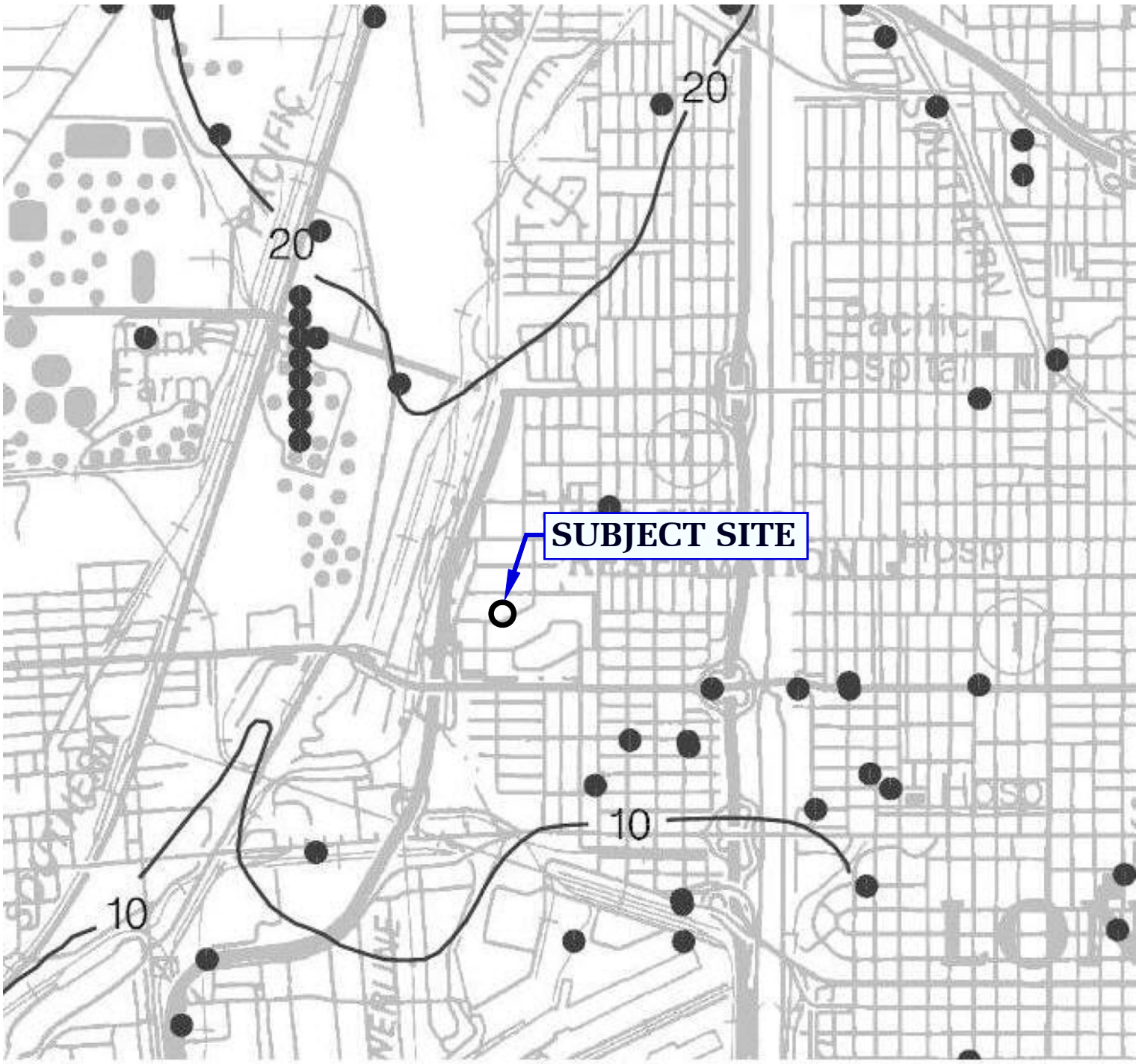
PLOT PLAN

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CENTURY HOUSING
2221 W. WILLIAMS ST., LONG BEACH

FILE No. 21855 | DRAWN BY: TC

DATE: September 2019



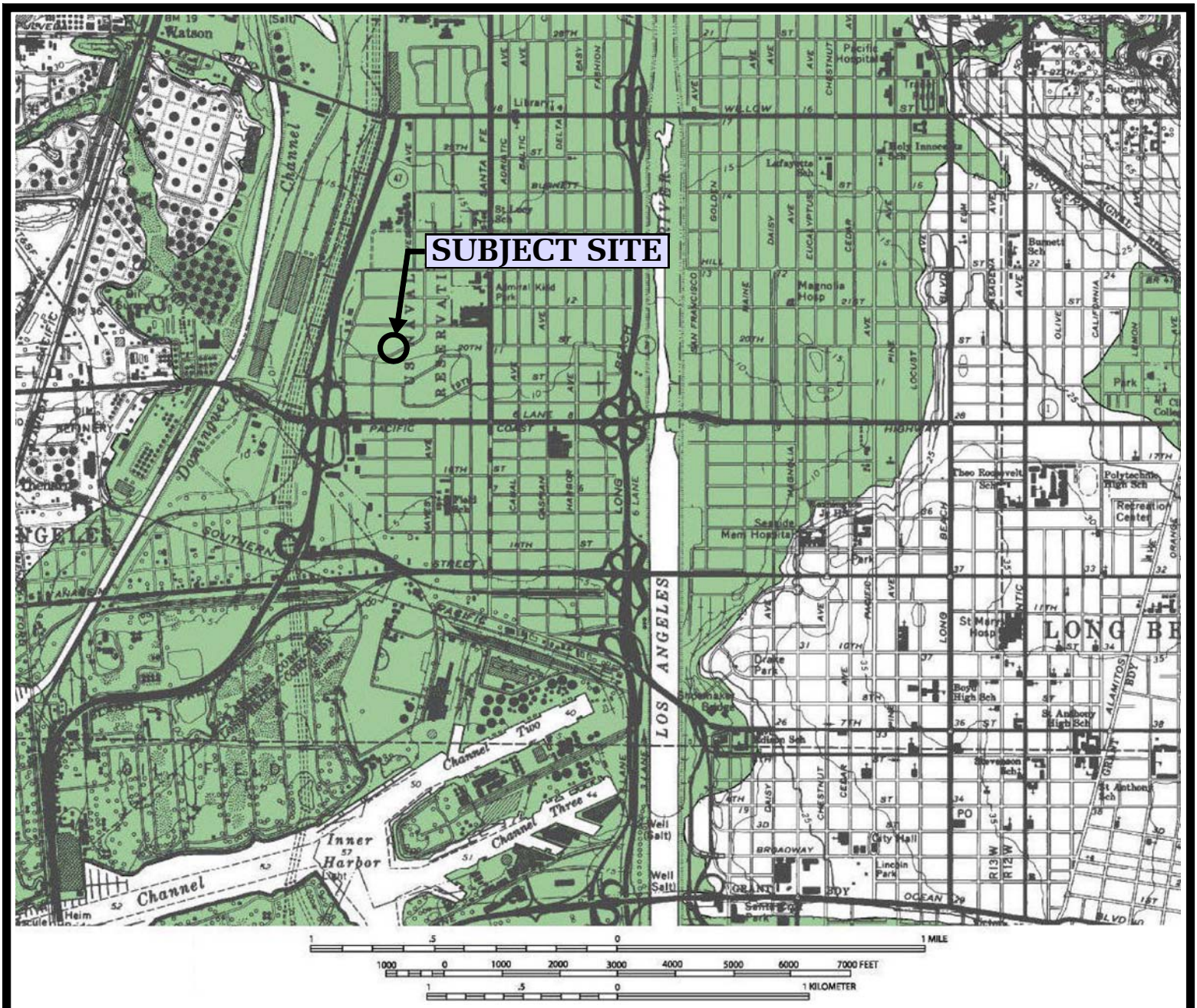
REFERENCE: CDMG, SEISMIC HAZARD ZONE REPORT, 028
LONG BEACH 7.5 - MINUTE QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA (1998, REVISED 2006)

HISTORICALLY HIGHEST GROUNDWATER LEVELS

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CENTURY HOUSING
2221 W. WILLIAMS ST., LONG BEACH

FILE NO. 21855



LIQUEFACTION AREA

REFERENCE: SEISMIC HAZARD ZONES, LONG BEACH QUADRANGLE OFFICIAL MAP (CDMG, 1999)

SEISMIC HAZARD ZONE MAP

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CENTURY HOUSING
2221 W. WILLIAMS ST., LONG BEACH

FILE NO. 21855

BORING LOG NUMBER 1

Century Housing **Date: 08/21/19**

File No. 21855 **Method: 5-inch diameter Rotary Wash Drill Rig**

dy

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 --		4-inch Asphalt over 6-inch Base
				-		
				1 --		FILL: Silty Sand, dark yellowish brown, moist, medium dense, fine grained
				-		
				2 --		
2.5	13	15.4	105.3	-		
				3 --		
				-	CL	ALLUVIUM: Silty Clay, grayish brown, very moist, loose, soft fine grained, minor fine grained sand
				4 --		
				-		
5	2	44.4	SPT	5 --		
				-		
				6 --		
				-		
				7 --		
7.5	PUSH	No Recovery		-		
				8 --		
				-		
				9 --		
				-		
10	3	32.3	SPT	10 --		dark yellowish brown
				-		
				11 --		
				-		
				12 --		
12.5	4	35.3	85.9	-		
				13 --		
				-		
				14 --		
				-		
15	15	23.9	SPT	15 --		
				-	SP	Sand, dark yellowish brown, wet, medium dense, fine grained
				16 --		
				-		
				17 --		
17.5	8	32.5	90.7	-		
				18 --	CL	Silty Clay, grayish brown with orange mottling, wet, medium stiff, fine grained sand intermixed
				-		
				19 --		
				-		
20	8	30.7	SPT	20 --		
				-		
				21 --		
				-		
				22 --		
22.5	5	34.4	84.6	-		
				23 --		gray, softer consistency
				-		
				24 --		
				-		
25	2	51.1	SPT	25 --		
				-		

BORING LOG NUMBER 1

Century Housing

File No. 21855

dy

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26		
				27		
27.5	8	35.0	81.3	28		Silty Clay, gray, wet, soft, some fine grained sand
				29		
30	6	25.3	SPT	30		
				31		
32.5	18	24.7	102.0	32		
				33		
				34		
35	7	38.4	SPT	35	CH	Silty Clay, gray, wet, medium stiff
				36		
37.5	17	38.8	79.7	37		
				38		
				39		
40	22	29.5	SPT	40	ML	Sandy Silt, gray, wet, very stiff
				41		
42.5	40	28.6	99.7	42		
				43		
				44		
45	19	41.0	SPT	45	ML	Clayey Silt, gray, wet, very stiff
				46		
47.5	32 50/5"	28.2	96.1	47		
				48	SM	Silty Sand, gray, wet, dense, fine grained
				49		
50	33	23.5	SPT	50		

BORING LOG NUMBER 1

Century Housing

File No. 21855

dy

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				- 51 -- - 52 --		
52.5	66	23.4	101.6	- 53 -- - 54 --		
55	31	24.6	SPT	- 55 -- - 56 --		
57.5	44	19.6	106.8	- 57 -- - 58 --	ML/SP	Sandy Silt to Sand, gray, wet, dense, very stiff, fine to medium grained, transitions from primarily fine grained silty material to granular sandy material
60	36	21.0	SPT	- 59 -- - 60 --	SP	Sand, gray, wet, dense, fine to medium grained
62.5	42	32.7	90.8	- 61 -- - 62 --	ML	Sandy Silt, gray, wet, very stiff
65	52	27.2	SPT	- 63 -- - 64 --	SP	Sand, gray, wet, very dense, fine grained
67.5	77	21.6	105.2	- 65 -- - 66 --		
70	57	23.3	SPT	- 67 -- - 68 --		
				- 69 -- - 70 --		Total Depth 70 feet Water at 8 feet Fill to 3 feet
				- 71 -- - 72 --		
				- 73 -- - 74 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				- 75 -- -		Used 5-inch diameter Rotary Wash Drill Rig 140 lb. Automatic Hammer, 30-inch drop Modified California Sampler unless otherwise noted
						SPT = Standard Penetration Test

BORING LOG NUMBER 2

Century Housing **Date: 08/23/19**

File No. 21855 **Method: 5-inch diameter Rotary Wash Drill Rig**

dy

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 --		4-inch Asphalt over 6-inch Base
				-		
				1 --		FILL: Silty Sand, mottled grayish to yellowish brown, moist, loose, fine grained, minor clay intermixed
				-		
2.5	7	28.9	88.5	2 --		
				-		
				3 --	CL	ALLUVIUM: Silty Clay, mottled yellowish brown, moist, soft, minor fine grained sand
				-		
				4 --		
				-		
5	PUSH	43.5	SPT	5 --		
				-		
				6 --		
				-		
				7 --		
7.5	2	44.4	Disturbed	-		
				8 --		
				-		
				9 --		
				-		
10	4	33.1	SPT	10 --		mottled grayish brown, wet
				-		
				11 --		
				-		
				12 --		
12.5	4	47.3	80.3	-		
				13 --		
				-		
				14 --		
				-		
15	8	24.4	SPT	15 --	SC	Clayey Sand, mottled grayish brown, wet, loose, fine grained
				-		
				16 --		
				-		
				17 --		
17.5	11	28.3	94.2	-		
				18 --		
				-		
				19 --		
				-		
20	10	34.4	SPT	20 --	CL	Sandy Clay, grayish brown, wet stiff, fine sand intermixed
				-		
				21 --		
				-		
				22 --		
22.5	30	25.6	99.9	-	SP	Sand, gray, wet, medium dense, fine grained
				23 --		
				-		
				24 --		
				-		
25	17	27.0	SPT	25 --	SM	Silty Sand, gray, wet, loose, fine grained, some clay
				-		

BORING LOG NUMBER 2

Century Housing

File No. 21855

dy

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26 --		
				27 --		
27.5	16	36.4	88.8	28 --	CL	Silty Clay, gray, wet, medium stiff, fine sand interbedded
				29 --		
30	7	25.4	SPT	30 --		
				31 --		
				32 --		
32.5	17	28.2	94.3	33 --		
				34 --		
35	10	35.2	SPT	35 --	CH	Silty Clay, gray, wet, stiff
				36 --		
				37 --		
37.5	39	30.6	95.6	38 --	SM	Silty Sand, dark gray, wet, medium dense, fine grained
				39 --		
40	13	31.7	SPT	40 --	CL	Silty Clay, gray, wet, stiff
				41 --		
				42 --		
42.5	27	29.6	93.8	43 --		
				44 --		
45	18	30.8	SPT	45 --		
				46 --		
				47 --		
47.5	42	28.4	96.2	48 --	SP	Sand, gray, wet, dense, fine grained
	50/5"			49 --		
50	53	19.2	SPT	50 --		very dense

BORING LOG NUMBER 2

Century Housing

File No. 21855

dy

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				51 --		
				52 --		
52.5	68	26.0	101.7	53 --		Sand, gray, wet, dense, fine grained
				54 --		
55	33	17.8	SPT	55 --		
				56 --		
				57 --		
57.5	46	23.1	102.0	58 --	SP/CL	Interbedded Sand to Silty Clay, gray, wet, dense, very stiff, fine to medium grained
				59 --		
60	32	30.9	SPT	60 --		
				61 --		
				62 --		
62.5	65	24.4	102.2	63 --	SM	Silty Sand, gray, wet, dense, fine grained
				64 --		
65	75	21.7	SPT	65 --	SP	Sand, gray, wet, dense, fine to medium grained
				66 --		
				67 --		
67.5	30 50/5"	18.1	106.6	68 --		
				69 --		
70	80	15.7	SPT	70 --		Total Depth 70 feet Water at 7½ feet Fill to 3 feet
				71 --		
				72 --		
				73 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				74 --		
				75 --		Used 5-inch diameter Rotary Wash Drill Rig 140 lb. Automatic Hammer, 30-inch drop Modified California Sampler unless otherwise noted
						SPT = Standard Penetration Test

BORING LOG NUMBER 3

Century Housing **Date: 08/22/19**

File No. 21855 **Method: 5-inch diameter Rotary Wash Drill Rig**

dy

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt
				0 --		4-inch Asphalt over 6-inch Base
				-		
				1 --		FILL: Silty Sand, yellowish brown, moist, loose, fine grained
				-		
				2 --		
2.5	13	17.9	104.0	-		
				3 --	SM	ALLUVIUM: Silty Sand, yellowish brown, loose, fine grained, interbedded clay pockets
				-		
				4 --		
				-		
5	3	18.7	SPT	5 --		
				-		
				6 --		
				-		
				7 --		
7.5	3	No Recovery		-		increased Sand, less Silt, some medium grains intermixed
				8 --		
				-		
				9 --		
				-		
10	5	5.4	SPT	10 --		
				-		some gravel
				11 --		
				-		
				12 --		
12.5	10	14.5	103.3	-		
				13 --		gray, wet
				-		
				14 --		
				-		
15	6	29.1	SPT	15 --		
				-	CL	Sandy Clay, gray, wet, soft, fine grained sand intermixed
				16 --		
				-		
				17 --		
17.5	10	36.3	89.7	-		
				18 --		
				-		
				19 --		
				-		
20	18	25.1	SPT	20 --		
				-	SP	Sand, gray , wet, medium dense, fine grained
				21 --		
				-		
				22 --		
22.5	2	45.7	76.5	-		
				23 --	CL	Silty Clay, gray, wet, soft
				-		
				24 --		
				-		
25	PUSH	42.2	SPT	25 --		
				-		

BORING LOG NUMBER 3

Century Housing

File No. 21855

dy

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26 --		
				27 --		
27.5	33	28.9	96.2	28 --	SP	Sand, gray to dark gray, wet, medium dense, fine grained
				29 --		
30	6	28.7	SPT	30 --	CL	Silty Clay, gray, wet, stiff, some fine grained sand intermixed
				31 --		
32.5	14	26.1	94.4	32 --		
				33 --		
				34 --		
35	11	39.7	SPT	35 --		
				36 --		
				37 --		
37.5	43	24.6	98.7	38 --	SM/SP	Silty Sand to Sand, gray, wet, dense, fine grained
				39 --		
40	32	25.8	SPT	40 --		
				41 --		
				42 --		
42.5	28	31.1	90.0	43 --	ML	Sandy Silt, gray, wet, stiff
				44 --		
45	21	28.8	SPT	45 --	CL	Sandy Clay, gray, wet, stiff
				46 --		
				47 --		
47.5	57	22.9	99.3	48 --	SP	Sand, gray, wet, dense, fine grained
				49 --		
50	48	20.1	SPT	50 --		

BORING LOG NUMBER 3

Century Housing

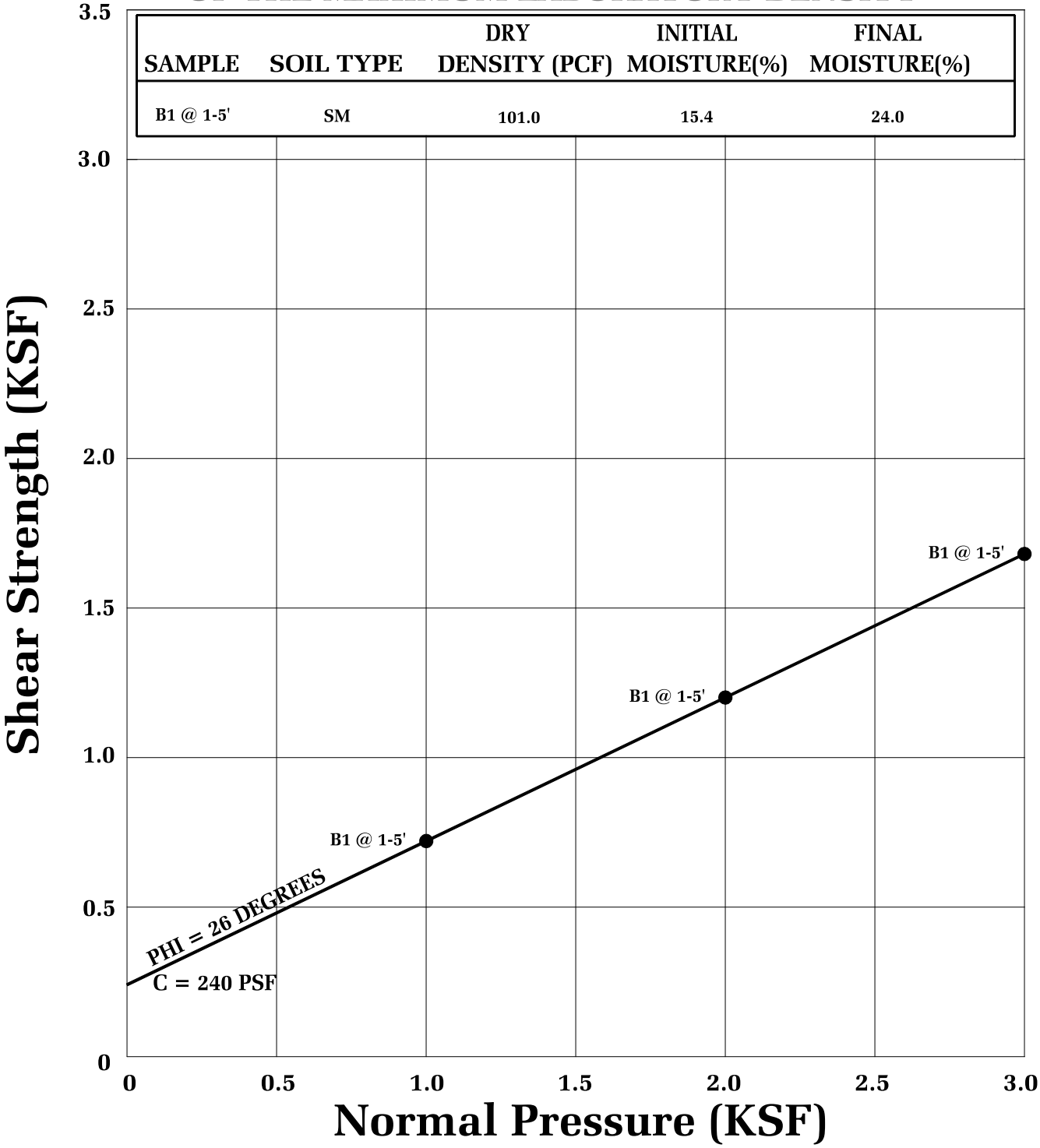
File No. 21855

dy

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				51 --		
				52 --		
52.5	82	21.4	104.3	53 --		increase in silt
				54 --		
55	27	18.4	SPT	55 --		
				56 --	ML	Sandy Silt, gray, wet, stiff
57.5	27	23.6	102.2	57 --	SP	Sand, gray, wet, very dense, fine grained
				58 --		
				59 --		
60	53	20.7	SPT	60 --		
				61 --		
				62 --		
62.5	64	24.1	101.6	63 --	SM/ML	Silty Sand to Sandy Silt, gray, very dense, very stiff, fine grained
				64 --		
65	76	15.9	SPT	65 --		
				66 --		
				67 --		
67.5	30 50/5"	21.2	101.6	68 --		
				69 --		
70	83	19.4	SPT	70 --		
				71 --		Total Depth 70 feet Water at 4½ feet Fill to 2.5 feet
				72 --		
				73 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				74 --		
				75 --		Used 5-inch diameter Rotary Wash Drill Rig 140 lb. Automatic Hammer, 30-inch drop Modified California Sampler unless otherwise noted
						SPT = Standard Penetration Test

BULK SAMPLE REMOLDED TO 90 PERCENT OF THE MAXIMUM LABORATORY DENSITY

SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
B1 @ 1-5'	SM	101.0	15.4	24.0



● Direct Shear, Saturated

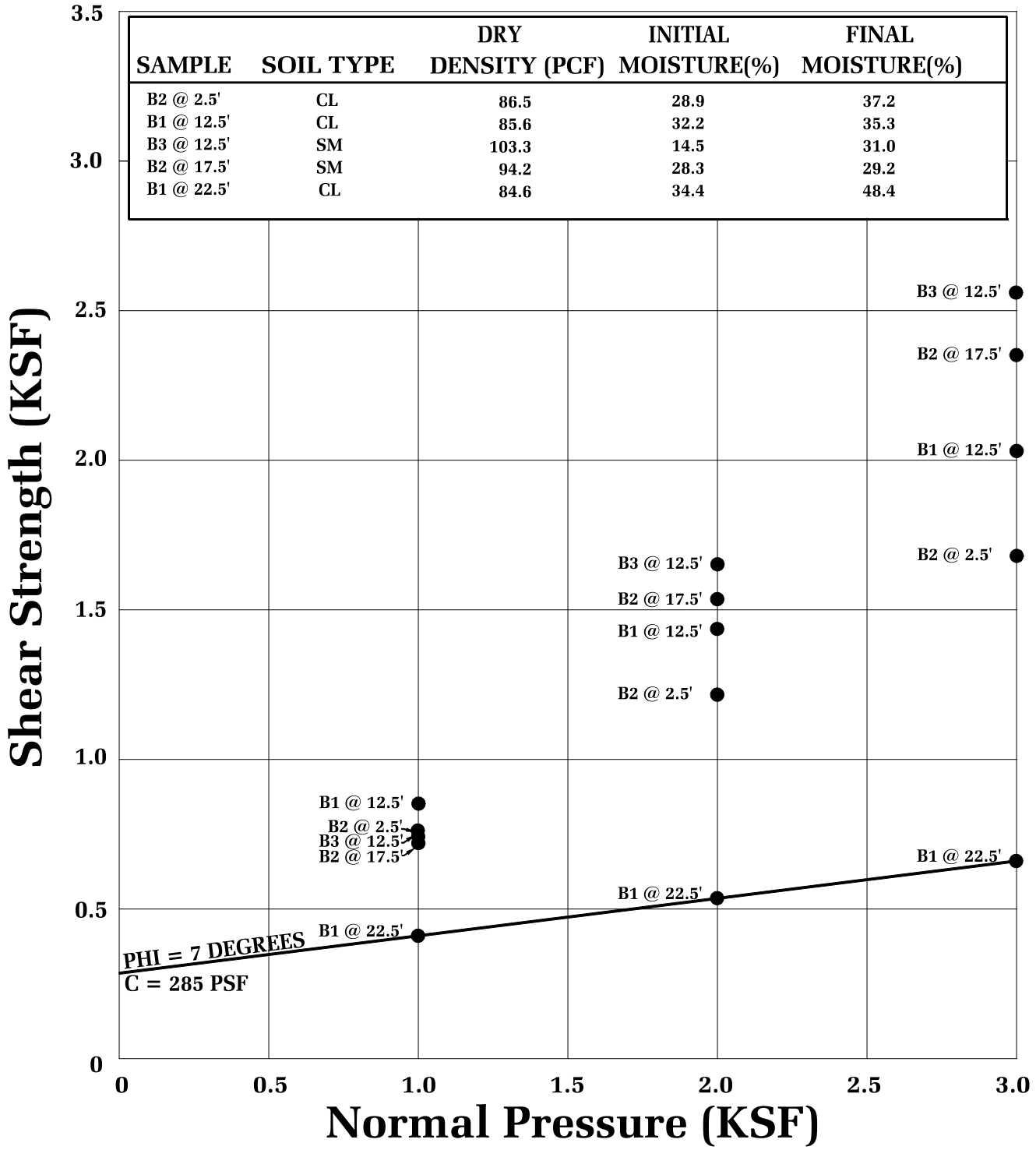
SHEAR TEST DIAGRAM

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Consulting Geotechnical Engineers

CENTURY HOUSING
2221 W. WILLIAMS ST., LONG BEACH

FILE NO. 21855 | **PLATE: B-1**

SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
B2 @ 2.5'	CL	86.5	28.9	37.2
B1 @ 12.5'	CL	85.6	32.2	35.3
B3 @ 12.5'	SM	103.3	14.5	31.0
B2 @ 17.5'	SM	94.2	28.3	29.2
B1 @ 22.5'	CL	84.6	34.4	48.4



● Direct Shear, Saturated

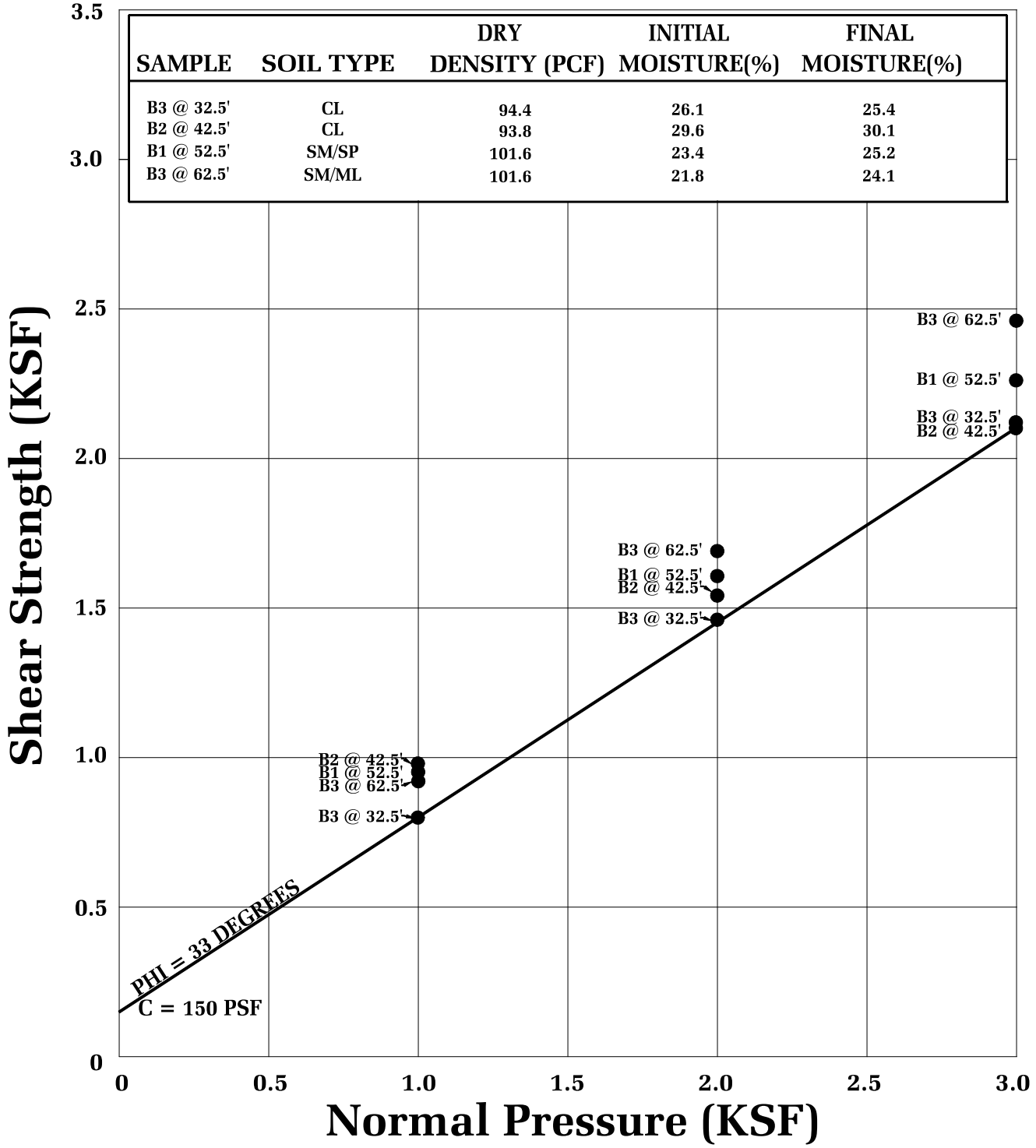
SHEAR TEST DIAGRAM

Geotechnologies, Inc.
Consulting Geotechnical Engineers

CENTURY HOUSING
2221 W. WILLIAMS ST., LONG BEACH

FILE NO. 21855 | PLATE: B-2

SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE(%)	FINAL MOISTURE(%)
B3 @ 32.5'	CL	94.4	26.1	25.4
B2 @ 42.5'	CL	93.8	29.6	30.1
B1 @ 52.5'	SM/SP	101.6	23.4	25.2
B3 @ 62.5'	SM/ML	101.6	21.8	24.1



● Direct Shear, Saturated

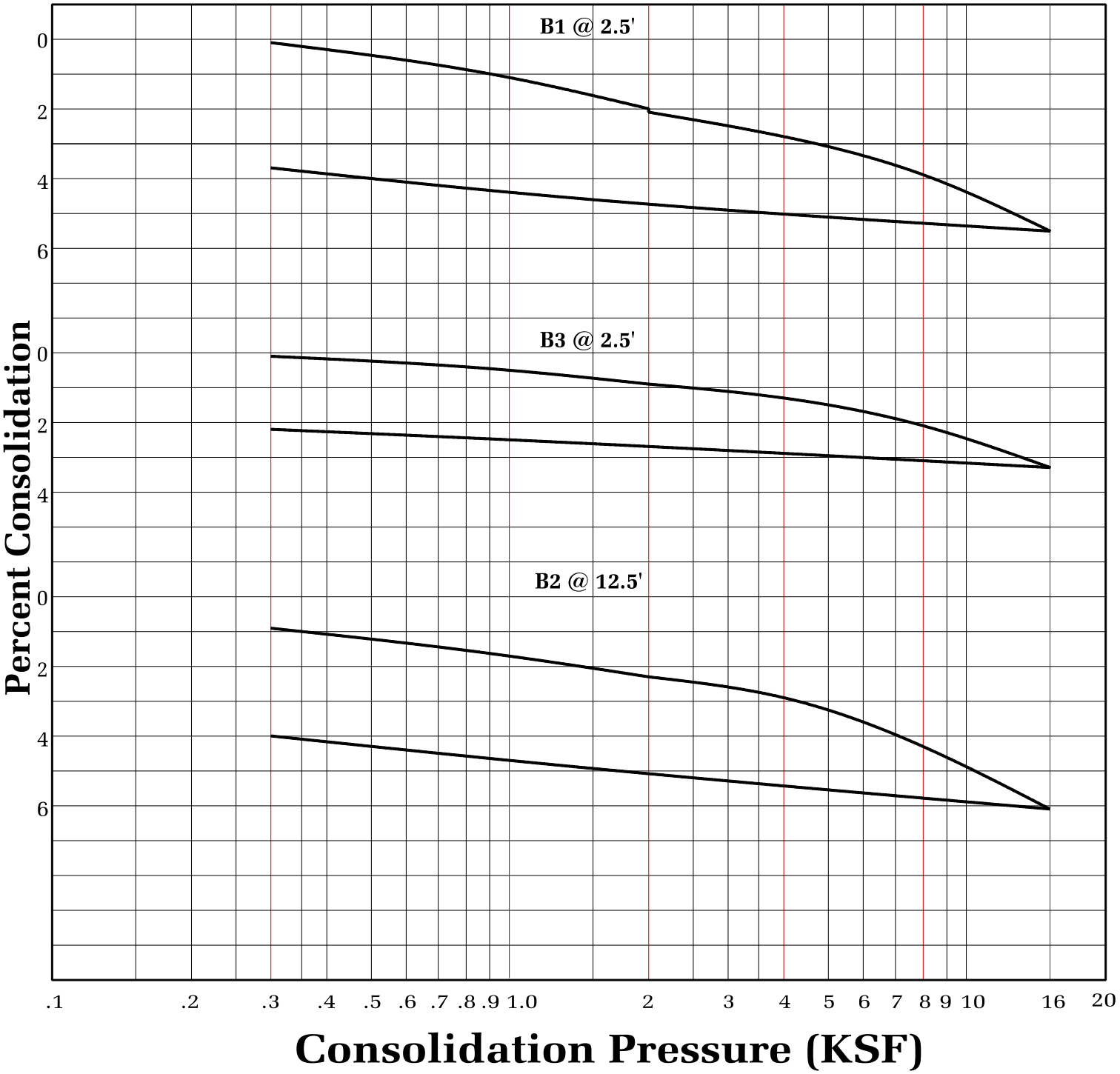
SHEAR TEST DIAGRAM

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Consulting Geotechnical Engineers

CENTURY HOUSING
2221 W. WILLIAMS ST., LONG BEACH

FILE NO. 21855 | PLATE: B-3

WATER ADDED AT 2 KSF



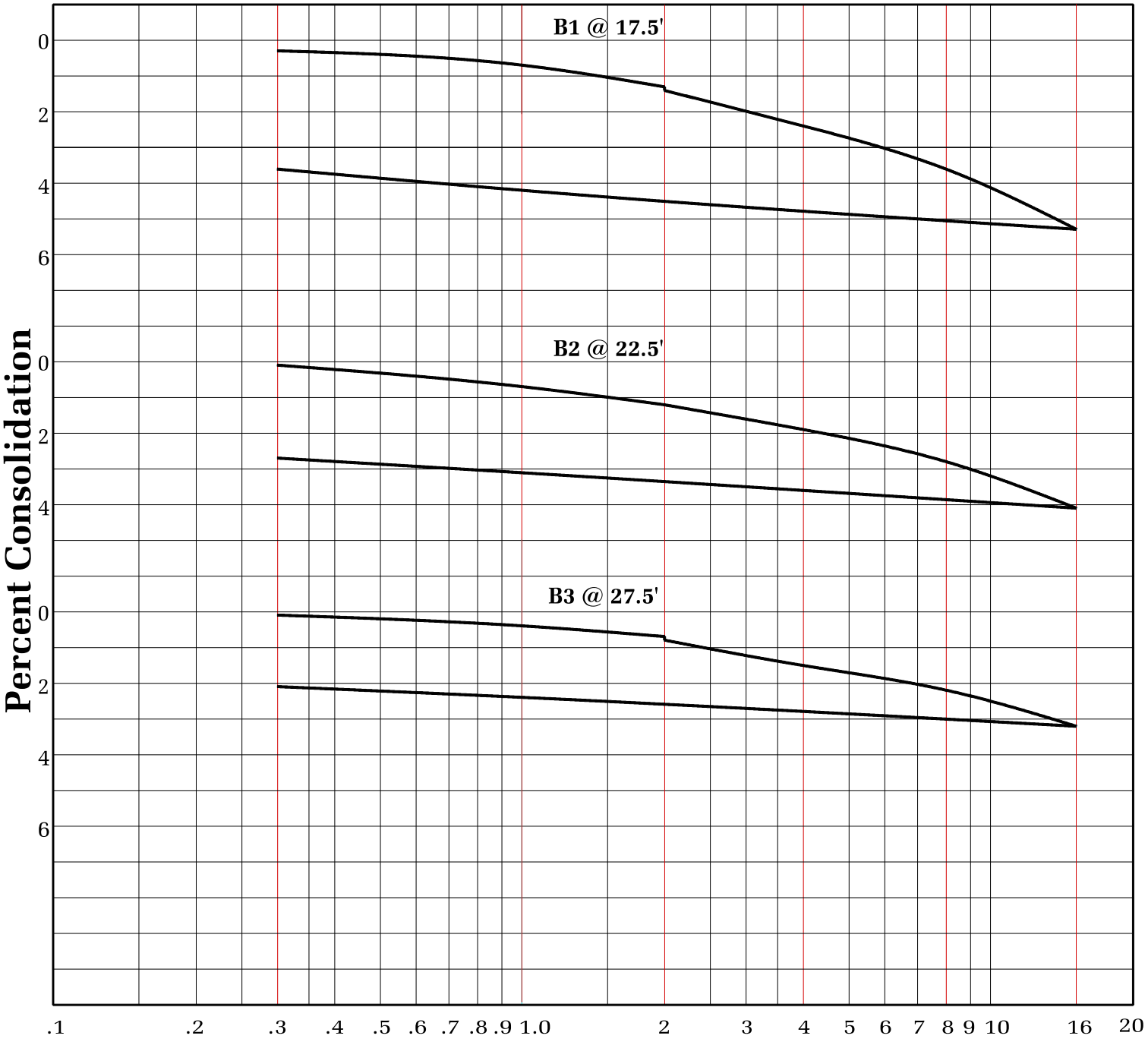
CONSOLIDATION TEST

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Consulting Geotechnical Engineers

CENTURY HOUSING
2221 W. WILLIAMS ST., LONG BEACH

FILE NO. 21855 | **PLATE: C-1**

WATER ADDED AT 2 KSF



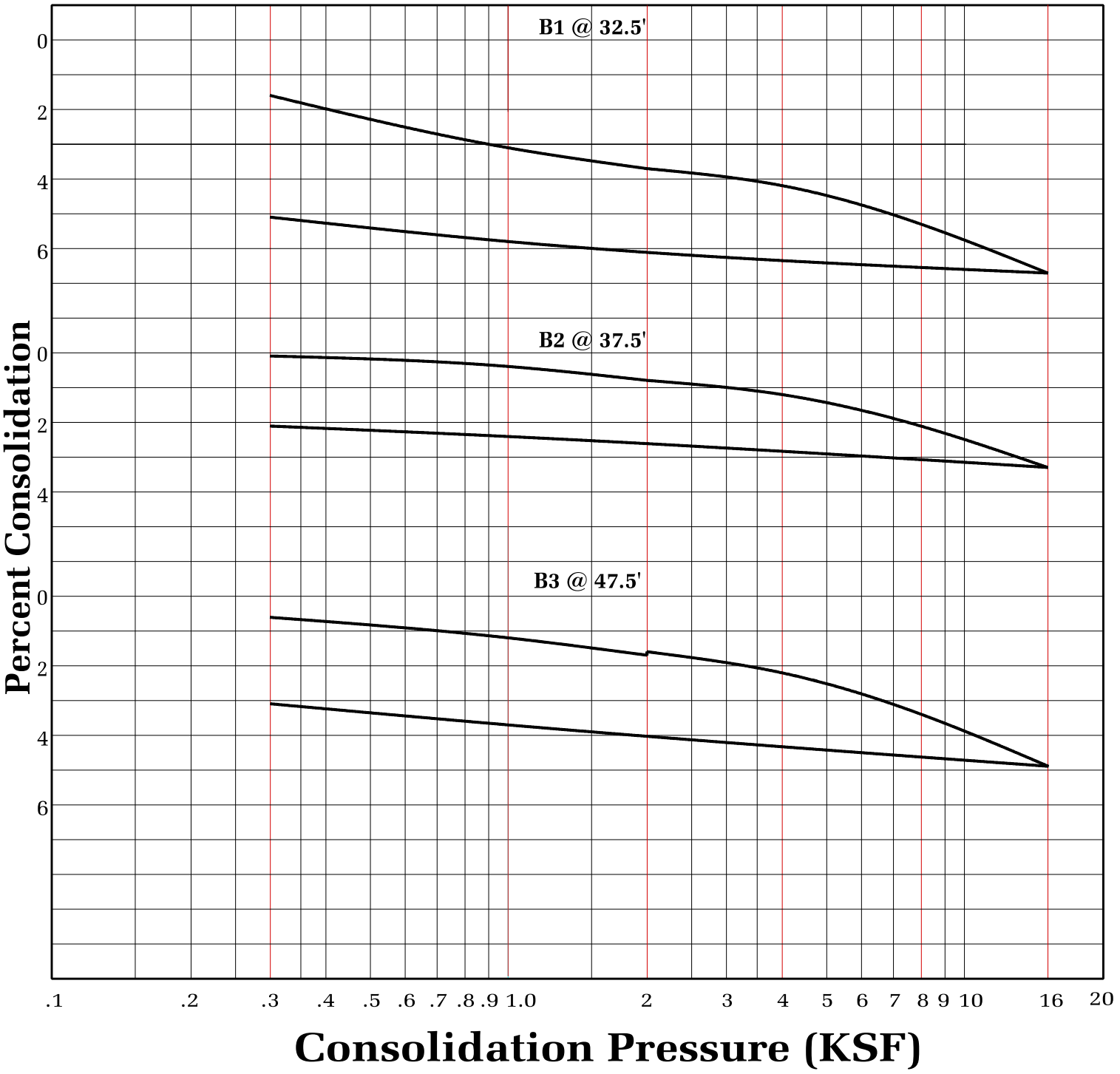
CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

CENTURY HOUSING
2221 W. WILLIAMS ST., LONG BEACH

FILE NO. 21855 | **PLATE: C-2**

WATER ADDED AT 2 KSF



CONSOLIDATION TEST

Geotechnologies, Inc.
Consulting Geotechnical Engineers

CENTURY HOUSING
2221 W. WILLIAMS ST., LONG BEACH

FILE NO. 21855 | **PLATE: C-3**

ASTM D-1557

SAMPLE	B1 @ 1-5'
SOIL TYPE:	SM
MAXIMUM DENSITY pcf.	112.2
OPTIMUM MOISTURE %	15.4

ASTM D 4829

SAMPLE	B1 @ 1-5'
SOIL TYPE:	SM
EXPANSION INDEX UBC STANDARD 18-2	10
EXPANSION CHARACTER	VERY LOW

SULFATE CONTENT

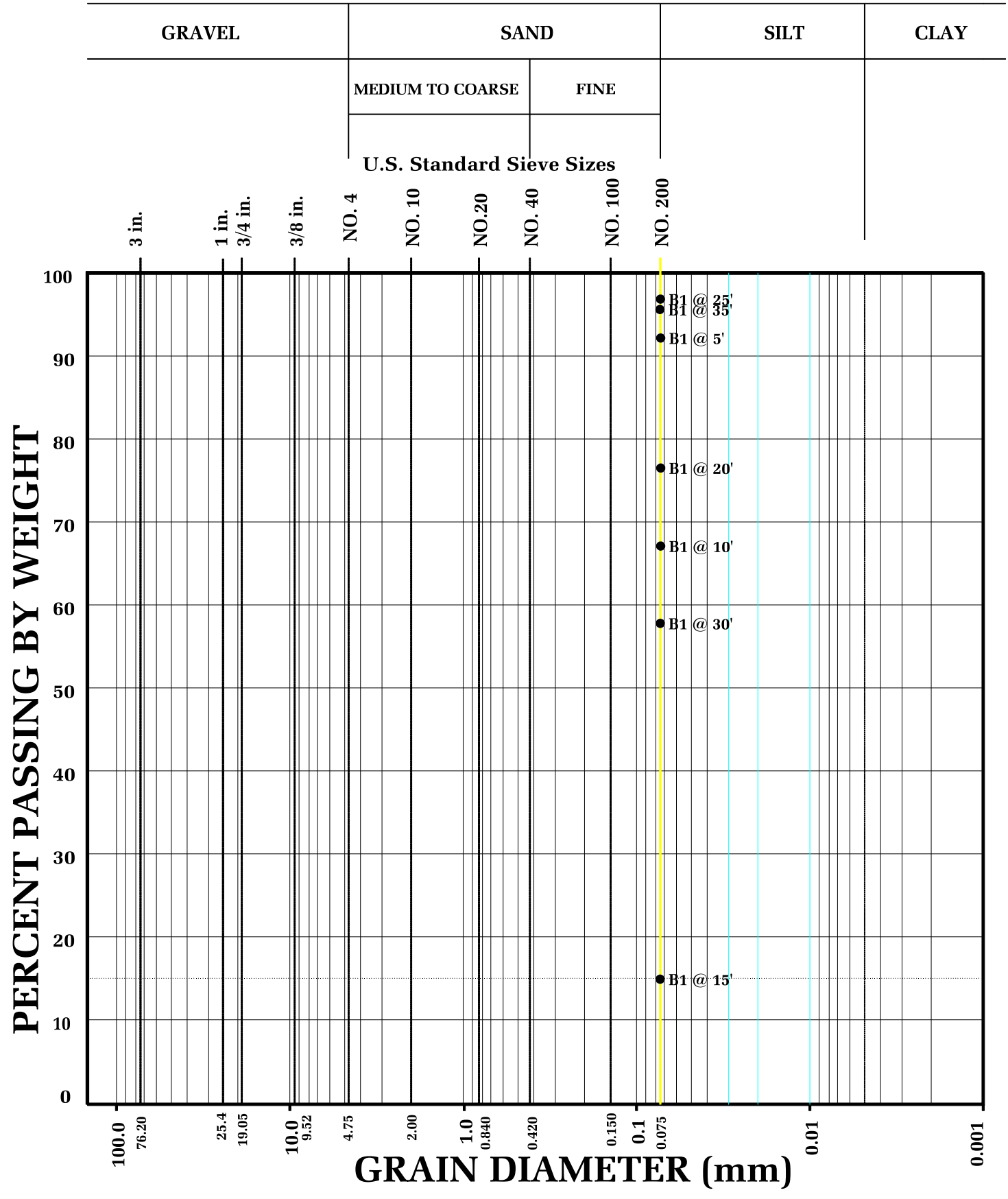
SAMPLE	B1 @ 1-5'
SULFATE CONTENT: (percentage by weight)	< 0.10 %

COMPACTION/EXPANSION/SULFATE DATA SHEET

Geotechnologies, Inc.
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CENTURY HOUSING
2221 W. WILLIAMS ST., LONG BEACH

FILE NO. 21855 | PLATE: D

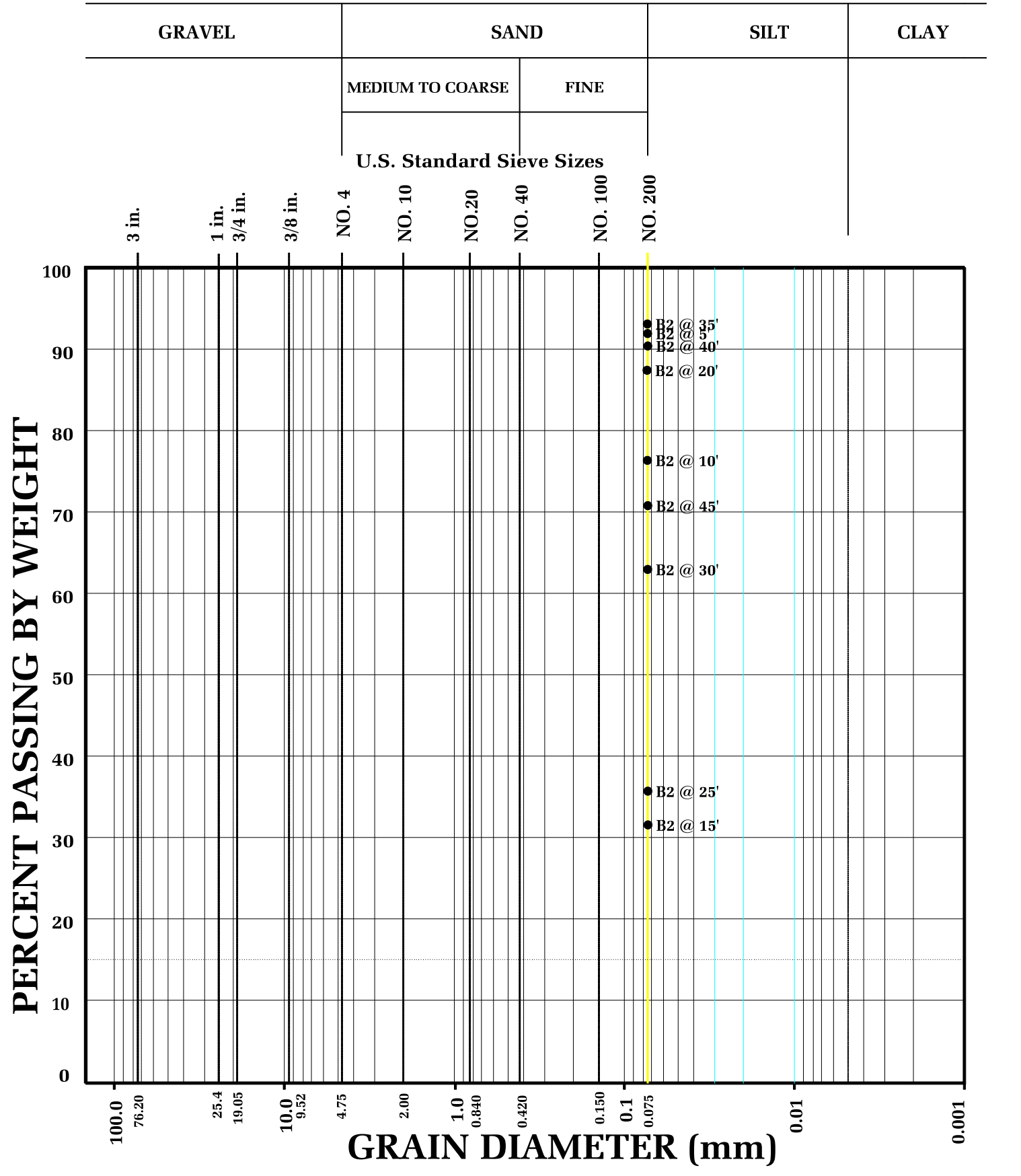


GRAIN SIZE DISTRIBUTION

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CENTURY HOUSING
2221 W. WILLIAMS ST., LONG BEACH

FILE NO. 21855 | **PLATE: E-1**

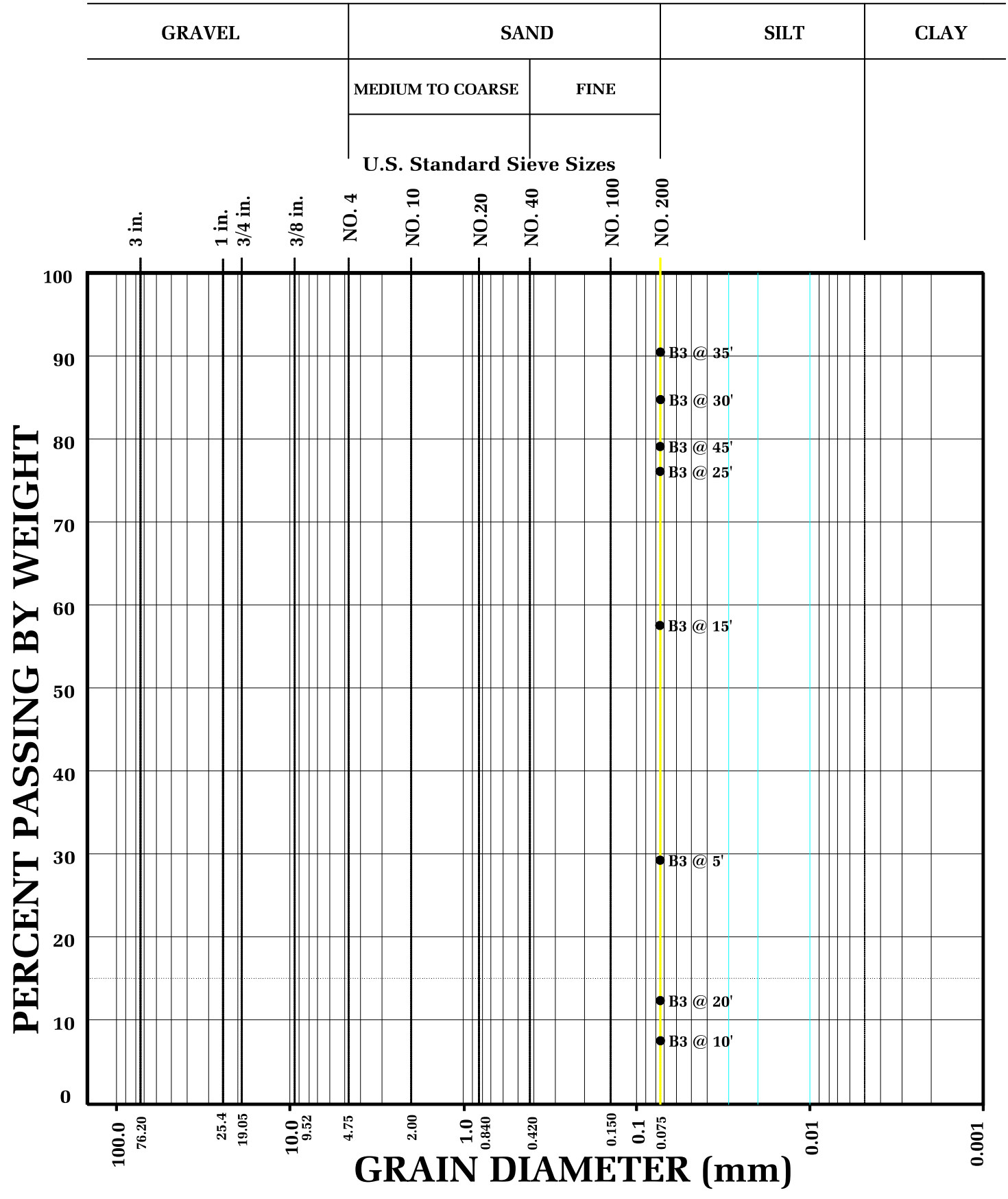


GRAIN SIZE DISTRIBUTION

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Consulting Geotechnical Engineers

CENTURY HOUSING
2221 W. WILLIAMS ST., LONG BEACH

FILE NO. 21855 | **PLATE: E-2**



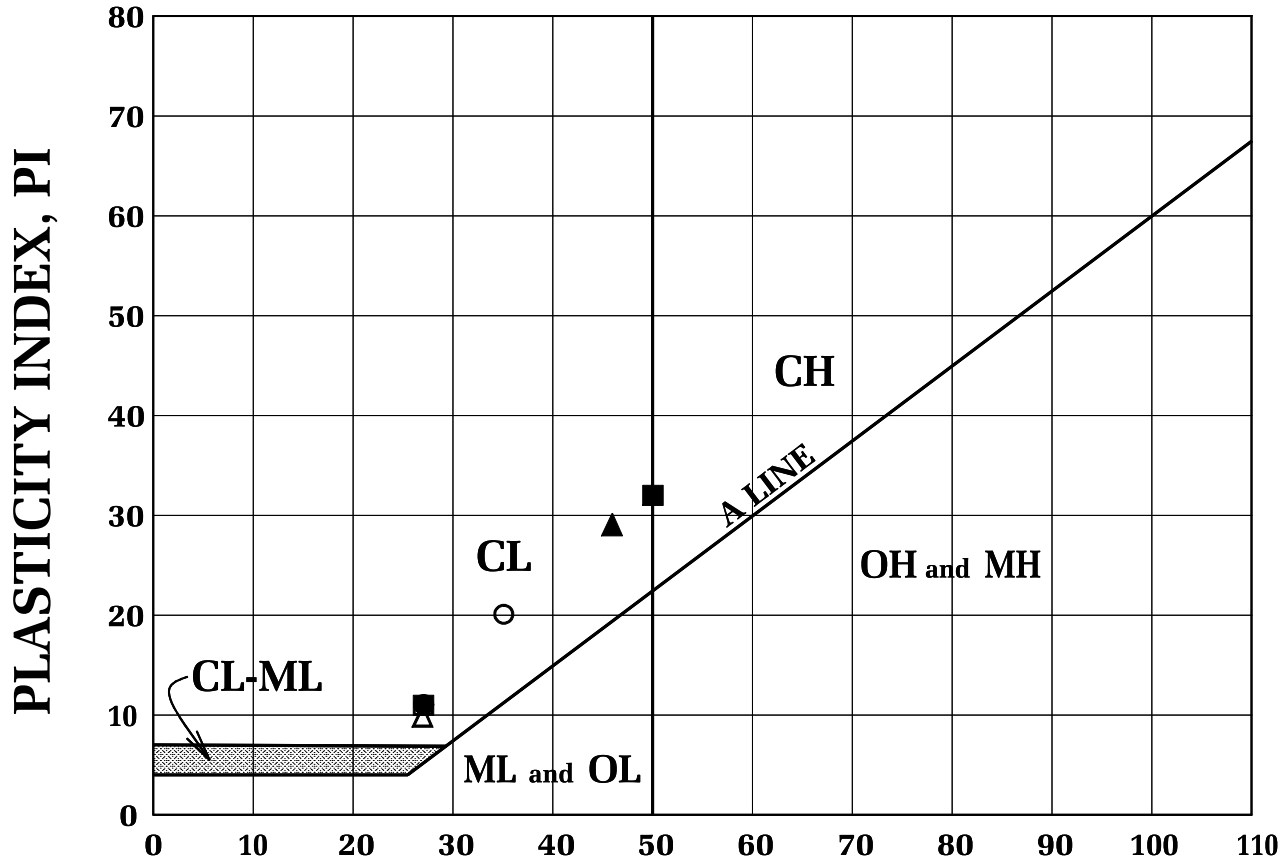
GRAIN SIZE DISTRIBUTION

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CENTURY HOUSING
2221 W. WILLIAMS ST., LONG BEACH

FILE NO. 21855 | **PLATE: E-3**

ASTM D4318



LIQUID LIMIT, LL

BORING NUMBER	DEPTH (FEET)	TEST SYMBOL	LL	PL	PI	DESCRIPTION
B1	5	○	35	15	20	CL
B1	10	●	27	16	11	CL
B1	20	△	27	17	10	CL
B1	25	▲	46	17	29	CL
B1	30	□	27	16	11	CL
B1	35	■	50	18	32	CH

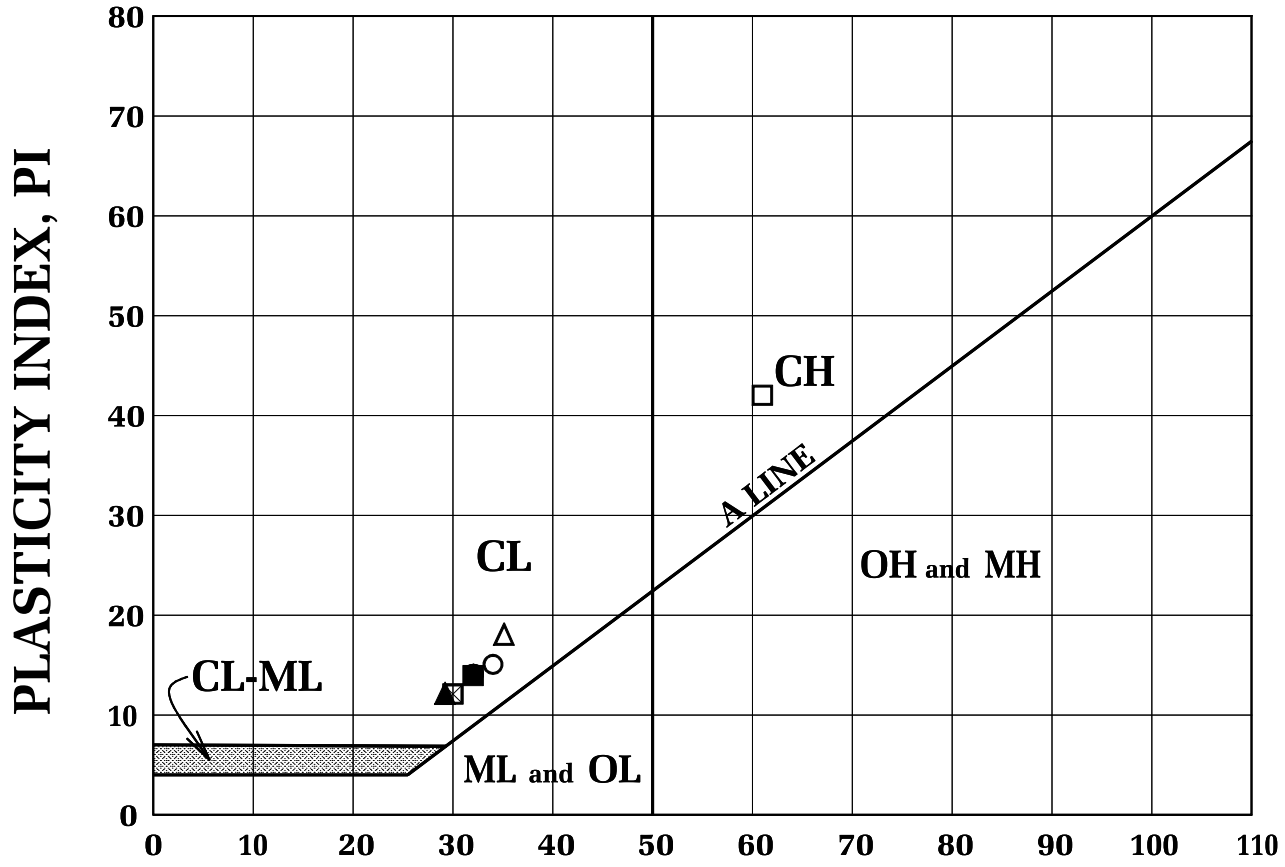
ATTERBERG LIMITS DETERMINATION

Geotechnologies, Inc.
Consulting Geotechnical Engineers

CENTURY HOUSING
2221 W. WILLIAMS ST., LONG BEACH

FILE NO. 21855 | PLATE: F1

ASTM D4318



BORING NUMBER	DEPTH (FEET)	TEST SYMBOL	LL	PL	PI	DESCRIPTION
B2	5	○	34	19	15	CL
B2	10	●	32	18	14	CL
B2	20	△	35	17	18	CL
B2	30	▲	29	17	12	CL
B2	35	□	61	19	42	CH
B2	40	■	32	18	14	CL
B2	45	⊠	30	18	12	CL

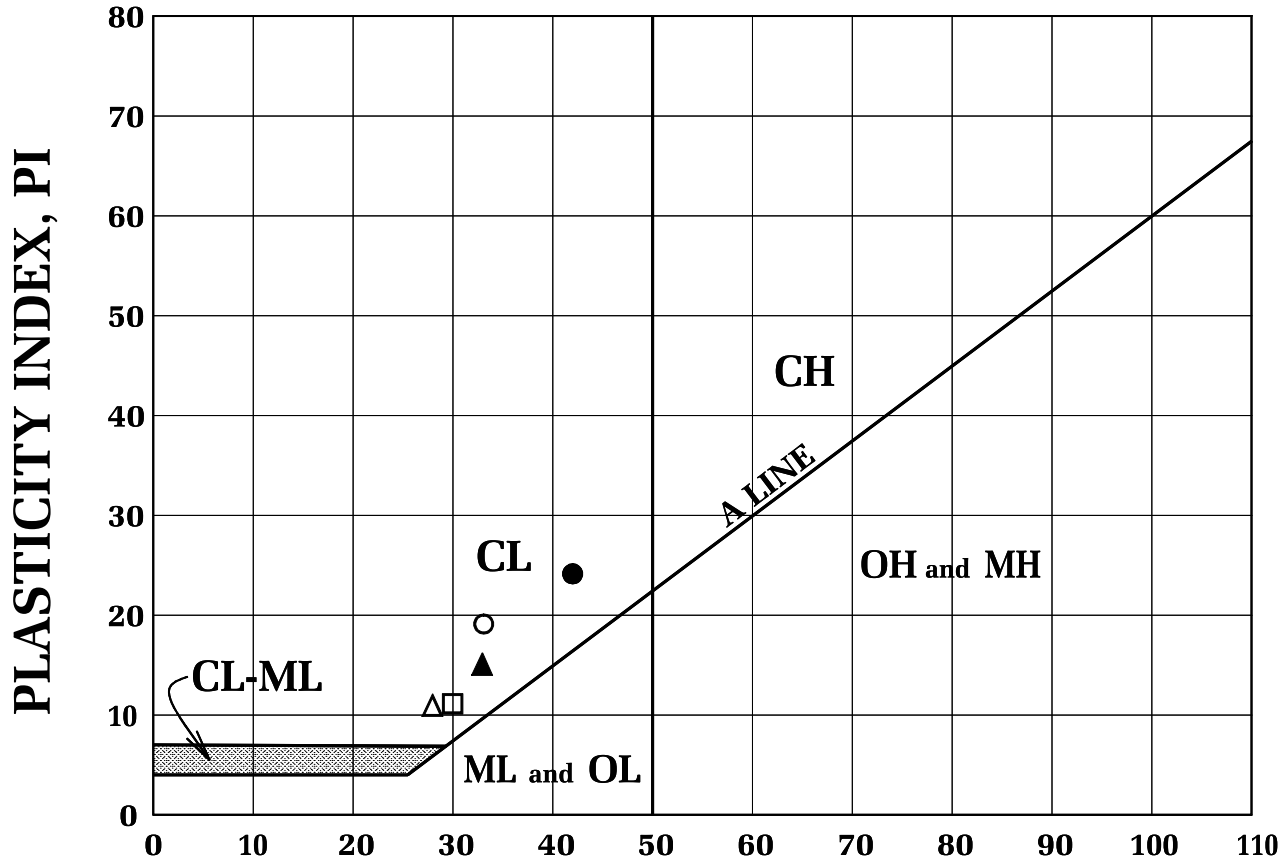
ATTERBERG LIMITS DETERMINATION

Geotechnologies, Inc.
Consulting Geotechnical Engineers

CENTURY HOUSING
2221 W. WILLIAMS ST., LONG BEACH

FILE NO. 21855 | PLATE: F2

ASTM D4318



LIQUID LIMIT, LL

BORING NUMBER	DEPTH (FEET)	TEST SYMBOL	LL	PL	PI	DESCRIPTION
B3	15	○	33	14	19	CL
B3	25	●	42	18	24	CL
B3	30	△	28	17	11	CL
B3	35	▲	33	18	15	CL
B3	45	□	30	19	11	CL

ATTERBERG LIMITS DETERMINATION

Geotechnologies, Inc.
Consulting Geotechnical Engineers

CENTURY HOUSING
2221 W. WILLIAMS ST., LONG BEACH

FILE NO. 21855 | PLATE: F3

Geotechnologies, Inc.

Project: Century Housing
File No.: 21855
Description: Liquefaction Analysis - Maximum Considered Earthquake (2% Probability of Exceedance in 50 years)
Boring Number: 1

LIQUEFACTION EVALUATION (Idriss & Boulanger, EERI NO 12)

EARTHQUAKE INFORMATION:

Earthquake Magnitude (M):	7.3
Peak Ground Horizontal Acceleration, PGA (g):	0.893
Calculated Mag.Wtg.Factor:	1.054

GROUNDWATER INFORMATION:

Current Groundwater Level (ft):	8.0
Historically Highest Groundwater Level* (ft):	8.0
Unit Weight of Water (pcf):	62.4

* Based on California Geological Survey Seismic Hazard Evaluation Report

BOREHOLE AND SAMPLER INFORMATION:

Borehole Diameter (inches):	8
SPT Sampler with room for Liner (Y/N):	Y

LIQUEFACTION BOUNDARY:

Plastic Index Cut Off (PI):	18
Minimum Liquefaction FS:	1

Depth to Base Layer (feet)	Total Unit Weight (pcf)	Current Water Level (feet)	Historical Water Level (feet)	Field SPT Blowcount N	Depth of SPT Blowcount (feet)	Fines Content #200 Sieve (%)	Plastic Index (PI)	Vetical Stress σ_{vo}, (psf)	Effective Vert. Stress σ_{vo}', (psf)	Fines Corrected $(N_1)_{60\,cs}$	Stress Reduction Coeff, r_d	Cyclic Shear Ratio CSR	Cyclic Resistance Ratio (CRR)	Factor of Safety CRR/CSR (F.S.)	Liquefaction Settlment ΔS_i (Inches)
1	121.5	Unsaturated	Unsaturated	2	5	92.2	20	121.5	121.5	9.5	1.00	0.583	0.133	Non-Liq.	0.00
2	121.5	Unsaturated	Unsaturated	2	5	92.2	20	243.0	243.0	9.5	1.00	0.581	0.133	Non-Liq.	0.00
3	121.5	Unsaturated	Unsaturated	2	5	92.2	20	364.5	364.5	9.5	1.00	0.580	0.133	Non-Liq.	0.00
4	121.5	Unsaturated	Unsaturated	2	5	92.2	20	486.0	486.0	9.5	1.00	0.578	0.133	Non-Liq.	0.00
5	121.5	Unsaturated	Unsaturated	2	5	92.2	20	607.5	607.5	9.8	0.99	0.577	0.135	Non-Liq.	0.00
6	121.5	Unsaturated	Unsaturated	2	5	92.2	20	729.0	729.0	9.8	0.99	0.575	0.135	Non-Liq.	0.00
7	121.5	Unsaturated	Unsaturated	2	5	92.2	20	850.5	850.5	9.8	0.99	0.573	0.133	Non-Liq.	0.00
8	121.5	Unsaturated	Unsaturated	2	5	92.2	20	972.0	972.0	9.7	0.98	0.572	0.131	Non-Liq.	0.00
9	121.5	Saturated	Saturated	2	5	92.2	20	1093.5	1031.1	9.8	0.98	0.604	0.131	Non-Liq.	0.00
10	116.2	Saturated	Saturated	3	10	67.1	11	1209.7	1084.9	11.7	0.98	0.633	0.146	0.2	0.41
11	116.2	Saturated	Saturated	3	10	67.1	11	1325.9	1138.7	11.5	0.97	0.659	0.144	0.2	0.41
12	116.2	Saturated	Saturated	3	10	67.1	11	1442.1	1192.5	11.4	0.97	0.682	0.142	0.2	0.42
13	116.2	Saturated	Saturated	3	10	67.1	11	1558.3	1246.3	11.2	0.97	0.702	0.140	0.2	0.42
14	116.2	Saturated	Saturated	3	10	67.1	11	1674.5	1300.1	11.1	0.96	0.721	0.138	0.2	0.42
15	118.2	Saturated	Saturated	15	15	14.9	0	1792.7	1355.9	33.8	0.96	0.737	1.018	1.4	0.00
16	118.2	Saturated	Saturated	15	15	14.9	0	1910.9	1411.7	33.2	0.96	0.752	0.911	1.2	0.00
17	118.2	Saturated	Saturated	15	15	14.9	0	2029.1	1467.5	32.7	0.95	0.765	0.819	1.1	0.00
18	118.2	Saturated	Saturated	8	20	76.6	10	2147.3	1523.3	20.2	0.95	0.776	0.229	0.3	0.27
19	118.2	Saturated	Saturated	8	20	76.6	10	2265.5	1579.1	19.9	0.94	0.787	0.224	0.3	0.28
20	116.9	Saturated	Saturated	8	20	76.6	10	2382.4	1633.6	19.6	0.94	0.796	0.219	0.3	0.28
21	116.9	Saturated	Saturated	8	20	76.6	10	2499.3	1688.1	19.3	0.94	0.805	0.215	0.3	0.28
22	116.9	Saturated	Saturated	8	20	76.6	10	2616.2	1742.6	19.1	0.93	0.812	0.211	0.3	0.29
23	111.8	Saturated	Saturated	2	25	96.9	29	2728.0	1792.0	8.8	0.93	0.820	0.118	Non-Liq.	0.00
24	111.8	Saturated	Saturated	2	25	96.9	29	2839.8	1841.4	8.8	0.92	0.827	0.117	Non-Liq.	0.00
25	111.8	Saturated	Saturated	2	25	96.9	29	2951.6	1890.8	8.7	0.92	0.833	0.116	Non-Liq.	0.00
26	111.8	Saturated	Saturated	2	25	96.9	29	3063.4	1940.2	8.7	0.91	0.838	0.116	Non-Liq.	0.00
27	111.8	Saturated	Saturated	2	25	96.9	29	3175.2	1989.6	8.6	0.91	0.843	0.115	Non-Liq.	0.00
28	111.8	Saturated	Saturated	2	25	96.9	29	3287.0	2039.0	8.7	0.91	0.847	0.116	Non-Liq.	0.00
29	111.8	Saturated	Saturated	2	25	96.9	29	3398.8	2088.4	8.7	0.90	0.851	0.115	Non-Liq.	0.00
30	118.5	Saturated	Saturated	6	30	57.8	11	3517.3	2144.5	15.0	0.90	0.853	0.164	0.2	0.34
31	118.5	Saturated	Saturated	6	30	57.8	11	3635.8	2200.6	14.9	0.89	0.855	0.163	0.2	0.35
32	118.5	Saturated	Saturated	6	30	57.8	11	3754.3	2256.7	14.8	0.89	0.856	0.161	0.2	0.35
33	118.5	Saturated	Saturated	6	30	57.8	11	3872.8	2312.8	14.6	0.88	0.857	0.160	0.2	0.35
34	118.5	Saturated	Saturated	6	30	57.8	11	3991.3	2368.9	14.5	0.88	0.857	0.158	0.2	0.35
35	118.9	Saturated	Saturated	7	35	95.6	32	4110.2	2425.4	15.8	0.87	0.858	0.169	Non-Liq.	0.00
36	118.9	Saturated	Saturated	7	35	95.6	32	4229.1	2481.9	15.7	0.87	0.858	0.167	Non-Liq.	0.00
37	118.9	Saturated	Saturated	7	35	95.6	32	4348.0	2538.4	15.5	0.86	0.857	0.166	Non-Liq.	0.00
38	118.9	Saturated	Saturated	7	35	95.6	32	4466.9	2594.9	15.4	0.86	0.857	0.164	Non-Liq.	0.00
39	118.9	Saturated	Saturated	7	35	95.6	32	4585.8	2651.4	15.3	0.85	0.856	0.163	Non-Liq.	0.00
40	119.5	Saturated	Saturated	22	40	0.0	0	4705.3	2708.5	37.7	0.85	0.854	1.953	2.3	0.00
41	119.5	Saturated	Saturated	22	40	0.0	0	4824.8	2765.6	37.4	0.84	0.853	1.893	2.2	0.00
42	119.5	Saturated	Saturated	22	40	0.0	0	4944.3	2822.7	37.1	0.84	0.851	1.742	2.0	0.00
43	119.5	Saturated	Saturated	22	40	0.0	0	5063.8	2879.8	36.8	0.83	0.850	1.610	1.9	0.00
44	119.5	Saturated	Saturated	22	40	0.0	0	5183.3	2936.9	36.6	0.83	0.848	1.496	1.8	0.00
45	125.8	Saturated	Saturated	19	45	0.0	0	5309.1	3000.3	29.9	0.82	0.845	0.469	0.6	0.11
46	125.8	Saturated	Saturated	19	45	0.0	0	5434.9	3063.7	29.6	0.82	0.842	0.452	0.5	0.12
47	125.8	Saturated	Saturated	19	45	0.0	0	5560.7	3127.1	29.4	0.81	0.839	0.436	0.5	0.12
48	124.3	Saturated	Saturated	33	50	0.0	0	5685.0	3189.0	55.3	0.81	0.836	1.852	2.2	0.00
49	124.3	Saturated	Saturated	33	50	0.0	0	5809.3	3250.9	55.0	0.80	0.833	1.840	2.2	0.00
50	124.3	Saturated	Saturated	33	50	0.0	0	5933.6	3312.8	54.8	0.80	0.829	1.828	2.2	0.00
51	124.3	Saturated	Saturated	33	50	0.0	0	6057.9	3374.7	54.5	0.79	0.826	1.816	2.2	0.00
52	124.3	Saturated	Saturated	33	50	0.0	0	6182.2	3436.6	54.2	0.79	0.823	1.805	2.2	0.00
53	124.3	Saturated	Saturated	33	50	0.0	0	6306.5	3498.5	54.0	0.78	0.819	1.794	2.2	0.00
54	124.3	Saturated	Saturated	33	50	0.0	0	6430.8	3560.4	53.7	0.78	0.816	1.783	2.2	0.00
55	126.6	Saturated	Saturated	31	55	0.0	0	6557.4	3624.6	50.2	0.77	0.812	1.772	2.2	0.00
56	126.6	Saturated	Saturated	31	55	0.0	0	6684.0	3688.8	50.0	0.77	0.808	1.761	2.2	0.00
57	126.6	Saturated	Saturated	31	55	0.0	0	6810.6	3753.0	49.8	0.76	0.805	1.750	2.2	0.00
58	124.2	Saturated	Saturated	36	60	0.0	0	6934.8	3814.8	57.6	0.76	0.801	1.740	2.2	0.00
59	124.2	Saturated	Saturated	36	60	0.0	0	7059.0	3876.6	57.3	0.75	0.797	1.730	2.2	0.00
60	124.2	Saturated	Saturated	36	60	0.0	0	7183.2	3938.4	57.1	0.75	0.794	1.720	2.2	0.00
61	124.2	Saturated	Saturated	36	60	0.0	0	7307.4	4000.2	56.8	0.75	0.790	1.711	2.2	0.00
62	124.2	Saturated	Saturated	36	60	0.0	0	7431.6	4062.0	56.6	0.74	0.786	1.701	2.2	0.00
63	124.2	Saturated	Saturated	36	60	0.0	0	7555.8	4123.8	56.4	0.74	0.783	1.692	2.2	0.00
64	124.2	Saturated	Saturated	36	60	0.0	0	7680.0	4185.6	56.2	0.73	0.779	1.682	2.2	0.00
65	124.3	Saturated	Saturated	52	65	0.0	0	7804.3	4247.5	80.8	0.73	0.775	1.673	2.2	0.00
66	124.3	Saturated	Saturated	52	65	0.0	0	7928.6	4309.4	80.5	0.72	0.772	1.664	2.2	0.00
67	124.3	Saturated	Saturated	52	65	0.0	0	8052.9	4371.3	80.2	0.72	0.768	1.655	2.2	0.00
68	128.0	Saturated	Saturated	57	70	0.0	0	8180.9	4436.9	87.6	0.71	0.764	1.646	2.2	0.00
69	128.0	Saturated	Saturated	57	70	0.0	0	8308.9	4502.5	87.3	0.71	0.760	1.637	2.2	0.00
70	128.0	Saturated	Saturated	57	70	0.0	0	8436.9	4568.1	86.9	0.71	0.756	1.628	2.2	0.00
														Total Liquefaction Settlement, S =	5.58 inches

Geotechnologies, Inc.

Project: Century Housing
File No.: 21855
Description: Liquefaction Analysis - Maximum Considered Earthquake (2% Probability of Exceedance in 50 years)
Boring Number: 2

LIQUEFACTION EVALUATION (Idriss & Boulanger, EERI NO 12)

EARTHQUAKE INFORMATION:

Earthquake Magnitude (M):	7.3
Peak Ground Horizontal Acceleration, PGA (g):	0.893
Calculated Mag.Wtg.Factor:	1.054

GROUNDWATER INFORMATION:

Current Groundwater Level (ft):	7.5
Historically Highest Groundwater Level* (ft):	7.5
Unit Weight of Water (pcf):	62.4

* Based on California Geological Survey Seismic Hazard Evaluation Report

BOREHOLE AND SAMPLER INFORMATION:

Borehole Diameter (inches):	8
SPT Sampler with room for Liner (Y/N):	Y

LIQUEFACTION BOUNDARY:

Plastic Index Cut Off (PI):	18
Minimum Liquefaction FS:	1

Depth to Base Layer (feet)	Total Unit Weight (pcf)	Current Water Level (feet)	Historical Water Level (feet)	Field SPT Blowcount N	Depth of SPT Blowcount (feet)	Fines Content #200 Sieve (%)	Plastic Index (PI)	Vetical Stress σ_{vo} (psf)	Effective Vert. Stress σ_{vo}' (psf)	Fines Corrected $(N_1)_{60\text{-}cs}$	Stress Reduction Coeff, r_d	Cyclic Shear Ratio CSR	Cyclic Resistance Ratio (CRR)	Factor of Safety CRR/CSR (F.S.)	Liquefaction Settlment ΔS_i (inches)
1	111.5	Unsaturated	Unsaturated	1	5	91.9	15	111.5	111.5	7.5	1.00	0.583	0.118	Non-Liq.	0.00
2	111.5	Unsaturated	Unsaturated	1	5	91.9	15	223.0	223.0	7.5	1.00	0.581	0.118	Non-Liq.	0.00
3	111.5	Unsaturated	Unsaturated	1	5	91.9	15	334.5	334.5	7.5	1.00	0.580	0.118	Non-Liq.	0.00
4	111.5	Unsaturated	Unsaturated	1	5	91.9	15	446.0	446.0	7.5	1.00	0.578	0.118	Non-Liq.	0.00
5	111.5	Unsaturated	Unsaturated	1	5	91.9	15	557.5	557.5	7.7	0.99	0.577	0.119	Non-Liq.	0.00
6	111.5	Unsaturated	Unsaturated	1	5	91.9	15	669.0	669.0	7.7	0.99	0.575	0.118	Non-Liq.	0.00
7	111.5	Unsaturated	Unsaturated	1	5	91.9	15	780.5	780.5	7.7	0.99	0.573	0.117	Non-Liq.	0.00
8	111.5	Saturated	Saturated	1	5	91.9	15	892.0	860.8	7.7	0.98	0.592	0.116	0.2	0.52
9	111.5	Saturated	Saturated	1	5	91.9	15	1003.5	909.9	7.8	0.98	0.628	0.117	0.2	0.51
10	118.3	Saturated	Saturated	4	10	76.4	14	1121.8	965.8	14.2	0.98	0.660	0.171	0.3	0.36
11	118.3	Saturated	Saturated	4	10	76.4	14	1240.1	1021.7	13.9	0.97	0.687	0.167	0.2	0.36
12	118.3	Saturated	Saturated	4	10	76.4	14	1358.4	1077.6	13.6	0.97	0.711	0.164	0.2	0.37
13	118.3	Saturated	Saturated	4	10	76.4	14	1476.7	1133.5	13.4	0.97	0.732	0.161	0.2	0.37
14	118.3	Saturated	Saturated	4	10	76.4	14	1595.0	1189.4	13.2	0.96	0.751	0.158	0.2	0.38
15	119.6	Saturated	Saturated	8	15	31.6	0	1714.6	1246.6	21.8	0.96	0.767	0.261	0.3	0.26
16	119.6	Saturated	Saturated	8	15	31.6	0	1834.2	1303.8	21.4	0.96	0.781	0.253	0.3	0.26
17	119.6	Saturated	Saturated	8	15	31.6	0	1953.8	1361.0	21.0	0.95	0.794	0.245	0.3	0.27
18	119.6	Saturated	Saturated	8	15	31.6	0	2073.4	1418.2	20.6	0.95	0.805	0.238	0.3	0.27
19	119.6	Saturated	Saturated	8	15	31.6	0	2193.0	1475.4	20.3	0.94	0.815	0.232	0.3	0.27
20	123.2	Saturated	Saturated	10	20	87.4	19	2316.2	1536.2	24.0	0.94	0.823	0.296	Non-Liq.	0.00
21	123.2	Saturated	Saturated	10	20	87.4	19	2439.4	1597.0	23.6	0.94	0.830	0.286	Non-Liq.	0.00
22	123.2	Saturated	Saturated	10	20	87.4	19	2562.6	1657.8	23.2	0.93	0.836	0.277	Non-Liq.	0.00
23	123.2	Saturated	Saturated	17	25	35.7	0	2685.8	1718.6	37.0	0.93	0.842	1.936	2.3	0.00
24	123.2	Saturated	Saturated	17	25	35.7	0	2809.0	1779.4	36.5	0.92	0.846	1.695	2.0	0.00
25	123.3	Saturated	Saturated	17	25	35.7	0	2932.3	1840.3	36.0	0.92	0.850	1.502	1.8	0.00
26	123.3	Saturated	Saturated	17	25	35.7	0	3055.6	1901.2	35.5	0.91	0.853	1.345	1.6	0.00
27	123.3	Saturated	Saturated	17	25	35.7	0	3178.9	1962.1	35.1	0.91	0.856	1.215	1.4	0.00
28	123.3	Saturated	Saturated	7	30	63.0	21	3302.2	2023.0	16.9	0.91	0.858	0.184	Non-Liq.	0.00
29	123.3	Saturated	Saturated	7	30	63.0	21	3425.5	2083.9	16.7	0.90	0.859	0.181	Non-Liq.	0.00
30	121.0	Saturated	Saturated	7	30	63.0	21	3546.5	2142.5	16.6	0.90	0.861	0.179	Non-Liq.	0.00
31	121.0	Saturated	Saturated	7	30	63.0	21	3667.5	2201.1	16.4	0.89	0.862	0.177	Non-Liq.	0.00
32	121.0	Saturated	Saturated	7	30	63.0	21	3788.5	2259.7	16.3	0.89	0.863	0.175	Non-Liq.	0.00
33	121.0	Saturated	Saturated	7	30	63.0	21	3909.5	2318.3	16.1	0.88	0.863	0.173	Non-Liq.	0.00
34	121.0	Saturated	Saturated	7	30	63.0	21	4030.5	2376.9	16.0	0.88	0.863	0.171	Non-Liq.	0.00
35	122.8	Saturated	Saturated	10	35	93.1	42	4153.3	2437.3	20.7	0.87	0.862	0.222	Non-Liq.	0.00
36	122.8	Saturated	Saturated	10	35	93.1	42	4276.1	2497.7	20.5	0.87	0.862	0.219	Non-Liq.	0.00
37	122.8	Saturated	Saturated	10	35	93.1	42	4398.9	2558.1	20.3	0.86	0.861	0.215	Non-Liq.	0.00
38	123.2	Saturated	Saturated	13	40	90.4	14	4522.1	2618.9	25.4	0.86	0.859	0.305	0.4	0.22
39	123.2	Saturated	Saturated	13	40	90.4	14	4645.3	2679.7	25.2	0.85	0.858	0.298	0.3	0.23
40	123.2	Saturated	Saturated	13	40	90.4	14	4768.5	2740.5	25.0	0.85	0.856	0.292	0.3	0.23
41	123.2	Saturated	Saturated	13	40	90.4	14	4891.7	2801.3	24.8	0.84	0.854	0.286	0.3	0.23
42	123.2	Saturated	Saturated	13	40	90.4	14	5014.9	2862.1	24.5	0.84	0.852	0.281	0.3	0.23
43	123.2	Saturated	Saturated	13	40	90.4	14	5138.1	2922.9	24.3	0.83	0.849	0.275	0.3	0.23
44	123.2	Saturated	Saturated	13	40	90.4	14	5261.3	2983.7	24.2	0.83	0.847	0.271	0.3	0.23
45	122.5	Saturated	Saturated	18	45	70.8	12	5383.8	3043.8	33.3	0.82	0.844	0.767	0.9	0.06
46	122.5	Saturated	Saturated	18	45	70.8	12	5506.3	3103.9	33.0	0.82	0.842	0.733	0.9	0.06
47	122.5	Saturated	Saturated	18	45	70.8	12	5628.8	3164.0	32.8	0.81	0.839	0.702	0.8	0.07
48	122.5	Saturated	Saturated	18	45	70.8	12	5751.3	3224.1	32.6	0.81	0.836	0.673	0.8	0.07
49	122.5	Saturated	Saturated	18	45	70.8	12	5873.8	3284.2	32.4	0.80	0.833	0.647	0.8	0.07
50	125.8	Saturated	Saturated	53	50	0.0	0	5999.6	3347.6	87.7	0.80	0.830	1.821	2.2	0.00
51	125.8	Saturated	Saturated	53	50	0.0	0	6125.4	3411.0	87.3	0.79	0.826	1.810	2.2	0.00
52	125.8	Saturated	Saturated	53	50	0.0	0	6251.2	3474.4	86.9	0.79	0.823	1.798	2.2	0.00
53	125.8	Saturated	Saturated	53	50	0.0	0	6377.0	3537.8	86.4	0.78	0.819	1.787	2.2	0.00
54	125.8	Saturated	Saturated	53	50	0.0	0	6502.8	3601.2	86.0	0.78	0.816	1.776	2.2	0.00
55	126.9	Saturated	Saturated	33	55	0.0	0	6629.7	3665.7	53.3	0.77	0.812	1.765	2.2	0.00
56	126.9	Saturated	Saturated	33	55	0.0	0	6756.6	3730.2	53.1	0.77	0.808	1.754	2.2	0.00
57	126.9	Saturated	Saturated	33	55	0.0	0	6883.5	3794.7	52.8	0.76	0.804	1.743	2.2	0.00
58	126.4	Saturated	Saturated	32	60	0.0	0	7009.9	3858.7	51.0	0.76	0.800	1.733	2.2	0.00
59	126.4	Saturated	Saturated	32	60	0.0	0	7136.3	3922.7	50.8	0.75	0.797	1.723	2.2	0.00
60	126.4	Saturated	Saturated	32	60	0.0	0	7262.7	3986.7	50.6	0.75	0.793	1.713	2.2	0.00
61	126.4	Saturated	Saturated	32	60	0.0	0	7389.1	4050.7	50.4	0.75	0.789	1.703	2.2	0.00
62	126.4	Saturated	Saturated	32	60	0.0	0	7515.5	4114.7	50.2	0.74	0.785	1.693	2.2	0.00
63	126.4	Saturated	Saturated	32	60	0.0	0	7641.9	4178.7	50.0	0.74	0.781	1.683	2.2	0.00
64	126.4	Saturated	Saturated	32	60	0.0	0	7768.3	4242.7	49.8	0.73	0.777	1.674	2.2	0.00
65	126.5	Saturated	Saturated	75	65	0.0	0	7894.8	4306.8	116.2	0.73	0.774	1.665	2.2	0.00
66	126.5	Saturated	Saturated	75	65	0.0	0	8021.3	4370.9	115.7	0.72	0.770	1.655	2.2	0.00
67	126.5	Saturated	Saturated	75	65	0.0	0	8147.8	4435.0	115.3	0.72	0.766	1.646	2.1	0.00
68	125.9	Saturated	Saturated	80	70	0.0	0	8273.7	4498.5	122.5	0.71	0.762	1.638	2.1	0.00
69	125.9	Saturated	Saturated	80	70	0.0	0	8399.6	4562.0	122.0	0.71	0.758	1.629	2.1	0.00
70	125.9	Saturated	Saturated	80	70	0.0	0	8525.5	4625.5	121.6	0.71	0.755	1.620	2.1	0.00
											Total Liquefaction Settlement, S =			6.14 inches	

Geotechnologies, Inc.

Project: Century Housing
File No.: 21855
Description: Liquefaction Analysis - Maximum Considered Earthquake (2% Probability of Exceedance in 50 years)
Boring Number: 3

LIQUEFACTION EVALUATION (Idriss & Boulanger, EERI NO 12)

EARTHQUAKE INFORMATION:

Earthquake Magnitude (M):	7.3
Peak Ground Horizontal Acceleration, PGA (g):	0.893
Calculated Mag.Wtg.Factor:	1.054

GROUNDWATER INFORMATION:

Current Groundwater Level (ft):	4.5
Historically Highest Groundwater Level* (ft):	4.5
Unit Weight of Water (pcf):	62.4

* Based on California Geological Survey Seismic Hazard Evaluation Report

BOREHOLE AND SAMPLER INFORMATION:

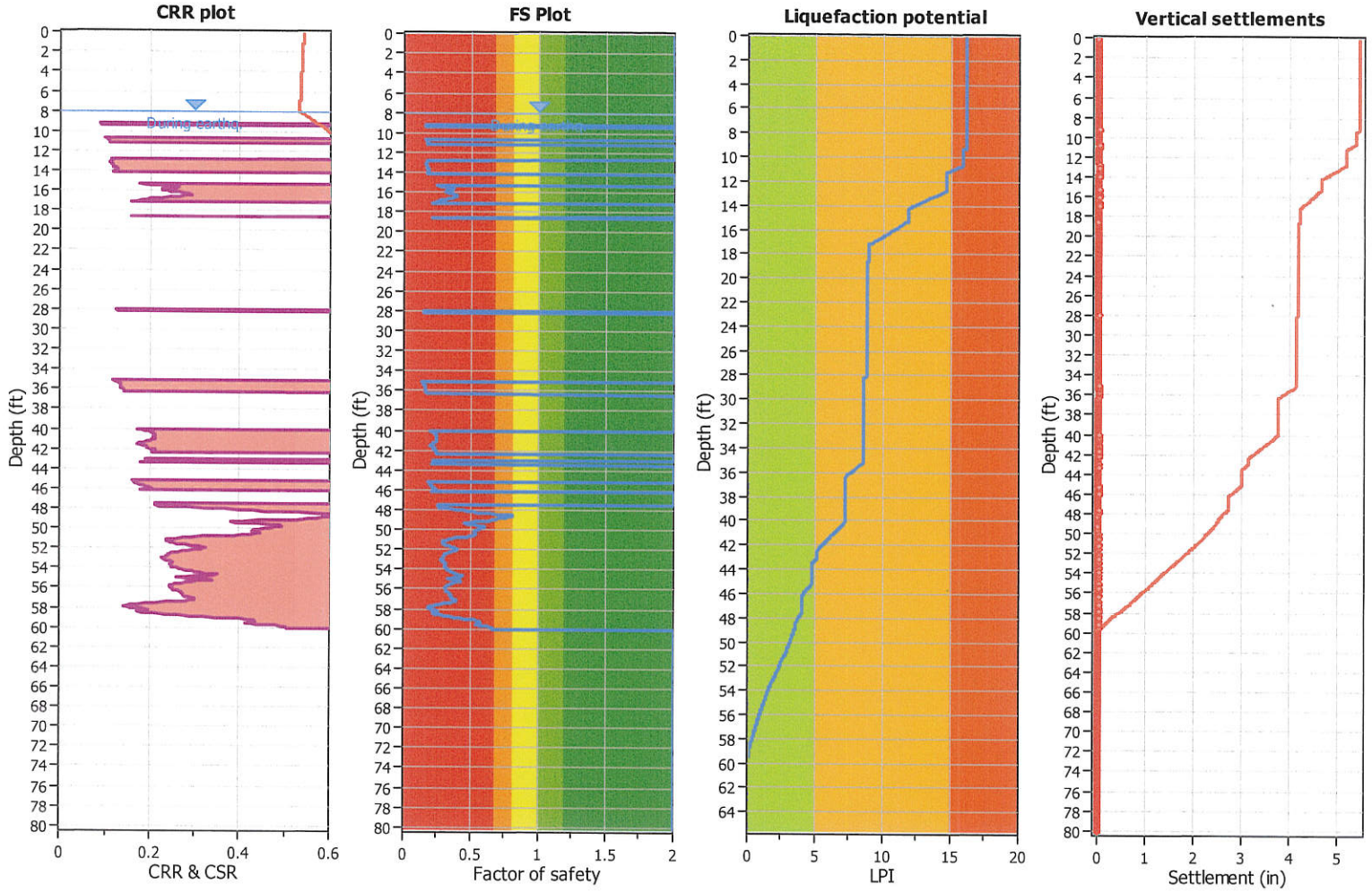
Borehole Diameter (inches):	8
SPT Sampler with room for Liner (Y/N):	Y

LIQUEFACTION BOUNDARY:

Plastic Index Cut Off (PI):	18
Minimum Liquefaction FS:	1

Depth to Base Layer (feet)	Total Unit Weight (pcf)	Current Water Level (feet)	Historical Water Level (feet)	Field SPT Blowcount N	Depth of SPT Blowcount (feet)	Fines Content #200 Sieve (%)	Plastic Index (PI)	Vetical Stress σ_{vc} (psf)	Effective Vert. Stress σ_{vc}' (psf)	Fines Corrected $(N_1)_{60\text{-}cs}$	Stress Reduction Coeff. r_d	Cyclic Shear Ratio CSR	Cyclic Resistance Ratio (CRR)	Factor of Safety CRR/CSR (F.S.)	Liquefaction Settlment ΔS_i (inches)
1	122.6	Unsaturated	Unsaturated	3	5	29.3	0	122.6	122.6	11.4	1.00	0.583	0.148	Non-Liq.	0.00
2	122.6	Unsaturated	Unsaturated	3	5	29.3	0	245.2	245.2	11.4	1.00	0.581	0.148	Non-Liq.	0.00
3	122.6	Unsaturated	Unsaturated	3	5	29.3	0	367.8	367.8	11.4	1.00	0.580	0.148	Non-Liq.	0.00
4	122.6	Unsaturated	Unsaturated	3	5	29.3	0	490.4	490.4	11.4	1.00	0.578	0.148	Non-Liq.	0.00
5	122.6	Saturated	Saturated	3	5	29.3	0	613.0	581.8	11.8	0.99	0.608	0.152	0.2	0.41
6	122.6	Saturated	Saturated	3	5	29.3	0	735.6	642.0	11.8	0.99	0.659	0.152	0.2	0.41
7	122.6	Saturated	Saturated	3	5	29.3	0	858.2	702.2	11.8	0.99	0.701	0.152	0.2	0.41
8	118.2	Saturated	Saturated	5	10	7.5	0	976.4	758.0	11.0	0.98	0.736	0.145	0.2	0.42
9	118.2	Saturated	Saturated	5	10	7.5	0	1094.6	813.8	11.7	0.98	0.766	0.150	0.2	0.41
10	118.2	Saturated	Saturated	5	10	7.5	0	1212.8	869.6	11.6	0.98	0.792	0.148	0.2	0.41
11	118.2	Saturated	Saturated	5	10	7.5	0	1331.0	925.4	11.1	0.97	0.814	0.143	0.2	0.42
12	118.2	Saturated	Saturated	5	10	7.5	0	1449.2	981.2	10.8	0.97	0.833	0.140	0.2	0.43
13	118.2	Saturated	Saturated	5	10	7.5	0	1567.4	1037.0	10.4	0.97	0.849	0.136	0.2	0.44
14	118.2	Saturated	Saturated	5	10	7.5	0	1685.6	1092.8	10.1	0.96	0.863	0.133	0.2	0.45
15	120.3	Saturated	Saturated	6	15	57.6	19	1805.9	1150.7	18.4	0.96	0.875	0.213	Non-Liq.	0.00
16	120.3	Saturated	Saturated	6	15	57.6	19	1926.2	1208.6	18.0	0.96	0.885	0.207	Non-Liq.	0.00
17	120.3	Saturated	Saturated	6	15	57.6	19	2046.5	1266.5	17.7	0.95	0.894	0.202	Non-Liq.	0.00
18	120.3	Saturated	Saturated	6	15	57.6	19	2166.8	1324.4	17.4	0.95	0.901	0.197	Non-Liq.	0.00
19	120.3	Saturated	Saturated	6	15	57.6	19	2287.1	1382.3	17.0	0.94	0.907	0.193	Non-Liq.	0.00
20	116.9	Saturated	Saturated	18	20	12.3	0	2404.0	1436.8	38.6	0.94	0.913	2.000	2.2	0.00
21	116.9	Saturated	Saturated	18	20	12.3	0	2520.9	1491.3	38.0	0.94	0.919	2.000	2.2	0.00
22	116.9	Saturated	Saturated	18	20	12.3	0	2637.8	1545.8	37.4	0.93	0.923	2.000	2.2	0.00
23	117.7	Saturated	Saturated	1	25	76.1	24	2755.5	1601.1	7.4	0.93	0.927	0.108	Non-Liq.	0.00
24	117.7	Saturated	Saturated	1	25	76.1	24	2873.2	1656.4	7.3	0.92	0.930	0.108	Non-Liq.	0.00
25	117.7	Saturated	Saturated	1	25	76.1	24	2990.9	1711.7	7.3	0.92	0.932	0.107	Non-Liq.	0.00
26	117.7	Saturated	Saturated	1	25	76.1	24	3108.6	1767.0	7.3	0.91	0.934	0.107	Non-Liq.	0.00
27	117.7	Saturated	Saturated	1	25	76.1	24	3226.3	1822.3	7.2	0.91	0.935	0.106	Non-Liq.	0.00
28	121.6	Saturated	Saturated	6	30	0.0	0	3347.9	1881.5	10.1	0.91	0.935	0.127	0.1	0.45
29	121.6	Saturated	Saturated	6	30	0.0	0	3469.5	1940.7	9.9	0.90	0.935	0.125	0.1	0.45
30	121.6	Saturated	Saturated	6	30	84.8	11	3591.1	1999.9	15.3	0.90	0.934	0.168	0.2	0.34
31	121.6	Saturated	Saturated	6	30	84.8	11	3712.7	2059.1	15.1	0.89	0.933	0.166	0.2	0.34
32	121.6	Saturated	Saturated	6	30	84.8	11	3834.3	2118.3	15.0	0.89	0.931	0.164	0.2	0.34
33	121.0	Saturated	Saturated	11	35	90.5	15	3955.3	2176.9	23.5	0.88	0.930	0.272	0.3	0.24
34	121.0	Saturated	Saturated	11	35	90.5	15	4076.3	2235.5	23.3	0.88	0.928	0.266	0.3	0.24
35	121.0	Saturated	Saturated	11	35	90.5	15	4197.3	2294.1	23.0	0.87	0.926	0.260	0.3	0.25
36	121.0	Saturated	Saturated	11	35	90.5	15	4318.3	2352.7	22.8	0.87	0.924	0.255	0.3	0.25
37	121.0	Saturated	Saturated	11	35	90.5	15	4439.3	2411.3	22.6	0.86	0.921	0.250	0.3	0.25
38	120.5	Saturated	Saturated	32	40	0.0	0	4559.8	2469.4	57.4	0.86	0.919	2.000	2.2	0.00
39	120.5	Saturated	Saturated	32	40	0.0	0	4680.3	2527.5	57.0	0.85	0.916	1.996	2.2	0.00
40	120.5	Saturated	Saturated	32	40	0.0	0	4800.8	2585.6	56.7	0.85	0.913	1.982	2.2	0.00
41	120.5	Saturated	Saturated	32	40	0.0	0	4921.3	2643.7	56.4	0.84	0.910	1.968	2.2	0.00
42	120.5	Saturated	Saturated	32	40	0.0	0	5041.8	2701.8	56.0	0.84	0.907	1.955	2.2	0.00
43	120.0	Saturated	Saturated	21	45	79.1	11	5161.8	2759.4	40.8	0.83	0.904	1.942	2.1	0.00
44	120.0	Saturated	Saturated	21	45	79.1	11	5281.8	2817.0	40.5	0.83	0.901	1.929	2.1	0.00
45	120.0	Saturated	Saturated	21	45	79.1	11	5401.8	2874.6	40.2	0.82	0.897	1.916	2.1	0.00
46	120.0	Saturated	Saturated	21	45	79.1	11	5521.8	2932.2	39.9	0.82	0.894	1.904	2.1	0.00
47	120.0	Saturated	Saturated	21	45	79.1	11	5641.8	2989.8	39.7	0.81	0.890	1.892	2.1	0.00
48	124.3	Saturated	Saturated	48	50	0.0	0	5766.1	3051.7	81.4	0.81	0.886	1.879	2.1	0.00
49	124.3	Saturated	Saturated	48	50	0.0	0	5890.4	3113.6	81.0	0.80	0.881	1.867	2.1	0.00
50	124.3	Saturated	Saturated	48	50	0.0	0	6014.7	3175.5	80.5	0.80	0.877	1.854	2.1	0.00
51	124.3	Saturated	Saturated	48	50	0.0	0	6139.0	3237.4	80.1	0.79	0.873	1.842	2.1	0.00
52	124.3	Saturated	Saturated	48	50	0.0	0	6263.3	3299.3	79.7	0.79	0.868	1.830	2.1	0.00
53	126.5	Saturated	Saturated	27	55	0.0	0	6389.8	3363.4	44.6	0.78	0.864	1.818	2.1	0.00
54	126.5	Saturated	Saturated	27	55	0.0	0	6516.3	3427.5	44.4	0.78	0.859	1.807	2.1	0.00
55	126.5	Saturated	Saturated	27	55	0.0	0	6642.8	3491.6	44.2	0.77	0.854	1.795	2.1	0.00
56	126.5	Saturated	Saturated	27	55	0.0	0	6769.3	3555.7	44.0	0.77	0.849	1.784	2.1	0.00
57	126.5	Saturated	Saturated	27	55	0.0	0	6895.8	3619.8	43.8	0.76	0.845	1.773	2.1	0.00
58	126.2	Saturated	Saturated	53	60	0.0	0	7022.0	3683.6	85.5	0.76	0.840	1.762	2.1	0.00
59	126.2	Saturated	Saturated	53	60	0.0	0	7148.2	3747.4	85.1	0.75	0.835	1.751	2.1	0.00
60	126.2	Saturated	Saturated	53	60	0.0	0	7274.4	3811.2	84.8	0.75	0.831	1.741	2.1	0.00
61	126.2	Saturated	Saturated	53	60	0.0	0	7400.6	3875.0	84.4	0.75	0.826	1.730	2.1	0.00
62	126.2	Saturated	Saturated	53	60	0.0	0	7526.8	3938.8	84.0	0.74	0.821	1.720	2.1	0.00
63	124.6	Saturated	Saturated	76	65	0.0	0	7651.4	4001.0	120.0	0.74	0.817	1.710	2.1	0.00
64	124.6	Saturated	Saturated	76	65	0.0	0	7776.0	4063.2	119.5	0.73	0.813	1.701	2.1	0.00
65	124.6	Saturated	Saturated	76	65	0.0	0	7900.6	4125.4	119.0	0.73	0.808	1.691	2.1	0.00
66	124.6	Saturated	Saturated	76	65	0.0	0	8025.2	4187.6	118.6	0.72	0.804	1.682	2.1	0.00
67	124.6	Saturated	Saturated	76	65	0.0	0	8149.8	4249.8	118.1	0.72	0.799	1.673	2.1	0.00
68	123.1	Saturated	Saturated	83	70	0.0	0	8272.9	4310.5	128.5	0.71	0.795	1.664	2.1	0.00
69	123.1	Saturated	Saturated	83	70	0.0	0	8396.0	4371.2	128.0	0.71	0.791	1.655	2.1	0.00
70	123.1	Saturated	Saturated	83	70	0.0	0	8519.1	4431.9	127.6	0.71	0.787	1.647	2.1	0.00
												Total Liquefaction Settlement, S =		7.34 inches	

Liquefaction analysis overall plots



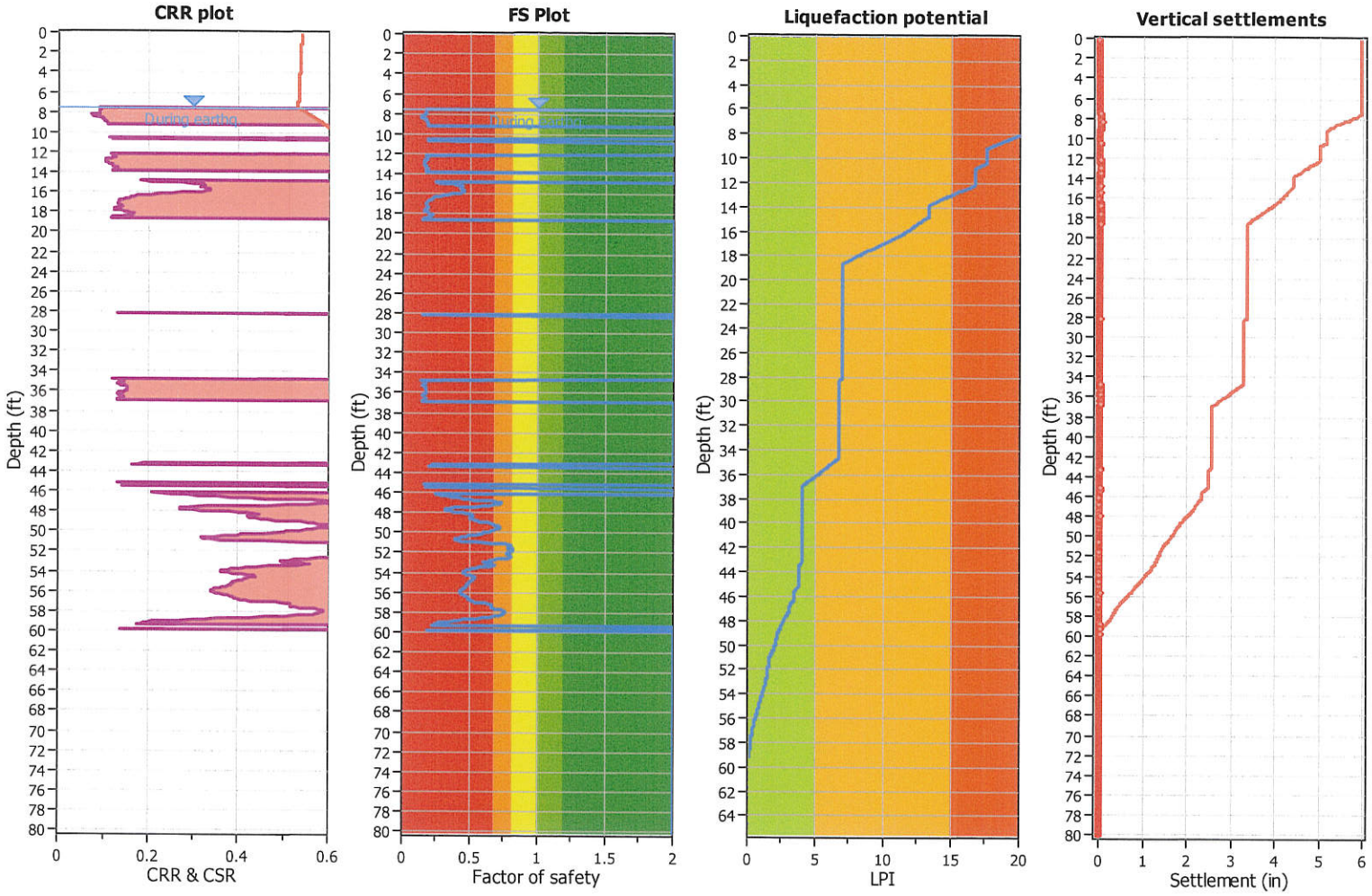
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.89	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 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

Liquefaction analysis overall plots



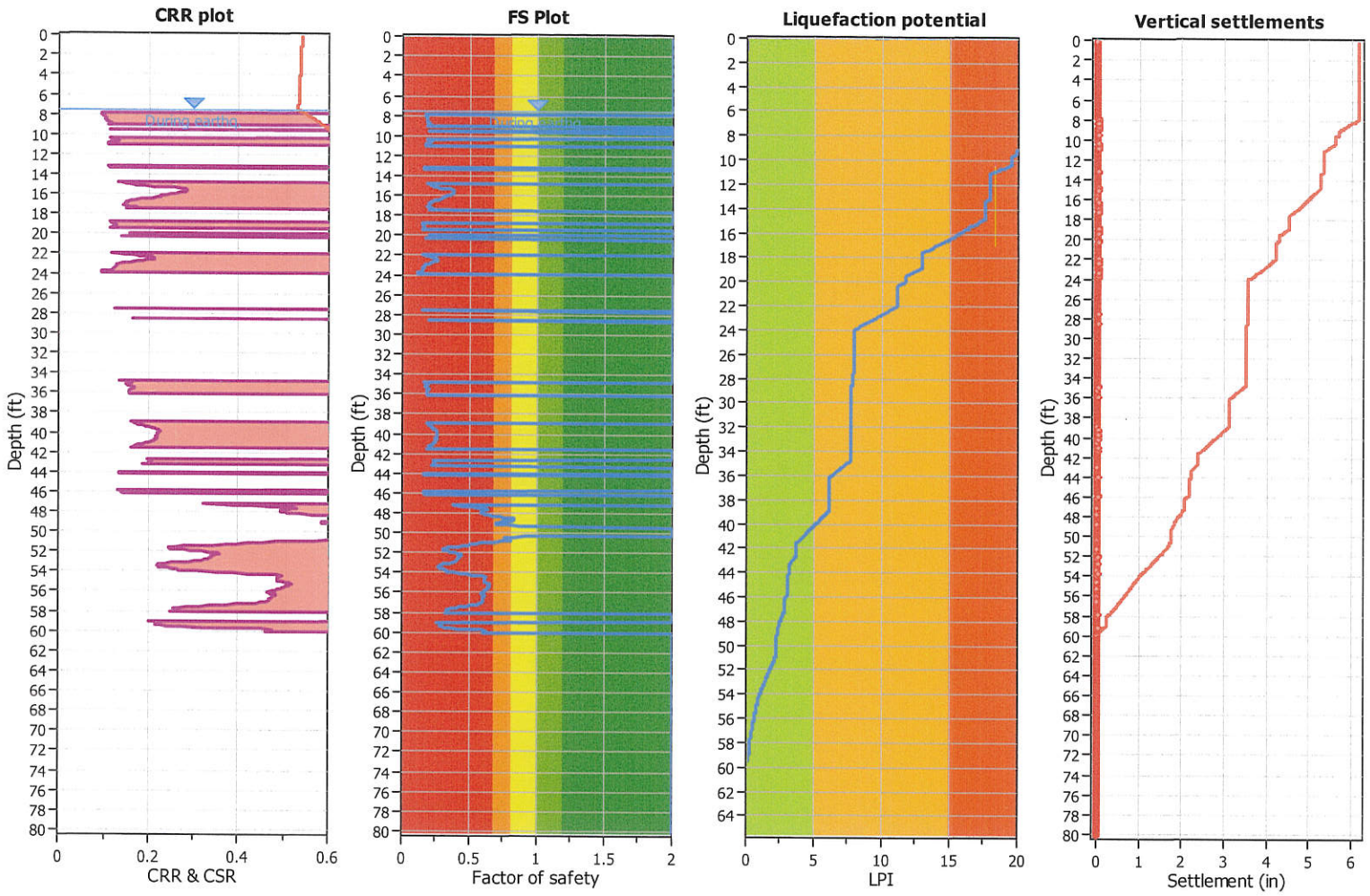
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	7.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.89	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	7.50 ft	Fill height:	N/A	Limit depth:	60.00 ft

F.S. color scheme

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Liquefaction analysis overall plots



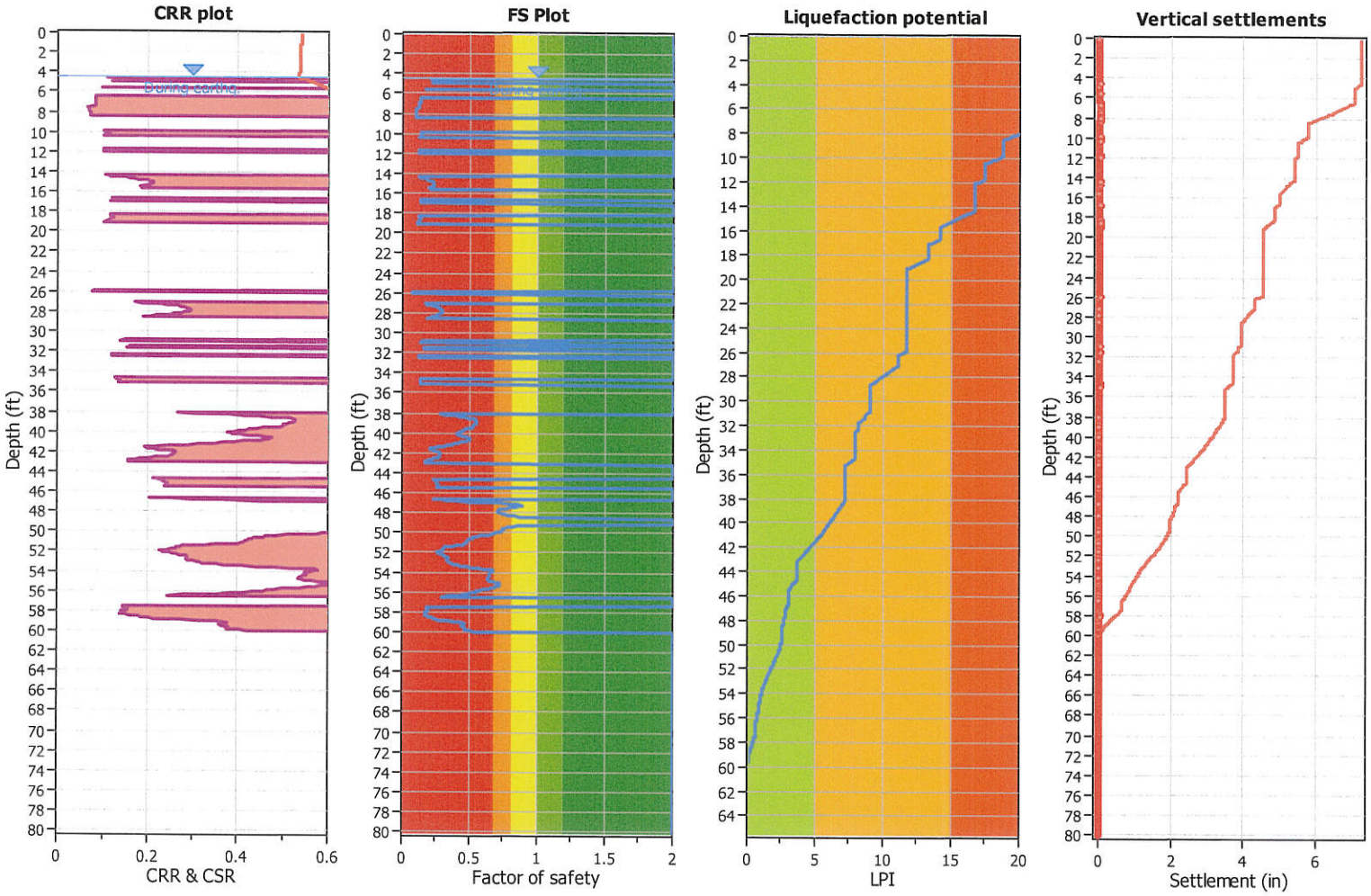
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	7.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.89	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	7.50 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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.89	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	4.50 ft	Fill height:	N/A	Limit depth:	60.00 ft

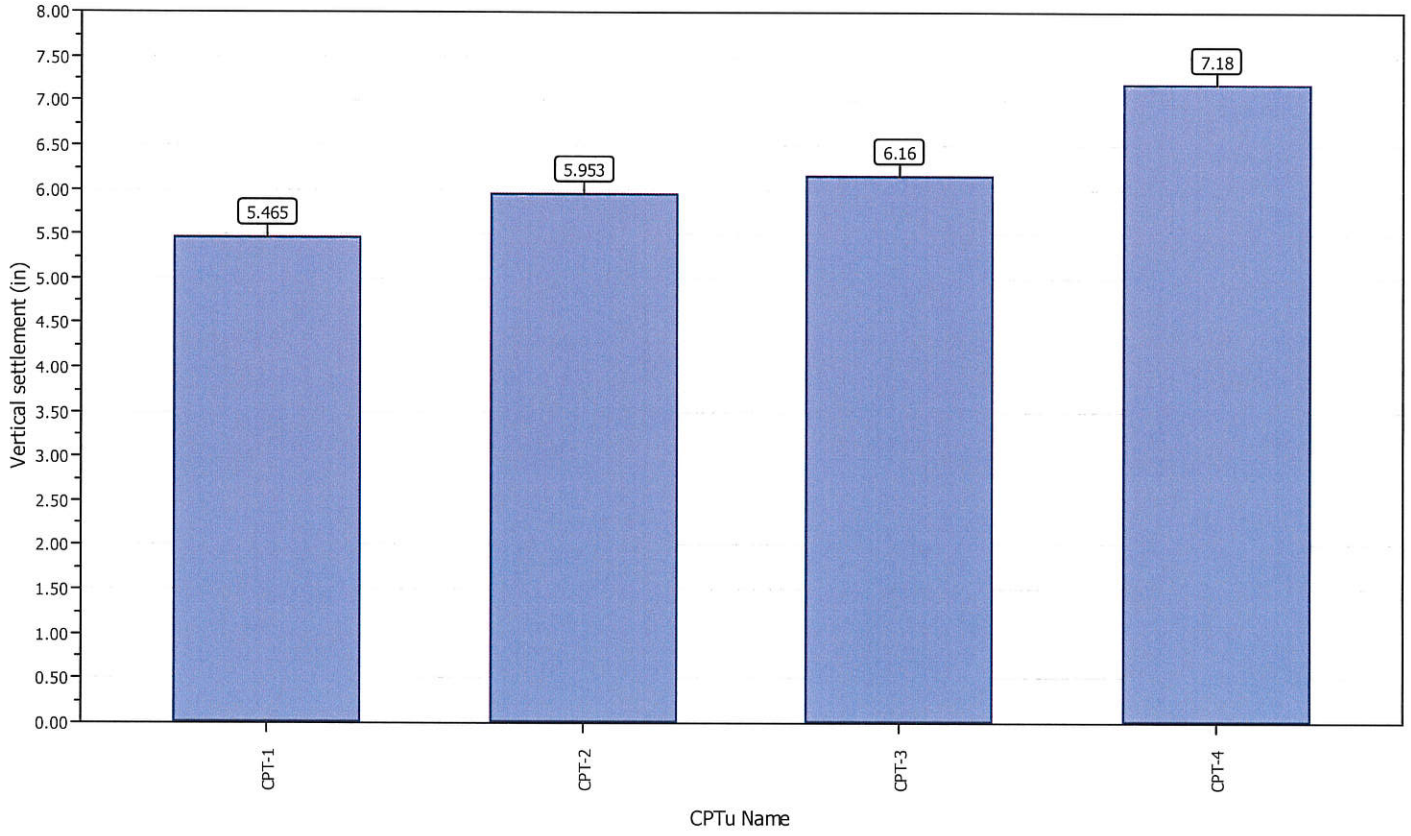
F.S. color scheme

- 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

Project title : File No. 21855 - Century Housing

Location : 2221 W. Williams St. Long Beach, CA

Overall vertical settlements report



October 25, 2019 via email: stang@geoteq.com

GEOTECHNOLOGIES, INC.
439 Western Ave.
Glendale, CA 91201

Attention: Mr. Stanley Tang

Re: Soil Corrosivity Study
Century Housing
Long Beach, CA
HDR #19-0693, GI #21855

Introduction

Laboratory tests have been completed on three soil samples provided for the referenced project. The purpose of these tests was to determine if the soils might have deleterious effects on underground utility piping, hydraulic elevator cylinders, and concrete structures. HDR Engineering, Inc. (HDR) assumes that the samples provided are representative of the most corrosive soils at the site.

The proposed residential structure has four stories and no subterranean levels. The site is located at 2221 Williams Street in Long Beach, California, and the water table is reportedly approximately 4.5 feet deep.

The scope of this study is limited to a determination of soil corrosivity and general corrosion control recommendations for materials likely to be used for construction. HDR's recommendations do not constitute, and are not meant as a substitute for, design documents for the purpose of construction. If the architects and/or engineers desire more specific information, designs, specifications, or review of design, HDR will be happy to work with them as a separate phase of this project.

Laboratory Soil Corrosivity Tests

The electrical resistivity of each sample was measured in a soil box per ASTM G187 in its as-received condition and again after saturation with distilled water. Resistivities are at about their lowest value when the soil is saturated. The pH of the saturated samples was measured per ASTM G51. A 5:1 water:soil extract from each sample was chemically analyzed for the major soluble salts commonly found in soil per ASTM D4327, ASTM D6919, and Standard Method 2320-B. Laboratory test results are shown in the attached Table 1.

Soil Corrosivity

A major factor in determining soil corrosivity is electrical resistivity. The electrical resistivity of a soil is a measure of its resistance to the flow of electrical current. Corrosion of buried metal is an electrochemical process in which the amount of metal loss due to corrosion is directly proportional to the flow of electrical current (DC) from the metal into the soil. Corrosion currents, following Ohm's Law, are inversely proportional to soil resistivity. Lower electrical resistivities result from higher moisture and soluble salt contents and indicate corrosive soil.

A correlation between electrical resistivity and corrosivity toward ferrous metals is:[1]

Soil Resistivity in ohm-centimeters	Corrosivity Category
Greater than 10,000	Mildly Corrosive
2,001 to 10,000	Moderately Corrosive
1,001 to 2,000	Corrosive
0 to 1,000	Severely Corrosive

Other soil characteristics that may influence corrosivity towards metals are pH, soluble salt content, soil types, aeration, anaerobic conditions, and site drainage.

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[1] *Romanoff, Melvin. Underground Corrosion, NBS Circular 579. Reprinted by NACE. Houston, TX, 1989, pp. 166–167.*

Electrical resistivities were in the moderately to severely corrosive categories with as-received moisture and at saturation. Some as-received resistivities were at or near their saturated values.

Soil pH values varied from 7.8 to 8.9. This range is mildly to strongly alkaline. Soil with a pH greater than 8.5 may be aggressive to aluminum.

The soluble salt content of the samples ranged from low to moderate Chloride and sulfate were found at low concentrations.

Nitrate was detected in low concentrations. Ammonium was not detected.

Tests were not made for sulfide and oxidation-reduction (redox) potential because these samples did not exhibit characteristics typically associated with anaerobic conditions.

This soil is classified as severely corrosive to ferrous metals and aggressive to aluminum.

Corrosion Control Recommendations

The life of buried materials depends on thickness, strength, loads, construction details, soil moisture, etc., in addition to soil corrosivity, and is, therefore, difficult to predict. Of more practical value are corrosion control methods that will increase the life of materials that would be subject to significant corrosion.

The following recommendations are based on the soil conditions discussed in the Soil Corrosivity section above. Unless otherwise indicated, these recommendations apply to the entire site or alignment.

Steel Pipe

1. Underground steel pipe with rubber gasketed, mechanical, grooved end, or other nonconductive type joints should be bonded for electrical continuity. Electrical continuity is necessary for corrosion monitoring and cathodic protection.
2. Install corrosion monitoring test stations to facilitate corrosion monitoring and the application of cathodic protection:
    a. At each end of the pipeline.
    b. At each end of all casings.

c. Other locations as necessary so the interval between test stations does not exceed 1,200 feet.

3. To prevent dissimilar metal corrosion cells and to facilitate the application of cathodic protection, electrically isolate each buried steel pipeline per NACE SP0286 from:

   a. Dissimilar metals.

   b. Dissimilarly coated piping (cement-mortar vs. dielectric).

   c. Above ground steel pipe.

   d. All existing piping.

4. Choose one of the following corrosion control options:

   OPTION 1

   a. Apply a suitable dielectric coating intended for underground use such as:

      i. Polyurethane per AWWA C222 *or*

      ii. Extruded polyethylene per AWWA C215 *or*

      iii. A tape coating system per AWWA C214 *or*

      iv. Hot applied coal tar enamel per AWWA C203 *or*

      v. Fusion bonded epoxy per AWWA C213.

   b. Apply cathodic protection to steel piping as per NACE SP0169.

   OPTION 2

   As an alternative to dielectric coating and cathodic protection, apply a ¾-inch cement mortar coating per AWWA C205 or encase in concrete 3 inches thick, using any type of ASTM C150 cement. Joint bonds, test stations, and insulated joints are still recommended for this alternative.

NOTE: Some steel piping systems, such as for oil, gas, and high-pressure piping systems, have special corrosion and cathodic protection requirements that must be evaluated for each specific application.

Hydraulic Elevators

1. Choose one of the following corrosion control options for the hydraulic steel cylinders.

   OPTION 1

   a. Coat hydraulic elevator cylinders with a suitable dielectric coating intended for underground use such as:

      i. Polyurethane per AWWA C222 or

      ii. Extruded polyethylene per AWWA C215 or

      iii. A tape coating system per AWWA C214 or

      iv. Hot applied coal tar enamel per AWWA C203 or

      v. Fusion bonded epoxy per AWWA C213.

   b. Electrically insulate each cylinder from building metals by installing dielectric material between the piston platen and car, insulating the bolts, and installing an insulated joint in the oil line.

   c. Apply cathodic protection to hydraulic cylinders as per NACE SP0169.

   OPTION 2

   As an alternative to electrical insulation and cathodic protection, place each cylinder in a plastic casing with a plastic watertight seal at the bottom.

2. The elevator oil line should be placed above ground if possible but, if underground, should be protected by one of the following corrosion control options:

   OPTION 1

   a. Provide a bonded dielectric coating.

   b. Electrically isolate the pipeline.

   c. Apply cathodic protection to steel piping as per NACE SP0169.

OPTION 2

Place the oil line in a PVC casing pipe with solvent-welded joints and sealed at both ends to prevent contact with soil and moisture.

Ductile Iron Pipe

1. To prevent dissimilar metal corrosion cells and to facilitate the application of cathodic protection, electrically insulate underground iron pipe from dissimilar metals and from above ground iron pipe with insulating joints per NACE SP0286.

2. Bond all nonconductive type joints for electrical continuity. Electrical continuity is necessary for corrosion monitoring and cathodic protection.

3. Install corrosion monitoring test stations to facilitate corrosion monitoring and the application of cathodic protection:

    a. At each end of the pipeline.

    b. At each end of any casings.

    c. Other locations as necessary so the interval between test stations does not exceed 1,200 feet.

4. Choose one of the following corrosion control options:

    OPTION 1

    a. Apply a suitable coating intended for underground use such as:

        i. Polyethylene encasement per AWWA C105; *or*

        ii. Epoxy coating; *or*

        iii. Polyurethane; *or*

        iv. Wax tape.

        NOTE: The thin factory-applied asphaltic coating applied to ductile iron pipe for transportation and aesthetic purposes does not constitute a corrosion control coating.

b. Apply cathodic protection to cast and ductile iron piping as per NACE SP0169.

OPTION 2

As an alternative to the coating systems described in Option 1 and cathodic protection, concrete encase all buried portions of metallic piping so that there is a minimum of 3 inches of concrete cover provided over and around surfaces of pipe, fittings, and valves using any type of ASTM C150 cement.

NOTE: Some iron piping systems, such as for fire water piping, have special corrosion and cathodic protection requirements that must be evaluated for each specific application.

Cast Iron Soil Pipe

1. Protect cast iron soil pipe with either a double wrap 4-mil or single wrap 8-mil polyethylene encasement per AWWA C105.
2. It is not necessary to bond the pipe joints or apply cathodic protection.
3. Provide 6 inches of clean sand backfill all around the pipe.

Clean Sand Backfill

1. Clean sand backfill must have the following parameters:
    a. Minimum saturated resistivity of no less than 3,000 ohm-cm; *and*
    b. pH between 6.0 and 8.0.
2. All backfill testing should be performed by a corrosion engineering laboratory.

Copper Tubing

1. Electrically insulate underground copper pipe from dissimilar metals and from above ground copper pipe with insulating devices per NACE SP0286.
2. Electrically insulate cold water piping from hot water piping systems.
3. Place cold water copper tubing in an 8-mil polyethylene sleeve or encase in double 4-mil thick polyethylene sleeves and bed and backfill with clean sand at least

2 inches thick surrounding the tubing. Copper tubing for cold water can also be treated the same as for hot water.

4. Hot water tubing may be subject to a higher corrosion rate. Protect hot copper tubing by one of the following measures:

    a. Preventing soil contact. Soil contact may be prevented by placing the tubing above ground or encasing the tubing with PVC pipe with solvent-welded joints. *or*

    b. Applying cathodic protection per NACE SP0169. The amount of cathodic protection current needed can be minimized by coating the tubing.

Plastic and Vitrified Clay Pipe

1. No special corrosion control measures are required for plastic and vitrified clay piping placed underground.

2. Protect all metallic fittings and valves with wax tape per AWWA C217, or with epoxy and appropriately sized cathodic protection per NACE SP0169.

All Pipe

1. On all pipes, appurtenances, and fittings not protected by cathodic protection, coat bare metal such as valves, bolts, flange joints, joint harnesses, and flexible couplings with wax tape per AWWA C217 after assembly.

2. Where metallic pipelines penetrate concrete structures such as building floors, vault walls, and thrust blocks use plastic sleeves, rubber seals, or other dielectric material to prevent pipe contact with the concrete and reinforcing steel.

Concrete Structures and Pipe

1. From a corrosion standpoint, any type of ASTM C150 cement may be used for concrete structures and pipe because the sulfate concentration is negligible, from 0 to 0.10 percent.[2,3,4]

2. Standard concrete cover over reinforcing steel may be used for concrete structures and pipe in contact with these soils due to the low chloride concentrations[5] found onsite. Limit the water-soluble chloride ion content in the concrete mix design to less than 0.3 percent by weight of cement.

3. Due to the high groundwater table encountered at this site, cyclical or continual wetting may be an issue. Any contact between concrete structures and groundwater should be prevented.

    a. For structures that extend below the water table, contact can be prevented with an impermeable waterproofing system. Options include a membrane such as Grace PrePrufe® products, a liquid applied barrier coating, or a waterproofing admixture such as Xypex® Admix. Visqueen, similar rolled barriers, or bentonite-based membranes are not viable waterproofing systems for corrosion protection.

    b. For structures above the water table, contact can be prevented with a gravel capillary break under the concrete and a vapor retarding membrane. Note that per ASTM E1643, "vapor retarders are not intended to provide a waterproofing function." [6] Alternatively, an impermeable waterproofing system may be used.

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[2] *2015 International Building Code (IBC) which refers to American Concrete Institute (ACI) 318-14 Table 19.3.2.1*

[3] *2015 International Residential Code (IRC) which refers to American Concrete Institute (ACI) 318-14 Table 19.3.2.1*

[4] *2016 California Building Code (CBC) which refers to American Concrete Institute (ACI) 318-14 Table 19.3.2.1*

[5] *Design Manual 303: Concrete Cylinder Pipe. Ameron. p.65*

[6] *ASTM E1643-11 (2017): Standard Practice for Selection, Design, Installation, and Inspection of Water Vapor Retarders Used in Contact with Earth or Granular Fill Under Concrete Slabs. ASTM International, 2017.*

Closure

The analysis and recommendations presented in this report are based upon data obtained from the laboratory samples. This report does not reflect variations that may occur across the site or due to the modifying effects of construction. If variations appear, HDR should be notified immediately so that further evaluation and supplemental recommendations can be provided.

HDR's services have been performed with the usual thoroughness and competence of the engineering profession. No other warranty or representation, either expressed or implied, is included or intended.

Please call if you have any questions.

Respectfully Submitted,
HDR Engineering, Inc.



James Keegan

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Enc: Table 1

Table 1 - Laboratory Tests on Soil Samples

Geotechnologies, Inc.
Century Housing
Your #21855, HDR Lab #19-0693SCS
14-Oct-19

Sample ID

			B1 @ 10'	B2 @ 5'	B3 @ 2.5'
Resistivity		**Units**			
as-received		ohm-cm	1,400	840	4,800
saturated		ohm-cm	1,320	840	2,720
pH			8.9	8.8	7.8
Electrical Conductivity		mS/cm	0.44	0.46	0.12
Chemical Analyses					
Cations					
calcium	Ca^{2+}	mg/kg	303	34	30
magnesium	Mg^{2+}	mg/kg	45	22	15
sodium	Na^{1+}	mg/kg	560	548	94
potassium	K^{1+}	mg/kg	37	33	9.6
Anions					
carbonate	CO_3^{2-}	mg/kg	257	200	ND
bicarbonate	HCO_3^{1-}	mg/kg	918	153	165
fluoride	F^{1-}	mg/kg	15	17	6.9
chloride	Cl^{1-}	mg/kg	46	61	56
sulfate	SO_4^{2-}	mg/kg	140	289	123
phosphate	PO_4^{3-}	mg/kg	26	13	1.3
Other Tests					
ammonium	NH_4^{1+}	mg/kg	ND	ND	ND
nitrate	NO_3^{1-}	mg/kg	5.0	10	5.3
sulfide	S^{2-}	qual	na	na	na
Redox		mV	na	na	na

Resistivity per ASTM G187, Cations per ASTM D6919, Anions per ASTM D4327, and Alkalinity per APHA 2320-B.
Electrical conductivity in millisiemens/cm and chemical analyses were made on a 1:5 soil-to-water extract.
mg/kg = milligrams per kilogram (parts per million) of dry soil.
Redox = oxidation-reduction potential in millivolts
ND = not detected
na = not analyzed

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