Appendix F

Updated Report of Preliminary Geotechnical Investigation

CHRISTIAN WHEELER
ENGINEERING

UPDATED REPORT OF PRELIMINARY GEOTECHNICAL INVESTIGATION

1122 4TH AVENUE DEVELOPMENT

1122 4TH AVENUE

SAN DIEGO, CALIFORNIA

PREPARED FOR

1122 4th AVE., LLC

301 N CARSON DRIVE, SUITE 205

BEVERLY HILLS, CALIFORNIA 90210

PREPARED BY

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June 30, 2015

1122 4th Ave., LLC
301 N Carson Drive, Suite 205
Beverly Hills, California 90210

CWE 2150381.01

Subject: **Updated Report of Preliminary Geotechnical Investigation**
Proposed 1122 4th Avenue Development
1122 4th Avenue, San Diego, California

In accordance with our Proposal dated June 17, 2015, we have completed an updated preliminary geotechnical investigation for the subject project. We are presenting herein our findings and recommendations.

In general, we found the subject property suitable for the proposed construction, provided the recommendations provided herein are followed. Based on the results of our investigation, we expect that the foundation zone for the subterranean parking garage will extend into dense formational soils that have moderately to relatively high strength parameters and relatively low settlement characteristics. Specific design criteria for shallow foundation systems are provided in the attached report.

If you have any questions after reviewing this report, please do not hesitate to contact our office. This opportunity to be of professional service is sincerely appreciated.

Respectfully submitted,
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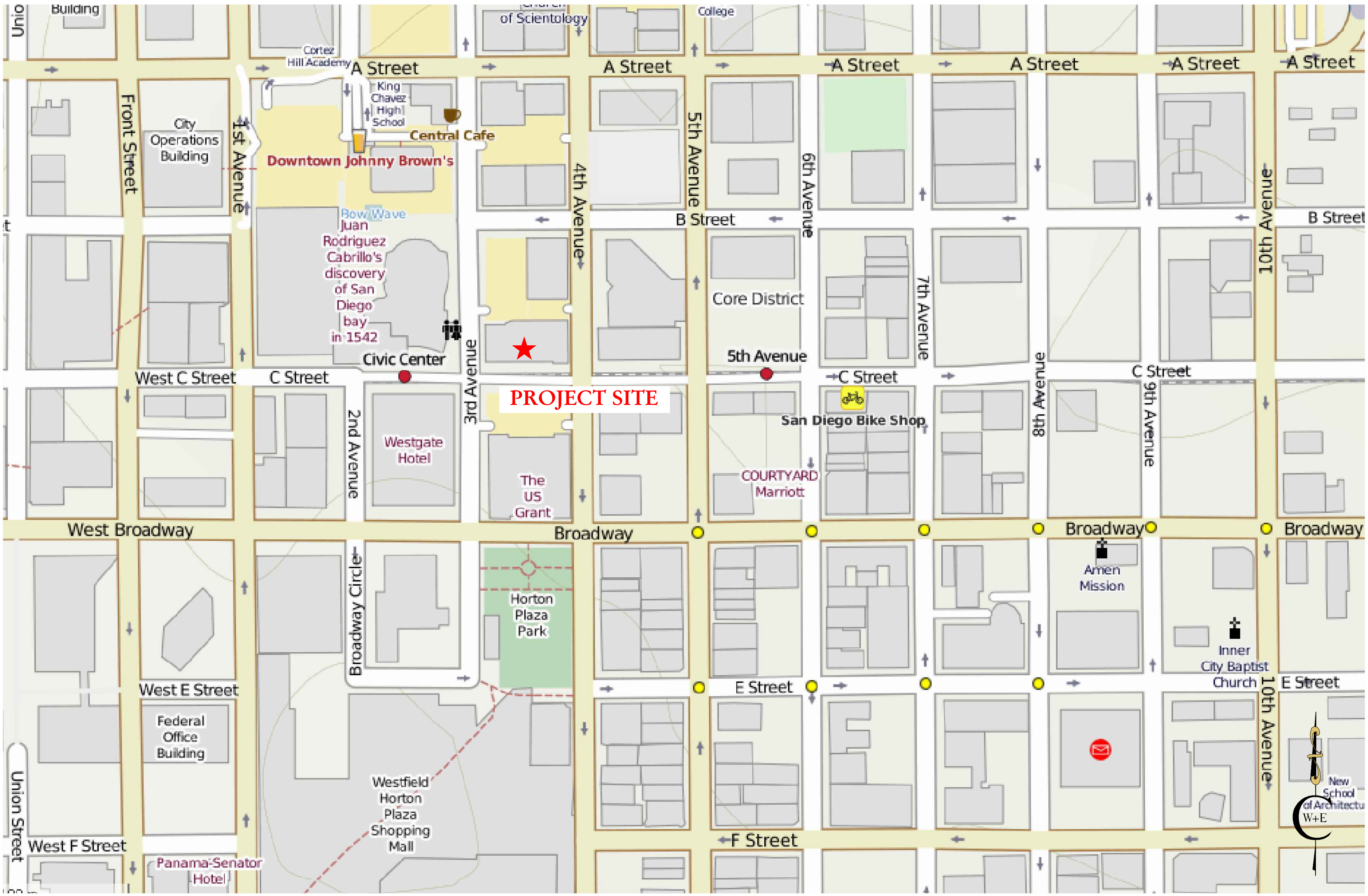
1122 4^{TH} AVENUE
SAN DIEGO, CALIFORNIA

INTRODUCTION AND PROJECT DESCRIPTION

This report presents the results of a previous preliminary geotechnical investigation that has been updated to address a proposed mixed-use project to be constructed at 1122 4^{th} Avenue, in the Downtown area of the city of San Diego, California. The following Figure Number 1 presents a vicinity map showing the location of the project.

The subject site includes the old California Theater and an attached 9-story office building. Based on our review of preliminary plans, we understand that the existing buildings will be demolished and replaced with a new mixed-use, podium-style development. The new development will include a 40-story residential tower core in the central portion of the site and a 9-story building in the eastern portion to "recreate" the existing structure. The remainder of the development will include two levels of underground parking, street level retail, lobby, and residential amenities, and four levels of above grade parking. The 7^{th} floor will include a podium deck with activity areas. We anticipate that the structures will consist of cast-in-place concrete construction and will be supported by shallow foundation systems. Grading will be limited to making the excavation for the proposed subterranean parking garage, which we expect will extend under the extents of the new construction.

This report has been prepared for the exclusive use of 1122 4^{TH} Ave., LLC, and its consultants for specific application to the project described herein. Should the project be modified, the new plans should be submitted to Christian Wheeler Engineering for review to determine whether the findings and recommendations presented herein remain applicable and if any additional subsurface investigation, laboratory testing and/or recommendations are necessary. Our professional services



SITE VICINITY MAP

©*OpenStreetMap contributors*

1122 4TH AVENUE DEVELOPMENT 4TH AVENUE AND C STREET SAN DIEGO, CALIFORNIA			
DATE:	JUNE 2015	JOB NO.:	2150381.01
BY:	SRD	FIGURE NO.:	1

CHRISTIAN WHEELER
ENGINEERING

have been performed, our findings obtained, and our recommendations prepared in accordance with generally accepted engineering principles and practices. This warranty is in lieu of all other warranties, expressed or implied.

PROJECT SCOPE

The update of our preliminary geotechnical investigation consisted of surface reconnaissance, review of previous subsurface explorations and laboratory test results, analysis of the field and laboratory data and review of relevant geologic literature. Our scope of service did not include additional subsurface exploration, assessment of hazardous substance contamination, recommendations to prevent floor slab moisture intrusion or the formation of mold within the structure, or any other services not specifically described in the scope of services presented below. More specifically, our intent was to provide the services listed below.

- Evaluate the subsurface conditions of the site to the depths influenced by the proposed construction by reviewing our previous subsurface explorations.
- Evaluate, by reviewing our previous laboratory tests and by relying on our past experience with similar soil types, the engineering properties of the various soil strata that may influence the proposed construction, including bearing capacities, expansive characteristics and settlement potential.
- Describe the general geology at the site including possible geologic hazards that could have an effect on the proposed construction, and provide the seismic design parameters as required by the 2013 edition of the California Building Code.
- Address potential construction difficulties that may be encountered due to soil conditions, groundwater or geologic hazards, and provide recommendations concerning these problems.
- Provide site preparation and grading recommendations for the anticipated work.
- Provide foundation recommendations for the type of construction anticipated and develop soil engineering design criteria for the recommended foundation designs.
- Provide design parameters for restrained and unrestrained retaining walls, as necessary.
- Prepare this report, which includes, in addition to our conclusions and recommendations, a plot plan showing the areal extent of the geological units and the locations of our exploratory borings, exploration logs, and a summary of the laboratory test results.

Although tests for the presence of chlorides and soluble sulfates, and for the pH and resistively of the soils that may be in contact with reinforced concrete were previously performed as part of the scope of our services, it should be understood Christian Wheeler Engineering does not practice corrosion engineering. If such an analysis is considered necessary, we recommend that the client retain an engineering firm that specializes in this field to consult with them on this matter. The results of these tests should only be used as a guideline to determine if additional testing and analysis is necessary.

FINDINGS

SITE DESCRIPTION

The subject property is a "L"-shaped area bordered by 3rd Avenue on the west, "C" Street on the south, 4th Avenue on the east, and to the north by an adjacent property that currently supports a street-level parking lot and structure. The site is identified as portions of Lots E, F, G, H, and I of Block 16 of Horton's Addition. The Assessor's Parcel Numbers for the property are 533-521-04, -05, and a portion of -06. On the east and west sides of the site are paved streets with concrete curbs and sidewalks approximately 12 feet in width. Small- to medium-size trees have been planted in the sidewalk areas along Third Avenue. On the south side of the site, "C" Street is closed to vehicles and is part of a downtown walking corridor with broad, brick sidewalks of varying widths, trees and shrubs in planters, with the two parallel San Diego Trolley tracks running down the center of "C" Street.

The site itself is approximately 25,000 square feet in area and has approximately 200 feet of frontage along "C" Street, approximately 100 feet of frontage along 3rd Avenue, and 150 feet along 4th Avenue. The subject property supports a vacant 9-story office building on the easterly side that was constructed in the 1920's and the California Theater. The office building has a full one-story basement. An apparent cistern, approximately three feet in diameter and four feet deep with a soil base, into which several pipes descended, was noted within the basement under the office building. This feature appeared to serve as a catch basin for overflow or pressure releases from other aspects of the building. The floor of the theater slopes downward towards the west, beginning at the ground floor level at the west end of the lobby to a basement level on the west end of the theater.

Apparently, neither of the structures has been used for several years. The entrance to the theater is from 4th Avenue, through the center of the office building. An exterior marquee extends out over the sidewalk along 4th Avenue. The lobby of the theater, with its various stairways, is within the footprint of the office building. The property also includes several retail shops around the south, west and east perimeter. At the corner of 4th Avenue and “C” Street, at street level, is a former restaurant area.

The south side of the structure adjacent to “C” Street is two stories, with a facade that appears to extend to a third story. This area is estimated to extend 15 feet northward from the “C” Street sidewalk, in the portion of the building west of the restaurant at the corner of 4th Avenue and C Street. The space along 3rd Avenue also has office space on the second story and commercial space on the street level, apparently used most recently for a small convenience store and Mexican food restaurant.

Topographically, the area around the site is relatively level with elevations ranging from approximately 57 feet above Mean Sea Level within the northeast portion of the site to approximately 51 feet within the southwest portion of the site. The surrounding area for at least several blocks in all directions slopes gently to the south and west towards San Diego Bay.

GENERAL GEOLOGY AND SUBSURFACE CONDITIONS

GEOLOGIC SETTING AND SOIL DESCRIPTION: The subject site is located in the Coastal Plains Physiographic Province of San Diego County. Based on the results of our subsurface exploration and analysis of readily available, pertinent geologic and geotechnical literature, the area around the site was determined to be underlain by man-placed fill material above Quaternary-age old paralic deposits locally referred to as the Bay Point Formation (Kennedy, 1975). The encountered geologic units observed to underlie the subject site are described below in order of increasing age:

> **ARTIFICIAL FILL (Qaf):** Man placed fill materials were encountered within each of our three exploratory borings. In general, the fill materials were noted to extend to depths ranging from approximately four feet to thirteen feet below existing site grades and are probably associated with backfill for the basement retaining walls. Based on the basement

levels below the structures, we expect that if any fill underlies the building area, it will be located under the easterly portion of the theater area.

In general, the encountered fill materials were noted to consist of medium brown to dark brown, silty sands (SM) and clayey sands (SC) which were damp to moist and loose to medium dense in consistency. Based on our visual observations and experience with similar materials in the vicinity of the site, the fill materials are anticipated to possess a “low” expansion potential. Based on their variable density and potentially compressible nature, the existing fill materials are not considered suitable for support of settlement-sensitive improvements on-site.

OLD PARALIC DEPOSITS (Qop): Quaternary-age old paralic deposits were found to underlie the fill materials in each of our three borings. In general, the old paralic deposits were encountered at depths of 4 feet to 13 feet below existing site grades and were noted to extend to depths in excess of the maximum explored depth of 100 feet below existing site grades.

As noted within our exploratory borings, the old paralic deposits predominantly consisted of reddish-brown, medium brown, and grayish-brown, silty sands (SM) and silty sands-poorly graded sands (SM-SP). In general, the silty sands (SM) and slightly silty sands (SM-SP) were found to be dense to very dense. However, lesser amounts of medium dense to dense silty sands (SM) and clayey sands (SC) were also noted in the upper portions of our borings. The old paralic deposits were noted to be generally moist to very moist above, and saturated below the local, static groundwater table, which was noted within our exploratory borings at elevations ranging from about –1 foot to 5 feet.

Based on our visual observations and experience with similar materials in the vicinity of the subject site, the majority of the old paralic deposits are anticipated to possess “very low” to “low” expansion potentials. Our laboratory testing indicated that the formational soils have moderate to relatively high strength parameters and relatively low settlement characteristics. In consideration of the competent nature of the encountered old paralic deposits, such materials are considered suitable to support of the proposed high-rise structure, provided the foundation recommendations presented herein are followed.

It should be noted that, although not encountered within any of our subsurface explorations performed at the subject site, our review of the referenced geotechnical literature and our experience within the vicinity of the site indicated that relatively minor, lenticular beds of sandy clays (CL) may be also present within the old paralic deposits that underlie the site. Furthermore, it should be recognized that, although the majority of the old paralic deposits were noted to be slightly to moderately cemented and cohesive, the silty sands-poorly graded sands (SM-SP) were noted to be relatively friable when disturbed, and the formation may contain some layers of relatively cohesionless sands. As such, special consideration should be taken when conducting temporary excavations that expose such friable sands.

GEOLOGIC STRUCTURE: Direct observation of the geologic structure within the old paralic deposits that underlie the site was not possible due to the small diameter nature of our exploratory borings drilled during our subsurface exploration. However, based on our past experience within the vicinity of the subject site, we anticipate the old paralic deposits to be generally massive with faint bedding that ranges from nearly horizontal to displaying dip angles of up to 3° towards the southwest.

GROUNDWATER: Groundwater was encountered within each of our previous exploratory borings performed across the site at approximate depths of 50 to 54 feet below existing site grades. These depths correspond to elevations ranging from +5 feet on the north side of the site, to -1 foot on the south side. The approximate levels of the local groundwater table, as observed during our subsurface explorations, are presented on the logs of our exploratory borings, included in Appendix A.

Although no seepage or wet soil was encountered above the local groundwater table in any of our subsurface explorations, it should be recognized that minor groundwater seepage problems might occur after development of a site even where none were present before development. These are usually minor phenomena and are often the result of an alteration in drainage patterns and/or an increase in irrigation water. Based on the permeability characteristics of the soil and the anticipated usage and development, it is our opinion that any seepage problems which may occur will be minor in extent. It is further our opinion that these problems can be most effectively corrected on an individual basis if and when they occur.

TECTONIC SETTING: It should be noted that much of Southern California, including the San Diego County area, is characterized by a series of Quaternary-age fault zones that consist of several individual, en echelon faults that generally strike in a northerly to northwesterly direction. Some of these fault zones (and the individual faults within the zone) are classified as “active” according to the criteria of the California Division of Mines and Geology. Active fault zones are those that have shown conclusive evidence of faulting during the Holocene Epoch (the most recent 11,000 years). The Division of Mines and Geology used the term “potentially active” on Earthquake Fault Zone maps until 1988 to refer to all Quaternary-age (last 1.6 million years) faults for the purpose of evaluation for possible zonation in accordance with the Alquist-Priolo Earthquake Fault Zoning Act and identified all Quaternary-age faults as “potentially active” except for certain faults that were presumed to be inactive based on direct geologic evidence of inactivity during all of Holocene time or longer. Some faults considered to be “potentially active” would be considered to be “active” but lack specific criteria used by the State Geologist, such as *sufficiently active* and *well-defined*. Faults older than Quaternary-age are not specifically defined in Special Publication 42, Fault Rupture Hazard Zones in California, published by the California Division of Mines and Geology. However, it is generally accepted that faults showing no movement during the Quaternary period may be considered to be “inactive”. The City of San Diego guidelines indicate that since the beginning of the Pleistocene Epoch marks the boundary between “potentially active” and “inactive” faults, unfaulted Pleistocene-age deposits are accepted as evidence that a fault may be considered to be “inactive”.

The subject site is located within the zone of influence of the active Rose Canyon Fault Zone and is located in the City of San Diego Downtown Special Fault Zone as shown on the City Seismic Safety Study Map No. 17. The site is not currently within an Alquist-Priolo Earthquake Fault Zone, but it should be recognized that the site is situated 8 blocks west of a portion of the Rose Canyon Fault Zone that has recently (November 1, 1991) been designated as an Alquist-Priolo Earthquake Fault Zone. Additionally, the site is located on the third block to the east of a portion of the Rose Canyon Fault Zone that, on May 1, 2003, was designated to be within an Alquist-Priolo Earthquake Fault Zone.

The Alquist-Priolo Earthquake Fault Zoning Act as codified in the state of California Public Resources Code, Division 2, Chapter 7.5 requires the State Geologist to delineate special studies zones around Quaternary-age faults that are "sufficiently active and well-defined" as to be subject to surface rupture. Cities and Counties affected by the Alquist-Priolo Earthquake Fault Zoning Act are required to adopt zoning laws, ordinances, rules, and regulations for implementing the Act and must regulate specified "projects" within Special Studies Zones. The City of San Diego defines specified “projects” as new residential structures, additions to existing residential structures if the addition is larger than 500 square feet, and certain other structures deemed suitable for human occupancy.

A review of available geologic maps indicates that a portion of the active Rose Canyon Fault Zone is located approximately one-tenth of a mile west of the subject site. Other active fault zones in the region that could possibly affect the site include the Newport-Inglewood and Palos Verdes Fault Zones to the northwest, the Coronado Bank and San Clemente Fault Zones to the southwest, and the Elsinore, Earthquake Valley, San Jacinto, and San Andreas Fault Zones to the northeast. The following Table I presents those proximal faults, which are anticipated to most significantly contribute to the ground-motion hazard at the site.

TABLE I: PROXIMAL FAULT ZONES

Fault Zone	Distance	Max. Magnitude Earthquake
Rose Canyon	0.1 mile	7.2 Magnitude
Coronado Bank	13 miles	7.6 Magnitude
Newport-Inglewood	34 miles	7.1 Magnitude
Elsinore (Julian)	42 miles	7.1 Magnitude
Earthquake Valley	47 miles	6.5 Magnitude
Palos Verdes	60 miles	7.3 Magnitude
San Jacinto (Anza)	63 miles	7.2 Magnitude

GEOLOGIC HAZARDS

GENERAL: Based on the site’s location within the City of San Diego Downtown Special Fault Zone, a fault investigation will be required to determine if the site is underlain by active or potentially active faulting. Such investigations include the excavation and detailed logging of the expressed geologic conditions observed within continuous trenches excavated across the site. In the downtown San Diego area, site specific fault investigations are typically performed prior to site demolition or re-development. However, based on the location of the existing structures on-site our previous scope of geotechnical services did not include the performance of a full-scale fault hazard investigation for the project as our previous client intended for us to perform a fault investigation at the subject site after site demolition was completed. However, it may be possible at some future time to obtain permission from the adjacent property owner to the north to trench for a fault investigation as well as to obtain appropriate permits from the City of San Diego to perform additional trenching in the public right-of-way (sidewalks and streets) along the southern, eastern, and western perimeters of the site.

Although we do not anticipate the subject site to be underlain by active or potentially active faulting, the client must understand that both active and potentially active faults within the Rose Canyon Fault Zone have been documented within relatively close proximity, to the west and east of the subject site. It should be understood that if the subject site is found to be underlain by active or potentially active faulting, site re-development may not be feasible or will have to be redesigned to accommodate structural setbacks from fault traces. In consideration of this fact, Christian Wheeler Engineering should be contacted once site demolition is scheduled, or if it is desired to obtain the necessary permission and permitting to conduct fault trenches off of the subject site proper, in order that we may submit a proposal for the fault investigation services. For planning purposes, we anticipate that, based on post-demolition site conditions, a fault investigation at the site should take about one week on-site. If the fault hazard evaluation is to be conducted with trenches in the public right-of-way and on the adjacent parcel to the north, the field investigation could take several weeks to complete once initial trenching and geologic logging begins.

Provided the site is not underlain by active or potentially active faulting, other than the potential for seismically induced ground shaking, as described herein, the site should be safe from geologic hazards at the conclusion of construction, provided the recommendations contained herein are implemented and sound construction practices are followed. In our professional opinion and to the best of our knowledge that provided no active or potentially active faults underlie the site, the site is suitable for the proposed development.

SEISMIC SAFETY STUDY: As part of our investigation, we have reviewed the City of San Diego Seismic Safety Study. This study is the result of a comprehensive investigation of the city, which rates areas according to geological risk potential (nominal, low, moderate and high), and identifies any potential geotechnical hazards and/or describes geomorphic conditions. As described above, the City of San Diego Seismic Safety Study places the site in Hazard Category 13 which is assigned to downtown areas associated with fault zones, yet are not necessarily within the limits of known active or potentially active fault zones. The potential risks in Category 13 are considered to be moderate.

SEISMIC HAZARD: A likely geologic hazard to affect the site is ground shaking as a result of movement along one of the major active fault zones mentioned in the "Tectonic Setting" section of this report. Per Chapter 16 of the 2013 California Building Code (CBC), the Risk-Targeted Maximum Considered Earthquake (MCE_R) ground acceleration is that which results in the largest maximum response to horizontal ground motions with adjustments for a targeted risk of structural collapse equal to one percent in 50 years. Figures 1613..3.1(1) and 1613.3.1(2) of the CBC present MCE_R accelerations for short (0.2 sec.) and long (1.0 sec.) periods, respectively, based on a soil Site Class B (CBC 1613.3.2) and a structural damping of five percent. For the subject site, correlation with blow counts indicates that the upper 100 feet of geologic subgrade can be characterized as Site Class C. In this case, the mapped MCE_R accelerations are modified using the Site Coefficients presented in Tables 1613.3.3(1) and (2). The modified MCE spectral accelerations are then multiplied by two-thirds in order to obtain the design spectral accelerations. These seismic design parameters for the subject site (32.7170°, -117.1616°), based on Chapter 16 of the CBC, are presented in Table II below.

TABLE II: CBC 2013 EDITION – SEISMIC DESIGN PARAMETERS

CBC – Chapter 16 Section	Seismic Design Parameter	Recommended Value
Section 1613.3.2	Soil Site Class	C
Figure 1613.3.1 (1)	MCE_R Acceleration for Short Periods (0.2 sec), S_s	1.221 g
Figure 1613.3.1 (2)	MCE_R Acceleration for 1.0 Sec Periods (1.0 sec), S_1	0.470 g
Table 1613.3.3 (1)	Site Coefficient, F_a	1.000
Table 1613.3.3 (2)	Site Coefficient, F_v	1.330
Section 1613.3.3	S_{MS} = MCE_R Spectral Response at 0.2 sec. = $(S_s)(F_a)$	1.221 g
Section 1613.3.3	S_{M1} = MCE_R Spectral Response at 1.0 sec. = $(S_1)(F_v)$	0.625 g
Section 1613.3.4	S_{DS} = Design Spectral Response at 0.2 sec. = $2/3(S_{MS})$	0.814 g
Section 1613.3.4	S_{D1} = Design Spectral Response at 1.0 sec. = $2/3(S_{M1})$	0.471 g
Section 1803.2.12	PGA_M per Section 11.8.3 of ASCE 7	0.547 g

LANDSLIDE POTENTIAL AND SLOPE STABILITY: A detailed, deterministic slope stability analysis was not included within our scope of services. Our analysis of readily available, pertinent geologic literature indicates that the site is considered to be marginally susceptible to landsliding (Tan, 1995). However, based on our experience within the vicinity of the site, as well as the competent nature of the encountered old paralic deposits, it is our opinion that the risk of deep-seated slope instability problems can be considered to be low. It is anticipated that the proposed construction will not increase the potential for slope failure, on or immediately adjacent to the subject site, provided the recommendations provided in this report are followed.

EXPANSIVE SOIL CONDITIONS: The near-surface soils encountered on-site are anticipated to generally possess “very low” to “low” expansion potentials. However, as previously described in the Geologic Setting and Soil Description section of this report, although not encountered within any of our subsurface explorations, relatively minor amounts of detrimentally expansive clayey soils may be encountered within materials of the Bay Point Formation on-site. Other than thoroughly blending any encountered detrimentally expansive clayey soils, which if encountered at all are anticipated to be very minor, with less expansive sandy soils prior to the use of the clayey soils as structural fill, no special construction considerations are necessary.

LIQUEFACTION: The near-surface soils encountered at the site are not considered susceptible to liquefaction due to such factors as depth to the groundwater table, soil density and grain-size distribution.

FLOODING: As delineated on Flood Insurance Rate Map (FIRM) 06073C1885G prepared by the Federal Emergency Management Agency, the site is not located within a flood hazard zone.

TSUNAMIS: Tsunamis are great sea waves produced by submarine earthquakes or volcanic eruptions. Due to the site’s elevation and location, the risk of the site being affected by a tsunami is considered low.

SEICHES: Seiches are periodic oscillations in large bodies of water such as lakes, harbors, bays or reservoirs. Due to the site’s location, it should not be affected by seiches.

CONCLUSIONS

In general, we found the subject property to be suitable from a geotechnical perspective for the proposed development, provided the recommendations presented herein are followed.

Relatively loose artificial fill soils were found near the surface of the site within each of our exploratory borings. These materials are considered unsuitable in their present condition to support the planned structures. However, we expect that the proposed excavations for the two-story subterranean parking garage will remove all such unsuitable materials and expose competent old paralic deposits at the foundation level.

Several alternatives may be considered for support of the proposed structures. These alternatives include a structural mat foundation, continuous and spread foundations, driven piles, and drilled cast-in-place concrete piers. Our findings indicate that the old paralic deposits at the proposed foundation level are dense to very dense and will provide adequate support for the proposed structures. Therefore, based on these factors, the use of drilled piers or driven piles was not further considered. However, if large vertical resistance and/or additional uplift resistance are necessary in design, drilled piers and/or soil anchors could be utilized. Design criteria for pier foundations and soil anchors can be provided upon request.

Based on the anticipated depths of the excavations for the subterranean parking structure and the proximity of the perimeter walls to the property lines, it will be necessary to provide shoring to support the temporary excavation slopes around the perimeter of the project. Design criteria are provided herein for temporary shoring along with design criteria for foundations and retaining walls.

As noted previously, the site is located within the City of San Diego "Downtown Fault Zone." As such, a fault investigation to determine if fault traces cross the subject property is required by the City. Since the entire site is covered by the two structures, a fault investigation was not performed during our previous geotechnical study of the site in 2003. However, it may be possible at some future time to obtain permission from the adjacent property owner to the north to trench for a fault investigation as well as to obtain appropriate permits from the City of San Diego to perform additional trenching in the public right-of-way (sidewalks and streets) along the southern, eastern, and western perimeters of the site. Short of this, it would be necessary to perform such an investigation after demolition of the existing improvements on-site that are planned for removal.

The site is located in an area that is relatively free of geologic hazards that will have a significant effect on the proposed development. Presuming that the site is not underlain by active or potentially active faulting, the most likely geologic hazard that could affect the site is ground shaking due to seismic activity along one of the regional active faults. However, construction in accordance with the requirements of the most recent edition of the California Building Code and the local governmental agencies should provide a level of life-safety suitable for the type of development proposed.

RECOMMENDATIONS

GRADING AND EARTHWORK

GENERAL: All grading should conform to the guidelines presented in Appendix J of the California Building Code, the minimum requirements of the City of San Diego, and the recommendations of this report. Prior to grading, a representative of Christian Wheeler Engineering should be present at the pre-construction meeting to provide additional grading guidelines, if necessary, and to review the earthwork schedule.

OBSERVATION OF GRADING: Continuous observation by the Geotechnical Consultant is essential during the grading operation to confirm conditions anticipated by our investigation, to allow adjustments in design criteria to reflect actual field conditions exposed, and to determine that the grading proceeds in general accordance with the recommendations contained herein.

CLEARING AND GRUBBING: Site preparation should begin with the removal of the existing improvements that are designated for demolition. The removals should include all abandoned utilities, foundations, slabs, vegetation, construction debris and other deleterious materials from the site. This should include all significant root material. The resulting materials should be disposed of off-site in a legal dumpsite. All underground utilities that currently service the building and will be abandoned should be properly capped off at the property line

SITE PREPARATION: After clearing and grubbing, the site preparation is anticipated to involve excavation for the two-story subterranean parking garage. It is anticipated that this excavation will extend through all near-surface materials, which are considered to be unsuitable for structural support. However, if the proposed excavations do not completely remove the unsuitable materials, overexcavation will be necessary to remove the unsuitable materials and replace them as properly compacted fill.

In areas of improvements outside the building perimeter, such as entry driveways, removal of unsuitable near-surface materials may be required to the underlying competent formational material, or to a depth of at least three feet below subgrade, whichever is less. The on-site granular soils are generally considered suitable for use as structural fill and backfill. Construction debris should not be incorporated into any structural fills or backfill for utility trenches or retaining walls. In addition, structural fill and backfill should consist of "very low" to "low" expansive materials (Expansion Index of 50 or less when tested in accordance with ASTM D4829).

EXCAVATION CHARACTERISTICS: Based on our exploratory borings, the subsurface materials at the site appear to be readily excavatable to the anticipated depths with conventional heavy-duty earthmoving equipment in good working order. However, due to the possibly loose nature of the existing near surface materials exposed in our exploratory excavations, it should be expected that vertical trench excavations and steep temporary slopes could experience caving or sloughing in the near-surface materials. Further, it should be noted that some of the sands comprising the old paralic deposits are friable and may not stand well in temporary vertical cuts or in drilled holes for shoring and tiebacks.

PROCESSING OF REMOVAL BOTTOM: Prior to placing any new fill soils or constructing any new improvements in areas that have been overexcavated as recommended in the “Site Preparation” section of this report, the exposed soils should be scarified to a depth of 12 inches, moisture conditioned, and compacted to at least 90 percent relative compaction.

COMPACTION AND METHOD OF FILLING: Structural fill placed at the site, including retaining wall and utility trench backfill, should be compacted to a relative compaction of at least 90 percent of the maximum dry density, as determined by the latest edition of ASTM Laboratory Test D1557. Fill and backfill should be placed at or slightly above the optimum moisture content, in lifts six to eight inches thick, with each lift compacted by mechanical means. Fills should consist of approved earth material, free of trash or debris, roots, vegetation, or other materials determined to be unsuitable by the Geotechnical Consultant. Fill material should be free of rocks or lumps of soil in excess of six inches in maximum dimension. Based upon the results of our sub-surface exploration and laboratory testing, most of the on-site earth materials appear suitable for use as structural fill material.

TEMPORARY CUT SLOPES: Based on our understanding of the preliminary grading plan combined with our site preparation recommendations, we anticipate that temporary excavations will typically be less than about 25 feet in depth. Temporary cut slopes of up to 25 feet in height for retaining wall construction and/or site preparation can be excavated at an inclination of 1.0 to 1.0 (horizontal to vertical) or flatter. Our firm should be contacted to observe all temporary cut slopes during grading to ascertain that no unforeseen adverse conditions exist. No surcharge loads such as foundation loads, or soil or equipment stockpiles, vehicles, etc. should be allowed within a distance from the top of temporary slopes equal to half the slope height. Where there is not enough room to construct temporary slopes in accordance with the above recommendations, temporary shoring of the excavation sides will be necessary. Specific design parameters for shoring are included in the following section of this report.

The contractor is solely responsible for designing and constructing stable, temporary excavations and will need to shore, slope, or bench the sides of trench excavations as required to maintain the stability of the excavation sides. The contractor's "competent person", as defined in the OSHA Construction Standards for Excavations, 29 CFR, Part 1926, should evaluate the soil exposed in the excavations as part of the contractor's safety process. Temporary cut slopes should be constructed in accordance with the recommendations presented in this section. In no other case should slope height, slope inclination, or excavation depth, including utility trench excavation depth, exceed those specified in local, state, and federal safety regulations.

SURFACE DRAINAGE: No special recommendations are considered necessary in regard to surface drainage other than ensuring that the areas around the structure are sloped such that no water will pond adjacent to the perimeter retaining walls.

STORM WATER INFILTRATION/PERCOLATION BMPS: As described in "Appendix F -Storm Water Infiltration/Percolation BMPs" of the City of San Diego's Guidelines for Geotechnical Reports (2011 edition), several geotechnical and topographic characteristics of a given site are considered to be "unsuitable" when storm water infiltration/percolation BMPs are proposed at that site. Accordingly, storm water infiltration/percolation BMPs are considered to be unfeasible on sites that display such unsuitable conditions.

Based on our past experience at the site and our review of the referenced documents, at least two conditions that are defined as unsuitable in the city's technical guidelines are present at the subject site. Based on these conditions, which are listed below, it is our professional opinion and judgment that the use of storm water infiltration/percolation BMPs is not feasible at the subject site. The unsuitable site conditions include the following:

- The presence of "engineered compacted fill (structural fill) subject to hydro-consolidation" and,
- The fact that "changes in soil moisture content or rising groundwater level will adversely impact existing structures or improvements."

In addition to the unsuitable site conditions listed above, much of the engineered fill soils underlying the site are expected to contain layers of soil that contain greater than 20% clays and/or greater than 40% silts. Such soils are typically not considered suitable for infiltration/percolation BMPs due to their low permeability rates.

GRADING PLAN REVIEW: The final grading plans should be submitted to this office for review in order to ascertain that the recommendations of this report have been implemented, and that no additional recommendations are needed due to changes in the anticipated development plans.

TEMPORARY SHORING

GENERAL: Where it is not possible to construct temporary cut slopes in accordance with the previously recommended criteria, it will be necessary to use temporary shoring to support the proposed excavations. For shoring systems, we considered the use of cantilevered soldier pile walls and soldier pile walls using tieback anchors or internal bracing (rakers). We are including herein recommendations for cantilevered walls, braced shoring, and tieback anchors. We recommend that a specialty contractor with experience in shoring and bracing provide the shoring recommendations and plans. It is recommended that a "survey" be made of adjacent properties and structures prior to the start of grading and excavation in order to establish the existing condition of existing neighboring structures and to reduce the possibility of potential damage claims as a result of site grading.

SHORING DESIGN AND LATERAL PRESSURES: For design of cantilevered shoring, a triangular distribution of lateral earth pressure may be used. It may be assumed that retained soils having a level surface behind the cantilevered shoring will exert a lateral pressure equal to that developed by a fluid with a density of 35 pounds per cubic foot. Cantilevered shoring is normally limited to excavations that do not exceed approximately 15 feet in depth in order to limit the deflection at the tops of the soldier piles.

For heights of shoring greater than about 15 feet, the use of braced or tied-back shoring should be considered to limit deflection of the shoring system. For the design of tied-back or braced shoring, we recommend the use of a trapezoidal distribution of earth pressure. The recommended pressure distribution, for the case where the grade is level behind the shoring, should consist of a maximum pressure of 26H pounds per square foot beginning at 0.25H below the top of the shoring and terminating at 0.25H above the bottom, where "H" is the height of the shoring in feet.

In addition to the recommended earth pressure, the upper 10 feet of shoring adjacent to streets should be designed to resist an additional uniform lateral pressure of 100 pounds per square foot on all sides adjacent to streets to account for the effects of the adjacent street traffic. However, if the traffic is kept back at least 10 feet from the shoring, the traffic surcharge may be neglected.

DESIGN OF SOLDIER PILES: Soldier piles should be spaced no closer than two diameters on center. The allowable lateral bearing value (passive value) of the soils below the level of excavation may be assumed to be 400 pounds per square foot per foot of depth from the excavated surface, up to a maximum of 5,600 pounds per square foot. To develop the full lateral value, provisions should be taken to assure firm contact between the soldier piles and the undisturbed soils. The concrete placed in the soldier pile excavations may be a lean mix concrete. However, the concrete used in that portion of the soldier pile which is below the planned excavation level should be of sufficient strength to adequately transfer the imposed loads to the surrounding soils.

The frictional resistance between the soldier piles and the retained earth may be used in resisting the downward component of anchor loads. The coefficient of friction between the soldier piles and the retained earth may be taken as 0.5. This value is based on the assumption that uniform full bearing will be developed between the steel soldier beam and the lean-mix concrete and between the lean-mix concrete and the retained earth materials. In addition, the soldier piles below the excavated level may be used to resist downward loads. The frictional resistance between the concrete soldier piles and the soils below the excavated level may be taken as equal to 600 pounds per square foot.

LAGGING: Continuous lagging will be required between the soldier piles. The soldier piles and anchors should be designed for the full anticipated lateral pressure. However, the pressure on the lagging will likely be somewhat less due to arching in the soils. We recommend that the lagging be designed for a semi-circular distribution of earth pressure where the maximum pressure is 400 pounds per square foot at the mid-point between soldier piles, and zero pounds per square foot at the soldier piles. This value does not include any surcharge pressures.

Timber lagging may be used between the soldier piles to support the exposed soils. If lagging is to be left in-place, treated lumber should be used. If possible, structural walls should be cast directly against the shoring to eliminate the need for backfilling of a narrow space. Special provisions for wall drainage and waterproofing, such as the use of a prefabricated composite drain, should be specified by others where the structural walls are cast directly against the shoring.

TIEBACK ANCHOR DESIGN: Tieback friction anchors may be used to resist lateral loads. For preliminary design purposes, it may be assumed that the active wedge adjacent to the shoring is defined by a plane drawn at 35 degrees from the vertical through the bottom of the excavation. The anchors should extend at least 20 feet beyond the potential active wedge; this provision is to provide global stability for the shored wall as opposed to adequate friction for the anchors.

The capacities of anchors should be determined by testing of the initial anchors as outlined by the anchor designer. For preliminary design purposes, it may be estimated that for conventionally drilled, gravity-grouted anchors the average bond stress between the grout and soil will be 1,500 pounds per square foot. Only the bond stress developed beyond the active wedge should be used in resisting lateral loads. If the anchors are spaced at least 4 feet on centers, no reduction in the capacity of the anchors need be considered due to group action. In no event should the anchors extend less than the minimum length beyond the potential active wedge as given above.

ANCHOR TESTING: Since the actual load-carrying capacity of tieback anchors will depend on various site-specific factors, the tieback capacity should be verified by load testing. The load testing program should be specified by the design engineer and be approved by the Geotechnical Consultant.

Christian Wheeler Engineering should be contacted to observe the tieback anchor installation and testing of the completed anchors. The shoring contractor should provide all appropriate testing equipment, including properly calibrated hydraulic jacking equipment, pressure gauges, and dial gauges for measuring tieback anchor movement. All anchor testing shall be performed under the observation of our firm.

INTERNAL BRACING: Alternatively, rakers may be used to internally brace the soldier piles. The raker bracing may be supported laterally by temporary concrete footings (deadmen). For design of such temporary footings, poured with the bearing surface normal to rakers inclined at 45 to 60 degrees with the vertical, a bearing value of 4,000 pounds per square foot may be used, provided the shallowest point of the footing is at least 1 foot below the lowest adjacent grade. To reduce the movement of the shoring, the rakers should be preloaded or at least tightly wedged between the footings and the soldier piles.

DEFLECTIONS: We recommend from a geotechnical standpoint that the deflection at the top of the shoring not exceed about one inch. If greater deflection occurs during construction, additional bracing may be necessary. If desired to reduce the deflection of the shoring, a greater lateral earth pressure could be used in the shoring design.

MONITORING: Some means of monitoring the performance of the shoring system is recommended. The monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of the soldier piles approximately every 50 lineal feet. We will be pleased to discuss this further with the design consultants and the contractor when the design of the shoring system has been finalized.

CONVENTIONAL SHALLOW FOUNDATIONS

GENERAL: It is our opinion that the proposed structures may be supported by conventional continuous and isolated spread footings that are founded in competent old paralic deposits. Site retaining walls and other exterior structures can be supported by conventional footings that are founded in old paralic deposits and/or new compacted fill soil. The following recommendations are considered the minimum based on the anticipated soil conditions and are not intended to be in lieu of structural considerations. All foundations should be designed by a qualified structural engineer.

DIMENSIONS: New spread footings supporting the proposed buildings should be embedded at least 36 inches below the finish pad grade or 36 inches into competent old paralic deposits, whichever depth is greater. Continuous and isolated footings should have minimum widths of 36 and 48 inches, respectively. Footings with these minimum dimensions may be designed for an allowable soil bearing pressure of 10,000 pounds per square foot.

New spread footings supporting minor at-grade structures or building improvements should be embedded at least 18 inches below the finish pad grade. Continuous and isolated footings should have minimum widths of 12 and 24 inches, respectively. New spread footings supporting site retaining walls should be embedded at least 18 inches below the finish pad grade and should have a minimum width of 24 inches. For these improvements, footings with the above recommended minimum dimensions may be designed for an allowable soil bearing pressure of 2,500 pounds per square foot.

The allowable bearing capacities are for dead plus live load conditions and may be increased by one-third for combinations of temporary loads, such as those due to wind or seismic loads.

FOOTING REINFORCING: Reinforcement requirements for foundations should be provided by a structural engineer. However, based on the anticipated soil conditions, we recommend that the minimum reinforcing for light miscellaneous improvement supported by continuous footings consist of at least two No. 5 bars positioned near the bottom of the footing and at least two No. 5 bars positioned near the top of the footing.

LATERAL LOAD RESISTANCE: Lateral loads against foundations may be resisted by friction between the bottom of the footing and the supporting soil, and by the passive pressure against the footing. The coefficient of friction between concrete and soil may be considered to be 0.30. The passive resistance may be considered to be equal to an equivalent fluid weight of 300 pounds per cubic foot. This assumes the footings are poured tight against undisturbed soil. If a combination of the passive pressure and friction is used, the friction value should be reduced by one-third.

SETTLEMENT CHARACTERISTICS: Provided the recommendations presented in this report are followed, the anticipated total and differential foundation settlement is expected to be less than about 1 inch and ¾ inch over 40 feet, respectively. It should be recognized that minor cracks normally occur in concrete slabs and foundations due to shrinkage during curing or redistribution of stresses, therefore some cracks should be anticipated. Such cracks are not necessarily an indication of excessive vertical movements.

EXPANSIVE CHARACTERISTICS: The anticipated foundation soils are expected to have a medium expansion potential. The recommendations presented in this report reflect this condition.

STRUCTURAL MAT FOUNDATION

A structural mat foundation, consisting of a rigid reinforced concrete mat supported on competent formational soils, is expected to be used for support of the tower structures and possibly the rest of the subterranean structure. Thickness and reinforcement of the mat foundation should be in accordance with the recommendations of the project structural engineer. The mat may be designed using an allowable bearing capacity of 8,000 pounds per square foot. The recommended allowable bearing capacity may be increased by up to one-third when considering loads of a short duration such as wind or seismic forces.

Mat foundations typically experience some deflection due to loads placed on the mat and the reaction of the soils underlying the mat. A design coefficient of subgrade reaction, K_{v1}, of 180 pounds per cubic inch (pci) may be used for evaluating such deflections at the site. This value is based on the soil conditions encountered in our exploratory excavations and is considered as applied to a unit square foot area. The value should be adjusted for the design mat size. The coefficient of subgrade reaction K_b for a mat of a specific width may be evaluated using the following equation:

$$\mathbf{K_b = K_{v1}[(b+1)/2b]^2}$$

Where **b** is the least width of the foundation

Based on our preliminary evaluation and the net increase in the effective stress on the foundation soils, the anticipated total settlement for rigid mat foundation should be less than approximately one inch. Anticipated maximum differential settlements of approximately 50 percent of the total settlement will likely occur between the center of the base of the structures and the corners.

Lateral forces may be resisted by using either friction or passive pressure resistance. A coefficient of sliding friction of 0.40 between concrete mat foundation and the underling soil or rock may be used. For passive pressure design, an allowable equivalent fluid pressure of 400 pounds per cubic foot may be used. The frictional resistance and the passive resistance of the soils may be combined without reduction in determining the total lateral resistance.

FOUNDATION PLAN REVIEW

The final foundation plan and accompanying details and notes should be submitted to this office for review. The intent of our review will be to verify that the plans used for construction reflect the minimum dimensioning and reinforcing criteria presented in this section and that no additional criteria are required due to changes in the foundation type or layout. It is not our intent to review structural plans, notes, details, or calculations to verify that the design engineer has correctly applied the geotechnical design values. It is the responsibility of the design engineer to properly design/specify the foundations and other structural elements based on the requirements of the structure and considering the information presented in this report.

FOUNDATION EXCAVATION OBSERVATION

All foundation excavations should be observed by the Geotechnical Consultant prior to placing reinforcing steel or formwork in order to determine if the foundation recommendations presented herein are followed. All footing excavations should be excavated neat, level, and square. All loose or unsuitable material should be removed prior to the placement of concrete.

CORROSIVE CHARACTERISTICS

The corrosion potential of the on-site materials at the project site was evaluated for its effect on steel and concrete structural members. The corrosion potential was evaluated using the results of laboratory tests on samples obtained during the subsurface exploration. Laboratory testing was performed on representative soil samples to evaluate pH, minimum electrical resistivity, and chloride and soluble sulfate content. The pH and minimum electrical resistivity tests were performed in accordance with

Caltrans Test (CT) 643, and sulfate and chloride tests were performed in accordance with CT 417 and 422, respectively.

For a soil sample obtained at a depth of 35 feet, test results indicate that the pH of the soil was on the order of 8.4. Minimum electrical resistivity for the soil was in the range of 802 ohm-cm. Testing also indicated that soluble sulfate contents of the soil was on the order of 0.022 percent and that the chloride contents of the soil was on the order of 20 ppm. For a soil sample obtained at a depth of 50 feet, test results indicate that the pH of the soil was on the order of 8.3. Minimum electrical resistivity for the soil was in the range of approximately 5600 ohm-cm. Testing also indicated that soluble sulfate contents of the soil was on the order of 0.001 percent and that the chloride contents of the soil was on the order of 11 ppm.

In general, a soil is considered corrosive when the soil contains more than 500 ppm of chlorides, more than 0.20 percent sulfates or has an electrical resistivity of less than 1000 ohm-cm. Therefore, based on the soluble sulfate content and the chloride content, the soil is considered noncorrosive. Based on the minimum resistivity, the soils near the elevation of 35 feet below grade may be considered to be corrosive and those near the elevation of 50 feet below grade may be considered to be non-corrosive.

ON-GRADE SLABS

GENERAL: It is our understanding that the floor system of the proposed building will consist of a concrete slab-on-grade. The following recommendations are considered the minimum slab requirements based on the soil conditions and are not intended to be in lieu of structural considerations.

INTERIOR SLAB: We recommend that the interior slab-on-grade floors that are not subject to vehicular loads be at least 4 inches thick (actual) and should be reinforced with at least No. 3 bars spaced at 18 inches on center each way. The reinforcing bars should extend at least six inches into the foundations and should be supported by chairs and be positioned in the center of the slab.

UNDER-SLAB VAPOR RETARDERS: Steps should be taken to minimize the transmission of moisture vapor from the subsoil through the interior slabs where it can potentially damage the interior floor coverings. We recommend that the owner/contractor follow national standards for the installation of vapor retarders below interior slabs as presented in currently published standards including ACI 302, "Guide to Concrete Floor and Slab Construction" and ASTM E1643, "Standard Practice for Installation of Water Vapor Retarder Used in Contact with Earth or Granular Fill Under Concrete Slabs". If sand is placed above or below the vapor retarding material, it should have a sand equivalent of at least 30 and contain less than 20% passing the Number 100 sieve and less than 10% passing the Number 200 sieve.

We recommend that the flooring installer perform standard moisture vapor emission tests prior to the installation of all moisture-sensitive floor coverings in accordance with ASTM F1869 "Standard Test Method for Measuring Moisture Vapor Emission Rate of Concrete Subfloor Using Anhydrous Calcium Chloride".

EXTERIOR CONCRETE FLATWORK: Exterior concrete slabs on grade that will not support vehicular loads should have a minimum thickness of four inches. Exterior slabs abutting perimeter foundations should be doweled into the footings. All slabs should be provided with weakened plane joints in accordance with the American Concrete Institute (ACI) guidelines. Alternative patterns consistent with ACI guidelines can also be used. A concrete mix with a 1-inch maximum aggregate size and a water/cement ratio of less than 0.6 is recommended for exterior slabs. Lower water content will decrease the potential for shrinkage cracks. Both coarse and fine aggregate should conform to the latest edition of the "Standard Specifications for Public Works Construction" ('Greenbook").

Special attention should be paid to the method of concrete curing to reduce the potential for excessive shrinkage and resultant random cracking. It should be recognized that minor cracks occur normally in concrete slabs due to shrinkage. Some shrinkage cracks should be expected and are not necessarily an indication of excessive movement or structural distress.

It should be expected that lightweight exterior improvements such as concrete flatwork, curbs and gutters, and pavements that are underlain by expansive soils can experience some amount of heave damage even if thickened and more heavily reinforced. The potential for heave damage to exterior improvements can be lessened by placing a two-foot-thick mat of sandy soils with an expansion index of 50 or less below the improvements; however, the decision to do so is an economic decision that will need to be made by the owner. It may be more cost effective for this project to provide occasional maintenance, repair and/or replacement of light exterior improvements.

EARTH RETAINING WALLS

FOUNDATIONS: Foundations for retaining walls can be designed in accordance with the foundation recommendations previously presented.

EQUIVALENT FLUID PRESSURES: The active soil pressure for the design of unrestrained earth retaining structures with level backfill surface may be assumed to be equivalent to the pressure of a fluid weighing 35 pounds per cubic foot. In the design of walls restrained from movement at the top (non-yielding walls), the at-rest soil pressure may be assumed to be equivalent to the pressure of a fluid weighing 55 pounds per cubic foot, provided there is a level backfill surface. Non-yielding building retaining walls braced by multiple floor levels should be designed to resist a uniform horizontal soil pressure of 23H (in pounds per square foot), where "H" is the wall height in feet. Thirty percent of any area surcharge placed adjacent to the retaining wall may be assumed to act as a uniform horizontal pressure against the wall. Where vehicles will be allowed within ten feet of the retaining wall, a uniform horizontal pressure of 100 pounds per square foot should be added to the upper 10 feet of the retaining wall to account for the effects of adjacent traffic. Special cases such as a combination of shored and sloping temporary slopes, or other surcharge loads not described above, may require an increase in the design values recommended above. These conditions should be evaluated by the project geotechnical engineer on a case-by-case basis. If any other loads are anticipated, the Geotechnical Consultant should be contacted for the necessary increase in soil pressure. All values are based on a drained backfill condition.

If it is necessary to consider seismic pressure, it may be assumed to be equivalent to the pressure of a fluid weighing 12 pounds per cubic foot, but the pressure distribution should be inverted so that the highest value is at the top of the wall. This corresponds to an approximate pseudo-static acceleration (Kh) of 0.15g.

PASSIVE PRESSURES: The passive pressure for the prevailing soil conditions may be considered to be 400 pounds per square foot per foot of depth for foundations in fill soil. This pressure may be increased one-third for seismic loading. The coefficient of friction for concrete to soil may be assumed to be 0.40 for the resistance to lateral movement. When combining frictional and passive resistance, the friction should be reduced by one-third.

WATERPROOFING AND SUBDRAINS: The project architect should provide (or coordinate) waterproofing details for the retaining walls. The design values presented above are based on a drained backfill condition and do not consider hydrostatic pressures. Unless hydrostatic pressures are incorporated into the design, the retaining wall designer should provide a subdrain detail. Typical retaining wall subdrain details are presented as Plate No. 5 of this report. Additionally, outlets points for the retaining wall subdrains should be coordinated by the project civil engineer. For subterranean walls, it may be necessary to collect the subdrain water in sumps and then pump it to an appropriate outlet.

BACKFILL: All retaining wall backfill should be compacted to at least 90 percent relative compaction. It is anticipated that the on-site soils are suitable for use as backfill material provided the design parameters given herein are used in the wall design. Retaining walls should not be backfilled until the masonry/concrete has reached an adequate strength.

PRELIMINARY PAVEMENT SECTIONS

We expect that pavements will consist of Portland Cement Concrete pavement. Such pavement should have a minimum thickness of 6 inches and can be placed directly on properly compacted subgrade material. Prior to placing concrete pavements, the subgrade soil should be scarified to a depth of 12 inches and compacted to at least 95 percent of its maximum dry density at a moisture content one to three percent above optimum. It is further recommended that in areas where heavy traffic or point loads are anticipated, the slab be reinforced with at least No. 3 bars placed at 18 inches on center each way. Concrete pavement construction should comply with the requirements set forth in Sections 201-1.1.2 and 302-6 of the Standard Specifications for Public Works Construction (concrete Class 560-C-3250).

The outside edge of concrete slabs that will support wheel loads should have a thickened edge or integral curb. The thickened edge should be at least 2 inches thicker than the slab and should taper back to the recommended slab thickness 3 feet from the edge of the slab.

LIMITATIONS

REVIEW, OBSERVATION AND TESTING

The recommendations presented in this report are contingent upon our review of final plans and specifications. Such plans and specifications should be made available to the geotechnical engineer and engineering geologist so that they may review and verify their compliance with this report and with the California Building Code.

It is recommended that Christian Wheeler Engineering be retained to provide continuous soil engineering services during the earthwork operations. This is to verify compliance with the design concepts, specifications or recommendations and to allow design changes in the event that subsurface conditions differ from those anticipated prior to start of construction.

UNIFORMITY OF CONDITIONS

The recommendations and opinions expressed in this report reflect our best estimate of the project requirements based on an evaluation of the subsurface soil conditions encountered at the subsurface exploration locations and on the assumption that the soil conditions do not deviate appreciably from those encountered. It should be recognized that the performance of the foundations and/or cut and fill slopes may be influenced by undisclosed or unforeseen variations in the soil conditions that may occur in the intermediate and unexplored areas. Any unusual conditions not covered in this report that may be encountered during site development should be brought to the attention of the geotechnical engineer so that he may make modifications if necessary.

CHANGE IN SCOPE

This office should be advised of any changes in the project scope or proposed site grading so that we may determine if the recommendations contained herein are appropriate. This should be verified in writing or modified by a written addendum.

TIME LIMITATIONS

The findings of this report are valid as of this date. Changes in the condition of a property can, however, occur with the passage of time, whether they be due to natural processes or the work of man on this or adjacent properties. In addition, changes in the Standards-of-Practice and/or Government Codes may occur. Due to such changes, the findings of this report may be invalidated wholly or in part by changes beyond our control. Therefore, this report should not be relied upon after a period of two years without a review by us verifying the suitability of the conclusions and recommendations.

PROFESSIONAL STANDARD

In the performance of our professional services, we comply with that level of care and skill ordinarily exercised by members of our profession currently practicing under similar conditions and in the same locality. The client recognizes that subsurface conditions may vary from those encountered at the locations where our test pits, surveys, and explorations are made, and that our data, interpretations, and recommendations be based solely on the information obtained by us. We will be responsible for those data, interpretations, and recommendations, but shall not be responsible for the interpretations by others of the information developed. Our services consist of professional consultation and observation only, and no warranty of any kind whatsoever, express or implied, is made or intended in connection with the work performed or to be performed by us, or by our proposal for consulting or other services, or by our furnishing of oral or written reports or findings.

CLIENT'S RESPONSIBILITY

It is the client’s responsibility, or its representatives, to ensure that the information and recommendations contained herein are brought to the attention of the structural engineer and architect for the project and incorporated into the project's plans and specifications. It is further their responsibility to take the necessary measures to insure that the contractor and his subcontractors carry out such recommendations during construction.

FIELD EXPLORATIONS

Three subsurface explorations were made at the locations indicated on the site plan and geotechnical map included herewith as Plate No. 1 between April 23 and May 1, 2003. These explorations consisted of small-diameter, hollow-stem borings drilled using a truck-mounted drill rig. The fieldwork was conducted by or under the observation of our engineering geology personnel.

The boring logs are presented in the attached Appendix A. The soils are described in accordance with the Unified Soils Classification. In addition, a verbal textural description, the wet color, the apparent moisture and the density or consistency are provided. The density of granular soils is given as either very loose, loose, medium dense, dense or very dense.

Relatively undisturbed drive samples were collected using a modified California sampler. The sampler, with an external diameter of 3.0 inches, is lined with 1-inch long, thin, brass rings with inside diameters of approximately 2.4 inches. The sample barrel was driven into the ground with the weight of a 140-pound hammer falling 30 inches in general accordance with ASTM D 3550-84. The driving weight is permitted to fall freely. The number of blows per foot of driving, or as indicated, are presented on the boring logs as an index to the relative resistance of the sampled materials. The samples were removed from the sample barrel in the brass rings, and sealed. Bulk samples of the encountered earth materials were also collected. Samples were transported to our laboratory for testing.

LABORATORY TESTING

Laboratory tests were performed in accordance with the generally accepted American Society for Testing and Materials (ASTM) test methods or suggested procedures. Brief descriptions of the tests performed are presented below and the subsequent results are presented in Appendix B.

a) **CLASSIFICATION:** Field classifications were verified in the laboratory by visual examination. The final soil classifications are in accordance with the Unified Soil Classification System.

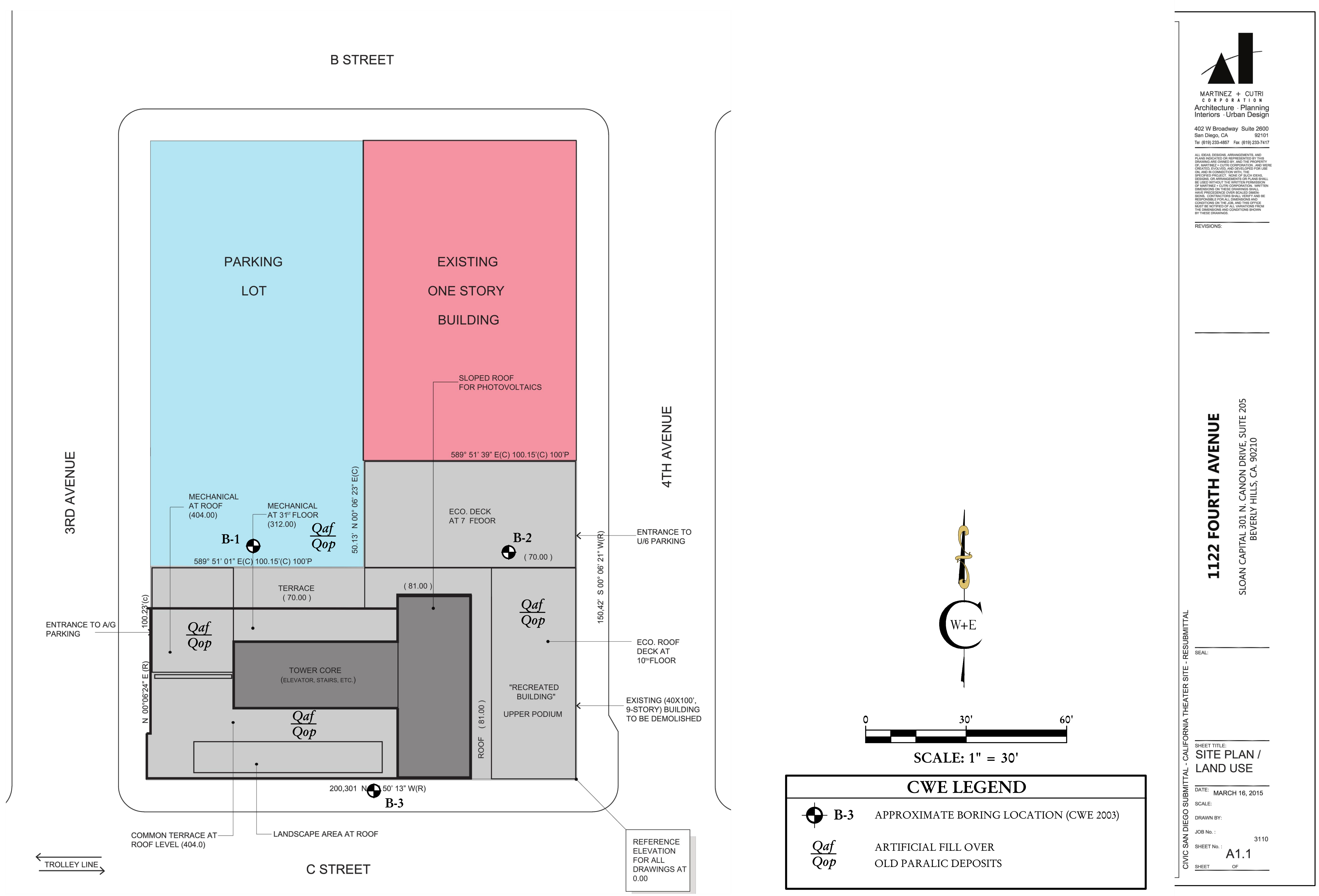
b) **MOISTURE-DENSITY:** In-place moisture contents and dry densities were determined for representative soil samples. This information was an aid to classification and permitted recognition of variations in material consistency with depth. The dry unit weight is determined in pounds per cubic foot, and the in-place moisture content is determined as a percentage of the soil's dry weight. The results of these tests are summarized in the boring logs.

c) **GRAIN SIZE DISTRIBUTION:** The grain size distribution was determined for selected representative soil samples in accordance with ASTM D422. The results of these tests are presented in Appendix B.

d) **DIRECT SHEAR TESTS:** Direct shear tests were performed on representative soil samples to determine the failure envelope based on yield shear strength. The shear box was designed to accommodate a sample having a diameter of 2.375 inches or 2.50 inches and a height of 1.0 inch. Samples were tested at different vertical loads and at saturated moisture content. The shear stress was applied at a constant rate of strain of approximately 0.05 inch per minute. The results of these tests are presented in Appendix B.

e) **CONSOLIDATION TEST:** Consolidation tests were performed on selected relatively “undisturbed” samples. The consolidation apparatus was designed to accommodate a 1-inch-high by 2.375-inch or 2.500-inch diameter soil sample laterally confined by a brass ring. Porous stones were placed in contact with the top and bottom of the sample to permit the addition of pore fluid during testing. Loads were applied to the sample in a geometric progression, after vertical movement ceased, resulting deformations were recorded. The percent consolidation is reported as the ratio of the amount of vertical compression to the original sample height. The test sample was inundated at some point in the test cycle to determine its behavior under the anticipated loads as soil moisture increases. In addition, at a selected vertical load, time versus settlement was recorded to determine the time rate characteristics of the soil. The results of the consolidation and time rate tests are presented in the form of a curve in Appendix B.

f) **SOLUBLE SULFATES, CHLORIDES, pH, and RESISTIVITY:** One representative soil sample was tested for its water-soluble sulfate content, chloride content, pH, and resistivity in accordance with California Tests 417, 422, and 643, respectively. The results of these tests are presented in Appendix B.

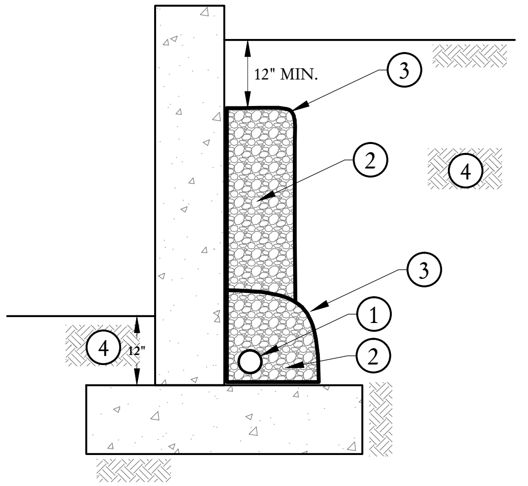


SITE PLAN AND GEOTECHNICAL MAP

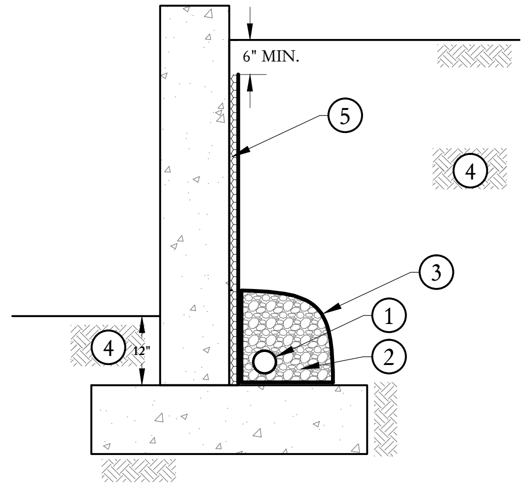
1122 4TH AVENUE DEVELOPMENT
1122 4TH AVENUE
SAN DIEGO, CALIFORNIA

DATE:	JUNE 2015	JOB NO.:	2150381.01
BY:	SRD	PLATE NO.:	1

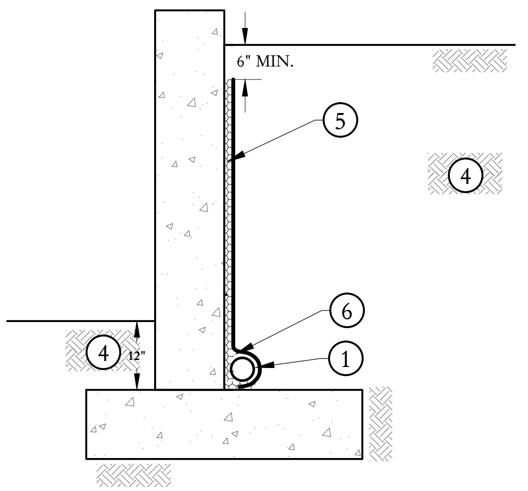
CHRISTIAN WHEELER
ENGINEERING



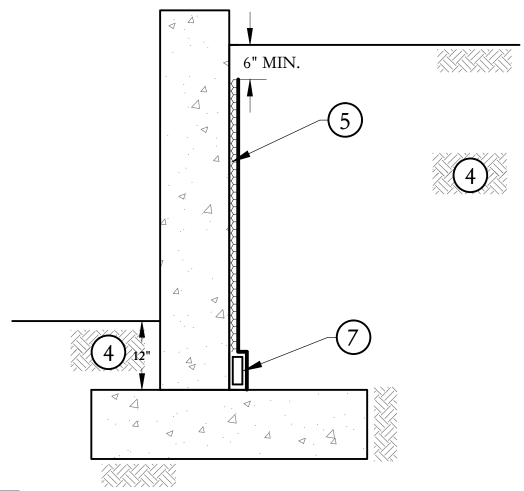
1 DETAIL



2 DETAIL



3 DETAIL



4 DETAIL

NOTES AND DETAILS

GENERAL NOTES:

1) THE NEED FOR WATERPROOFING SHOULD BE EVALUATED BY OTHERS.
2) WATERPROOFING TO BE DESIGNED BY OTHERS (CWE CAN PROVIDE A DESIGN IF REQUESTED).
3) EXTEND DRAIN TO SUITABLE DISCHARGE POINT PER CIVIL ENGINEER.
4) DO NOT CONNECT SURFACE DRAINS TO SUBDRAIN SYSTEM.

DETAILS:

1 4-INCH PERFORATED PVC PIPE ON TOP OF FOOTING, HOLES POSITIONED DOWNWARD (SDR 35, SCHEDULE 40, OR EQUIVALENT).

2 ¾ INCH OPEN-GRADED CRUSHED AGGREGATE.

3 GEOFARBRIC WRAPPED COMPLETELY AROUND ROCK.

4 PROPERLY COMPACTED BACKFILL SOIL.

5 WALL DRAINAGE PANELS (MIRADRAIN OR EQUIVALENT) PLACED PER MANUFACTURER'S REC'S.

6 UNDERLAY SUBDRAIN WITH AND CUT FABRIC BACK FROM DRAINAGE PANELS AND WRAP FABRIC AROUND PIPE.

7 COLLECTION DRAIN (TOTAL DRAIN OR EQUIVALENT) LOCATED AT BASE OF WALL DRAINAGE PANEL PER MANUFACTURER'S RECOMMENDATIONS.

CANTILEVER RETAINING WALL DRAINAGE SYSTEMS

1122 4TH AVENUE DEVELOPMENT
1122 4TH AVENUE
SAN DIEGO, CALIFORNIA

DATE:	JUNE 2015	JOB NO.:	2150381.01
BY:	SRD	PLATE NO.:	2

CHRISTIAN WHEELER
ENGINEERING

Appendix A

Boring Logs

LOG OF TEST BORING NUMBER B-1

Date Excavated:	4/23/2003	Logged by:	TSW
Equipment:	CME55	Project Manager:	CHC
Existing Elevation:	54.0 feet	Depth to Water:	50 feet
Finish Elevation:	N/A	Drive Weight:	140 lbs.

DEPTH (feet)	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: SAMPLE TYPE	SAMPLES: BULK	PENETRATION (blows/foot)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
	1½" layer of Asphaltic Concrete over 3" layer of Base.						
2	**Artificial Fill (Qaf):** Light brown, moist, loose, CLAYEY SAND (SC), occasional gravels.						SO$_4$ pH Ω
4		Cal		14			Cl
6		SPT		5			
8	At 9 feet becomes loose to medium dense.						
10		Cal		18	15.2	108.5	DS SA
12		SPT		6			
14	**Bay Point Formation (Qbp):** Reddish-brown to grayish-brown, moist, medium dense, CLAYEY SAND (SC), fine to coarse grained.	Cal		50			
		Cal		38	15.1	113.8	
16	Abundant gravels from 15' to 16'. At 16' grades to medium dense to dense.	SPT		39			
18	Reddish-brown, moist, very dense, SILTY SAND (SM), fine to medium grained, micaceous.						
20		Cal		50/5"	110.0	11.4	DS

Boring continued on Plate No. 3.

CHRISTIAN WHEELER
ENGINEERING

PROPOSED HIGH-RISE DEVELOPMENT
4th Avenue & C Street, San Diego, California

BY:	HF	DATE:	May 2003
JOB NO. :	203.338	PLATE NO.:	2

LOG OF TEST BORING NUMBER B-1 (Continued)

Date Excavated:	4/23/2003	Logged by:	TSW
Equipment:	CME55	Project Manager:	CHC
Existing Elevation:	54.0 feet	Depth to Water:	50 feet
Finish Elevation:	N/A	Drive Weight:	140 lbs.

DEPTH (feet)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: SAMPLE TYPE	SAMPLES: BULK	PENETRATION (blows/foot)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
22		**<u>Bay Point Formation (Qbp):</u>** Reddish-brown, damp to moist, dense to very dense, SILTY SAND (SM), fine to medium grained, micaceous.	SPT		53			SA
24			Cal		50/6"	4.0	112.5	
26		At 25 feet becomes light brown to light reddish-brown, very fine to fine-grained.	SPT		52			
28		Light brown to light reddish-brown, moist, very dense, SILTY SAND-POORLY GRADED SAND (SP), fine to coarse grained.						
30			Cal		50/5"	6.4	99.1	DS pH-Ω
32			SPT		78			CL
34			Cal		50/5"	22.2	97.1	Con SA
36			SPT		75			
38								
40			Cal		50/5"	6.3	95.2	DS

Boring continued on Plate No. 4.

LOG OF TEST BORING NUMBER B-1 (Continued)

Date Excavated:	4/23/2003	Logged by:	TSW
Equipment:	CME55	Project Manager:	CHC
Existing Elevation:	54.0 feet	Depth to Water:	50 feet
Finish Elevation:	N/A	Drive Weight:	140 lbs.

DEPTH (feet)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: SAMPLE TYPE	SAMPLES: BULK	PENETRATION (blows/foot)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
		Bay Point Formation (Qbp): Light brown to reddish-brown, moist, very dense, SILTY SAND-POORLY GRADED SAND (SM-SP), fine to coarse grained, micaceous.						
42			SPT		72			
44								
			Cal		50/3"	6.4	94.2	
46		At 46 feet becomes moist to wet.						
48		Light brown to reddish-brown, wet, very dense, SILTY SAND (SM), fine to medium grained.						
		At 50 feet becomes saturated.	Cal		50/5"	22.4	97.4	Con
50			SPT		50/5"			SA
52		At 51½ feet becomes light grayish-brown, fine to coarse-grained.						
54			Cal		50/5"	15.5	108.5	
56								
58								
60			Cal		50/5"	24.6	96.8	DS

Boring continued on Plate No. 5.

CHRISTIAN WHEELER ENGINEERING

PROPOSED HIGH-RISE DEVELOPMENT
4th Avenue & C Street, San Diego, California

BY:	HF	DATE:	May 2003
JOB NO. :	203.338	PLATE NO.:	4

LOG OF TEST BORING NUMBER B-1 (Continued)

Date Excavated:	4/23/2003	Logged by:	TSW
Equipment:	CME55	Project Manager:	CHC
Existing Elevation:	54.0 feet	Depth to Water:	50 feet
Finish Elevation:	N/A	Drive Weight:	140 lbs.

DEPTH (feet)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: SAMPLE TYPE	SAMPLES: BULK	PENETRATION (blows/foot)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
		<u>Bay Point Formation (Qbp):</u> Light grayish-brown, saturated, very dense, SILTY SAND (SM), fine to coarse-grained, micacous.						
62			SPT		59			
64			Cal		50/5"	20.2	108.5	
66								
68								
70			Cal		50/4"	16.1	112.7	
			SPT		50/5"			
72		Light grayish-brown, saturated, dense to very dense, SILTY SAND-POORLY GRADED SAND (SM-SP), fine to coarse-grained.						
74			Cal		50/5"	27.0	99.9	
76								
78								
80			Cal		50/5"	28.5	94.6	DS

Boring continued on Plate No. 6.

CHRISTIAN WHEELER
ENGINEERING

PROPOSED HIGH-RISE DEVELOPMENT
4th Avenue & C Street, San Diego, California

BY:	HF	DATE:	May 2003
JOB NO. :	203.338	PLATE NO.:	5

LOG OF TEST BORING NUMBER B-1 (Continued)

Date Excavated:	4/23/2003	Logged by:	TSW
Equipment:	CME55	Project Manager:	CHC
Existing Elevation:	54.0 feet	Depth to Water:	50 feet
Finish Elevation:	N/A	Drive Weight:	140 lbs.

DEPTH (feet)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: SAMPLE TYPE	SAMPLES: BULK	PENETRATION (blows/foot)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
		Bay Point Formation (Qbp): Light grayish-brown, saturated, dense to very dense, SILTY SAND-POORLY GRADED SAND (SM-SP), fine to coarse-grained, micaceous.	SPT		34			SA
82								
84			Cal		50/4"	21.4	130.3	
86								
88		Occasional gravels from 88 to 91 feet.						
90			Cal		50/6"	16.5	115.8	
92								
94			Cal		50/4"	19.4	111.5	
96								
98								
100		Boring properly backfilled with 34 cubic feet of grout slurry.	Cal		50/5"	20.1	113.7	

Boring terminated at 100 feet.

CHRISTIAN WHEELER
ENGINEERING

PROPOSED HIGH-RISE DEVELOPMENT			
4th Avenue & C Street, San Diego, California			
BY:	HF	DATE:	May 2003
JOB NO. :	203.338	PLATE NO.:	6

LOG OF TEST BORING NUMBER B-2

Date Excavated:	4/24/2003	Logged by:	TSW
Equipment:	CME55	Project Manager:	CHC
Existing Elevation:	57.0 feet	Depth to Water:	52 feet
Finish Elevation:	N/A	Drive Weight:	140 lbs.

DEPTH (feet)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: SAMPLE TYPE	SAMPLES: BULK	PENETRATION (blows/foot)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
		1" layer of Asphaltic Concrete over 2" layer of Concrete.						
2		**Artificial Fill (Qaf):** Reddish-brown, moist, loose to medium dense, CLAYEY SAND (SC), fine to medium-grained, with gravel.						
4		Grayish-brown, moist, loose to medium dense, SILTY SAND (SM), very fine to fine-grained.	Cal		5	9.1	95.3	
6								
8								
10		**Bay Point Formation (Qbp):** Reddish-brown, moist, medium dense, CLAYEY SAND (SC), fine to medium grained.	Cal		32	13.0	113.0	DS
12								
14		Grayish-brown to reddish-brown, moist, dense, SILTY SAND-POORLY GRADED SAND (SM-SP), fine to medium-grained.	Cal		31	14.0	108.8	
16								
18		Reddish-brown, moist, dense, CLAYEY SAND (SC), fine to	Cal		16/6"	21.6	105.7	
20		coarse-grained, abundant gravels and slight cobbles.	Cal		50/6"			

Boring continued on Plate No. 8.

CHRISTIAN WHEELER
ENGINEERING

PROPOSED HIGH-RISE DEVELOPMENT
4th Avenue & C Street, San Diego, California

BY:	HF	DATE:	May 2003
JOB NO. :	203.338	PLATE NO.:	7

LOG OF TEST BORING NUMBER B-2 (Continued)

Date Excavated:	4/24/2003	Logged by:	TSW
Equipment:	CME55	Project Manager:	CHC
Existing Elevation:	57.0 feet	Depth to Water:	52 feet
Finish Elevation:	N/A	Drive Weight:	140 lbs.

DEPTH (feet)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: SAMPLE TYPE	SAMPLES: BULK	PENETRATION (blows/foot)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
22		**<u>Bay Point Formation (Qbp):</u>** Reddish-brown, moist,dense, CLAYEY SAND (SC), with abundant gravels and slight cobbles.						
24		Reddish-brown to light brown, moist, very dense, SILTY SAND-POORLY GRADED SAND (SM-SP), fine to medium-grained, micaceous.	Cal		50/3"	14.7	101.5	
26								
28								
30		At 30 feet becomes very fine to fine-grained.	Cal		50/5"	13.0	113.3	DS
32		Becomes fine to coarse-grained.						
34			Cal		50/5"	5.5	100.0	Con
36		At 35 feet becomes medium to very coarse-grained.						
38								
40			Cal		50/5"	5.0	106.7	

Boring continued on Plate No. 9.

CHRISTIAN WHEELER ENGINEERING

PROPOSED HIGH-RISE DEVELOPMENT
4th Avenue & C Street, San Diego, California

BY:	HF	DATE:	May 2003
JOB NO. :	203.338	PLATE NO.:	8

LOG OF TEST BORING NUMBER B-2 (Continued)

Date Excavated:	4/24/2003	Logged by:	TSW
Equipment:	CME55	Project Manager:	CHC
Existing Elevation:	57.0 feet	Depth to Water:	52 feet
Finish Elevation:	N/A	Drive Weight:	140 lbs.

DEPTH (feet)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: SAMPLE TYPE	SAMPLES: BULK	PENETRATION (blows/foot)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
		Bay Point Formation (Qbp): Light brown to light reddish-brown, moist, very dense, SILTY SAND-POORLY GRADED SAND (SM-SP), fine to coarse-grained.						
42								
44								
		At 45 feet becomes very fine to medium-grained.	Cal		50/5"	7.7	96.6	
46								
48								
		Becomes wet at 49 feet.						
50			Cal		50/5"	20.0	86.6	Con
52	▽	Becomes saturated at 52 feet.						
54								
		At 55 feet becomes medium to very coarse-grained.	Cal		50/5"	28.4	93.8	
56								
58								
60			Cal		50/5"	24.0	93.7	

Boring continued on Plate No. 10.

CHRISTIAN WHEELER
ENGINEERING

PROPOSED HIGH-RISE DEVELOPMENT
4th Avenue & C Street, San Diego, California

BY:	HF	DATE:	May 2003
JOB NO. :	203.338	PLATE NO.:	9

LOG OF TEST BORING NUMBER B-2 (Continued)

Date Excavated:	4/24/2003	Logged by:	TSW
Equipment:	CME55	Project Manager:	CHC
Existing Elevation:	57.0 feet	Depth to Water:	52 feet
Finish Elevation:	N/A	Drive Weight:	140 lbs.

DEPTH (feet)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: SAMPLE TYPE	SAMPLES: BULK	PENETRATION (blows/foot)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
		Bay Point Formation (Qbp): Light brown to grayish-brown, saturated, very dense, SILTY SAND-POORLY GRADED SAND (SM-SP), fine to coarse-grained.						
62								
64			Cal*		50/4"			
66		Becomes grayish-brown in color.						
68								
70			SPT		50/5"			
		Cobble and gravel layer from 70 to 70½ feet.						
72								
74		Slight gravels from 73 to 85 feet.	SPT		50/3"			
76								
78								
80			SPT		50/5"			

Boring continued on Plate No. 11.

* No sample recovery.

CHRISTIAN WHEELER ENGINEERING

PROPOSED HIGH-RISE DEVELOPMENT
4th Avenue & C Street, San Diego, California

BY:	HF	DATE:	May 2003
JOB NO. :	203.338	PLATE NO.:	10

LOG OF TEST BORING NUMBER B-2 (Continued)

Date Excavated:	4/24/2003	Logged by:	TSW
Equipment:	CME55	Project Manager:	CHC
Existing Elevation:	57.0 feet	Depth to Water:	52 feet
Finish Elevation:	N/A	Drive Weight:	140 lbs.

DEPTH (feet)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: SAMPLE TYPE	SAMPLES: BULK	PENETRATION (blows/foot)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
82		**Bay Point Formation (Qbp):** Grayish-brown, saturated, very dense, SILTY SAND-POORLY GRADED SAND (SM-SP), fine to coarse-grained.						
84		Slight gravels from 73 to 85 feet.						
			SPT		50/5"			
86								
88		Very well cemented lense at 89 feet, ± 1"-2" - thick.						
			SPT		50/4"			
90								
92								
94								
96								
98								
100		Boring properly backfilled with 34 cubic feet of grout slurry.	SPT		50/4"			

Boring terminated at 100 feet.

* No sample recovery.

CHRISTIAN WHEELER ENGINEERING

PROPOSED HIGH-RISE DEVELOPMENT
4th Avenue & C Street, San Diego, California

BY:	HF	DATE:	May 2003
JOB NO. :	203.338	PLATE NO.:	11

LOG OF TEST BORING NUMBER B-3

Date Excavated:	5/21/2003	Logged by:	TSW
Equipment:	CME75	Project Manager:	CHC
Existing Elevation:	53.0 feet	Depth to Water:	54 feet
Finish Elevation:	N/A	Drive Weight:	140 lbs.

DEPTH (feet)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: SAMPLE TYPE	SAMPLES: BULK	PENETRATION (blows/foot)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
		8" layer of Concrete over 8" layer of Base.						
2–4		**Artificial Fill (Qaf):** Medium to dark brown, moist, medium dense, SILTY SAND (SM), fine to medium-grained.						
4–16		**Bay Point Formation (Qbp):** Medium brown, moist, dense to very dense, SILTY SAND (SM), fine to medium grained.	SPT		65			
			SPT		37			
		Becomes light brown, medium to coarse grained, abundant gravels.	SPT		50/5"			
16–20		Light brown, moist, dense, SILTY SAND-POORLY GRADED SAND (SM-SP), fine to medium-grained.						
20			SPT		50			

Boring continued on Plate No. 13.

CHRISTIAN WHEELER ENGINEERING

PROPOSED HIGH-RISE DEVELOPMENT
4th Avenue & C Street, San Diego, California

BY:	HF	DATE:	May 2003
JOB NO. :	203.338	PLATE NO.:	12

LOG OF TEST BORING NUMBER B-3 (Continued)

Date Excavated:	5/21/2003	Logged by:	TSW
Equipment:	CME75	Project Manager:	CHC
Existing Elevation:	53.0 feet	Depth to Water:	54 feet
Finish Elevation:	N/A	Drive Weight:	140 lbs.

DEPTH (feet)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: SAMPLE TYPE	SAMPLES: BULK	PENETRATION (blows/foot)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
22–24		**Bay Point Formation (Qbp):** Light brown, moist, dense to very dense, SILTY SAND-POORLY GRADED SAND (SP), fine to medium-grained. Gravels present from 22 to 23½ feet.						
24–26		Reddish-brown, moist, very dense, SILTY SAND/CLAYEY SAND (SM/SC), medium to coarse-grained.	SPT		50/5"			
26–40		Light brown to reddish-brown, moist, very dense, SILTY SAND-POORLY GRADED SAND (SM-SP), fine to coarse-grained, micaceous. At 30½ feet becomes light brown.	SPT		58			
			SPT		50/5"			
		Gravel layer present from 36 to 37 feet.						
40			SPT		50/5"			

Boring continued on Plate No. 14.

CHRISTIAN WHEELER ENGINEERING

PROPOSED HIGH-RISE DEVELOPMENT
4th Avenue & C Street, San Diego, California

BY:	HF	DATE:	May 2003
JOB NO. :	203.338	PLATE NO.:	13

LOG OF TEST BORING NUMBER B-3 (Continued)

Date Excavated:	5/21/2003	Logged by:	TSW
Equipment:	CME75	Project Manager:	CHC
Existing Elevation:	53.0 feet	Depth to Water:	54 feet
Finish Elevation:	N/A	Drive Weight:	140 lbs.

DEPTH (feet)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: SAMPLE TYPE	SAMPLES: BULK	PENETRATION (blows/foot)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
42		**Bay Point Formation (Qbp):** Light brown and reddish-brown, moist, very dense, SILTY SAND-POORLY GRADED SAND (SM-SP), fine to coarse-grained.						
44			SPT		50/6"			
46		Gravel layer from 45 to 45½ feet.						
48								
50			SPT		50/4"			
		Gravel lenses from 50 to 55 feet with an approximate 4 inch thickness.						
52								
54	▽	At 54 feet becomes saturated.			50/3"			
56								
58								
60			SPT		50/3"			

Boring continued on Plate No. 15.

CHRISTIAN WHEELER
ENGINEERING

PROPOSED HIGH-RISE DEVELOPMENT
4th Avenue & C Street, San Diego, California

BY:	HF	DATE:	May 2003
JOB NO. :	203.338	PLATE NO.:	14

LOG OF TEST BORING NUMBER B-3 (Continued)

Date Excavated:	5/21/2003	Logged by:	TSW
Equipment:	CME75	Project Manager:	CHC
Existing Elevation:	53.0 feet	Depth to Water:	54 feet
Finish Elevation:	N/A	Drive Weight:	140 lbs.

DEPTH (feet)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: SAMPLE TYPE	SAMPLES: BULK	PENETRATION (blows/foot)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
		<u>Bay Point Formation (Qbp):</u> Reddish-brown and light brown, saturated, very dense, SILTY SAND-POORLY GRADED SAND (SM-SP), fine to coarse-grained.						
62								
64			SPT		68			
66								
68								
70			SPT		62			
72								
74			SPT		72			
76								
78								
80		Boring properly backfilled with 27 cubic feet of grout slurry.	SPT		50/4"			

Boring terminated at 80 feet.

CHRISTIAN WHEELER ENGINEERING

PROPOSED HIGH-RISE DEVELOPMENT
4th Avenue & C Street, San Diego, California

BY:	HF	DATE:	May 2003
JOB NO. :	203.338	PLATE NO.:	15

Appendix B

Laboratory Test Results

LABORATORY TEST RESULTS

DIRECT SHEAR TESTS

Sample Location	**Boring B-1 @ 10’**	**Boring B-1 @ 20’**
Description	Undisturbed	Undisturbed
Angle of Friction	35 Degrees	35 Degrees
Apparent Cohesion	650 psf	425 psf
Material	Fill Material	Bay Point Formation

Sample Location	**Boring B-1 @ 30’**	**Boring B-1 @ 40’**
Description	Undisturbed	Undisturbed
Angle of Friction	34 Degrees	34 Degrees
Apparent Cohesion	275 psf	200 psf
Material	Bay Point Formation	Bay Point Formation

Sample Location	**Boring B-1 @ 60’**	**Boring B-1 @ 80’**
Description	Undisturbed	Undisturbed
Angle of Friction	29 Degrees	34 Degrees
Apparent Cohesion	200 psf	250 psf
Material	Bay Point Formation	Bay Point Formation

Sample Location	**Boring B-2 @ 10’**	**Boring B-6 @ 30’**
Description	Undisturbed	Undisturbed
Angle of Friction	36 Degrees	36 Degrees
Apparent Cohesion	150 psf	150 psf
Material	Bay Point Formation	Bay Point Formation

GRAIN SIZE DISTRIBUTION

Sample Number	**B-1 @ 10’-11’**	**B-1 @ 22’-27’**	**B-3 @ 34’-40’**
Sieve Size	Percent Passing	Percent Passing	Percent Passing
3”			
2”	100		
1”	95		
¾”	95		
½”	92		
³/₈”	91		
#4	90	100	100
#8	88	99	99
#16	86	96	93
#30	80	89	77
#50	61	69	57
#100	34	41	26
#200	20	28	10
Classification	(SC)	(SM)	(SM-SP)

LABORATORY TEST RESULTS (Continued)

Sample Number	**B-1 @ 50'-51'**	**B-1 @ 80'**
Sieve Size	Percent Passing	Percent Passing
3"		
2"		
1"		
¾"		
½"		
3/8"		
#4		100
#8		99
#16	100	99
#30	99	94
#50	95	89
#100	74	66
#200	22	11
Classification	(SM)	(SM-SP)

WATER SOLUBLE SULFATE CONTENT TESTS

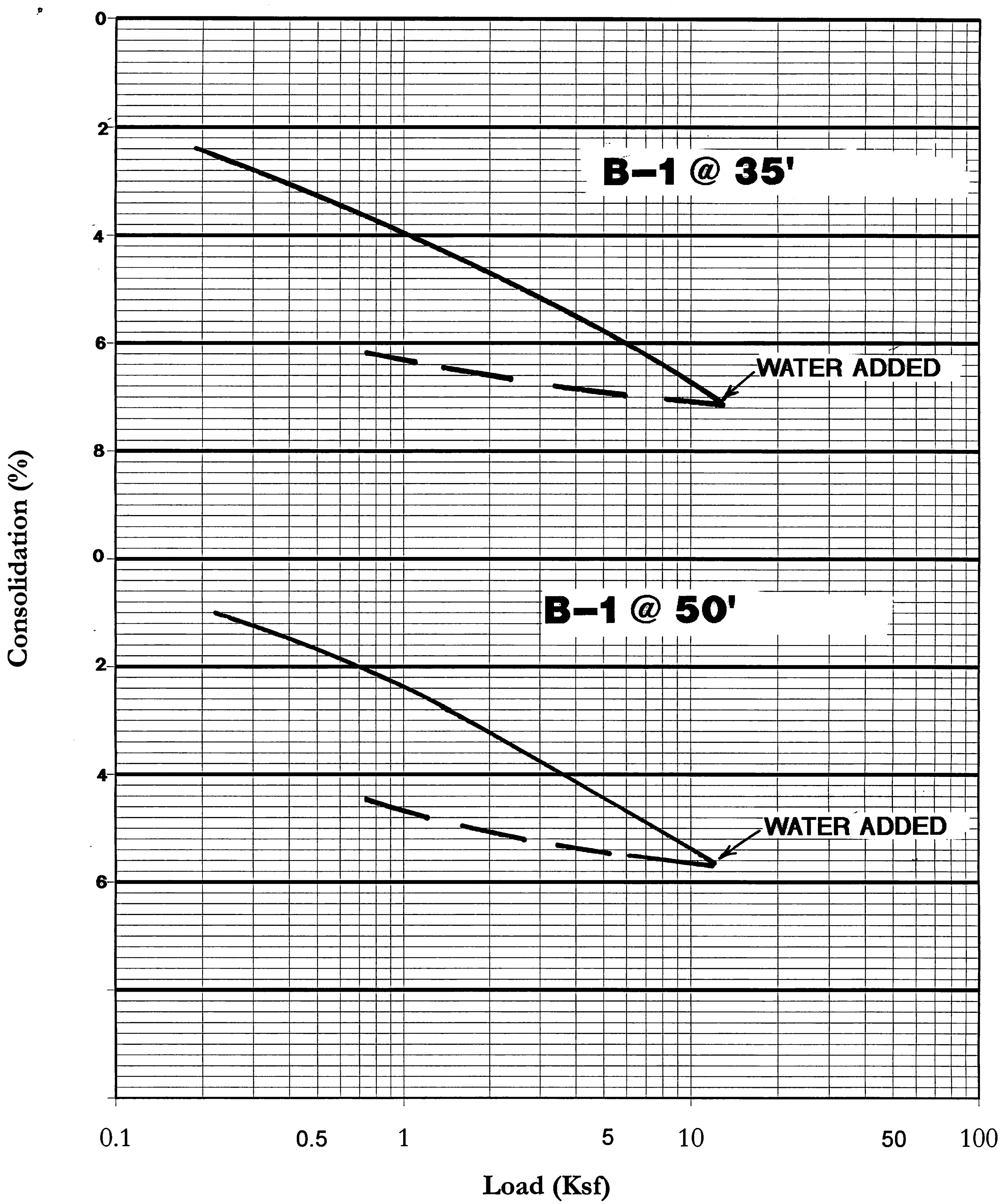
Sample Number	**Boring B-1 @ 35 feet**	**Boring B-1 @ 50 feet**
SO_4, Percent (California Test 417):	0.022 %	0.001 %
Sulfate Exposure	Negligible	Negligible

WATER SOLUBLE CHLORIDE TESTS

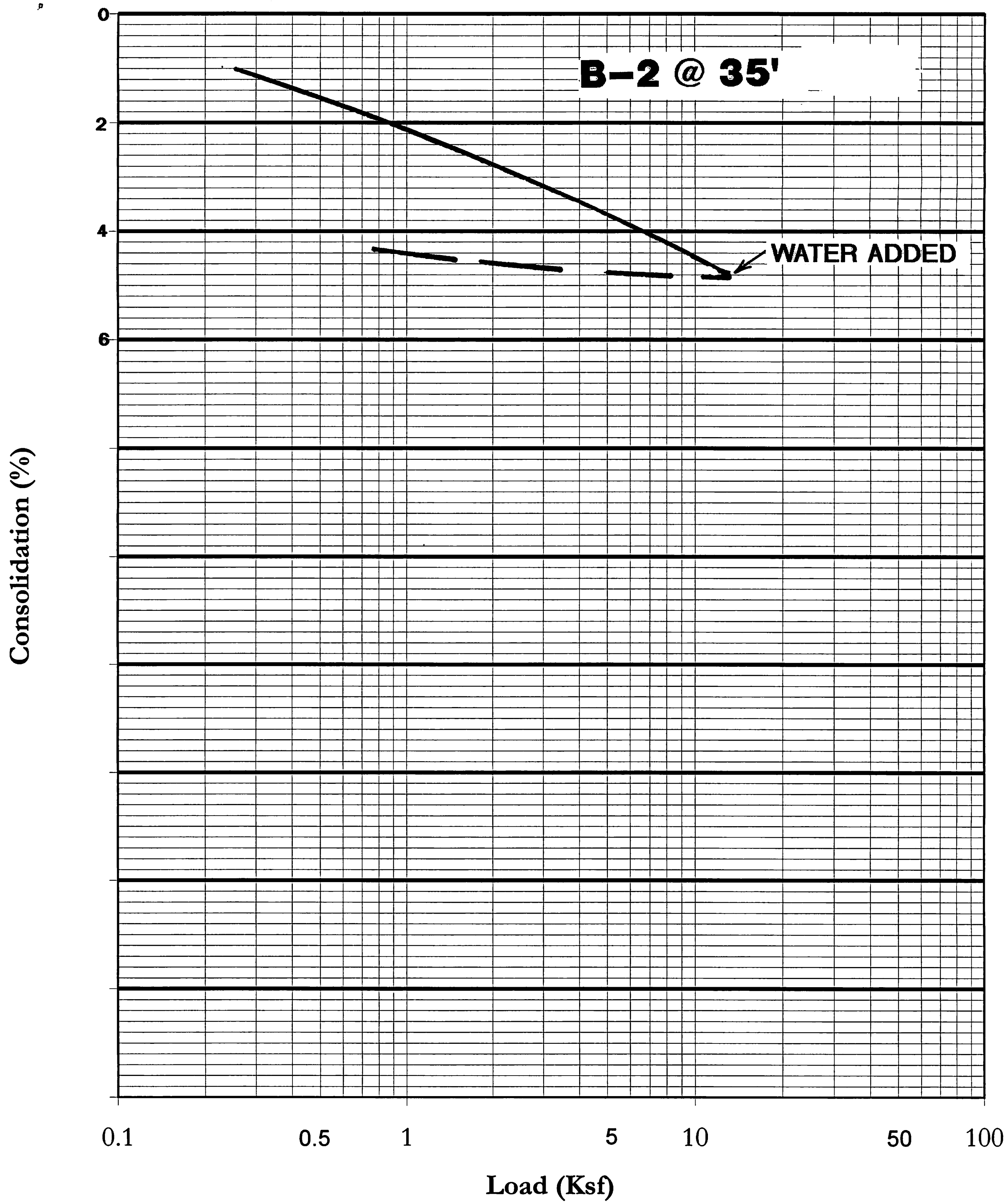
Sample Number	**Boring B-3 @ 35 feet**	**Boring B-3 @ 50 feet**
Chloride concentration	0.020 %	0.011 %

pH AND RESISTIVITY TESTS

Sample Number	**Boring B-3 @ 35 feet**	**Boring B-3 @ 50 feet**
pH	8.3	8.4
Resistivity	802 ohm-cm	5611 ohm-cm



CHRISTIAN WHEELER ENGINEERING	BY: CHC/HC	DATE: May-03
	JOB NUMBER: 203.338	PLATE NUMBER: 18



CHRISTIAN WHEELER ENGINEERING	BY: CHC/HC	DATE: May-03
	JOB NUMBER: 203.338	PLATE NUMBER: 19

Appendix C

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