APPENDIX D

Geotechnical Study

GARCREST
ENGINEERING AND CONSTRUCTION, INC

REPORT OF GEOTECHNICAL INVESTIGATION

PROPOSED BUILDING

6422 Selma Avenue

Los Angeles, California

Prepared for:

NA & ASSOCIATES, Inc.

Lake Forest, California

May 17, 2021

G21-003/1

GARCREST
ENGINEERING AND CONSTRUCTION, INC

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May 17, 2021

Mr. George Ayoub
NA & Associates, Inc.
22672 Lambert St, #606
Lake Forest, CA 92630

Subject: **Report for Geotechnical Investigation**
Proposed Building
6422 Selma Avenue
Los Angeles, California
Project No.: G21-003/1

Dear Mr. Ayoub:

We are pleased to present the results of our geotechnical investigation for the proposed building located at the subject site.

Following the removal of the existing building, temporary shoring and excavation will be required for the construction of the proposed basement level of the building. Temporary shoring recommendations are presented in this report. The proposed building may be supported on a mat foundation established at the bottom of the basement level and at approximately 16 feet below grade. Recommendations for mat foundation, and basement walls below grade are presented in this report. The recommendations presented in this report should be incorporated into the design and construction of the proposed project.

The results of our investigation, our conclusions, and recommendations are presented in this report. The conclusions and recommendations presented in this report are subject to the limitations presented in Section 9 of this report. Part of obtaining a building permit for the project involves the submittal of this report by you or your representative to the appropriate government agencies.

We appreciate the opportunity to be of services to you. Please feel free to contact us should you have any further questions or if we can be of further service.

Respectfully submitted,
GARCREST Engineering and Construction, Inc.

REGISTERED PROFESSIONAL ENGINEER
ARMEN GAPRELIAN
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1.0 - SCOPE

This report provides foundation design recommendations for the proposed building located at 6422 Selma Avenue, in Los Angeles, California. The site location is shown on Plate 1, Site Location Map. The proposed building footprint is shown on Plate 2, Plot Plan.

The site investigation was authorized to evaluate the subsurface conditions at the site, and to provide geotechnical recommendations for the design and construction of the proposed building. Our scope of services was performed in general accordance with our proposal and included performing a field investigation, laboratory testing, and preparing a geotechnical report including the following items and recommendations:

- Vicinity map and plot plan showing approximate field exploration locations;
- Logs of borings;
- Discussion of the scope of work;
- Discussion of field exploration methods;
- Results of laboratory testing;
- Discussion of subsurface conditions, as encountered in our field exploration;
- Discussion of liquefaction potential;
- Results of percolation testing;
- Recommendations for grading and site preparation;
- Recommendations for temporary excavations;
- Recommendations for utility trench backfill;
- Recommendations for seismic near-source factors and discussions of the ground motion study;
- Recommendations for spread foundations, deep foundations, foundation settlement, and lateral resistance;
- Recommendations for support of minor foundations;
- Recommendations for slabs on grade;
- Discussion of potential for creating perched water conditions;

- Discussion of expansive and collapsible soils;
- Recommendations for flexible and rigid pavement.

The assessment of general site environmental conditions for the presence of the contamination in the soils and groundwater was beyond the scope of this investigation.

Our recommendations are based on the results of our field exploration, laboratory testing, and appropriate engineering analyses. Our analyses are based on the ultimate soil strength properties.

2.0 - PROJECT DESCRIPTION

We understand that a new 14 story residential tower building is proposed for the subject site. The proposed building will consist of concrete structure and is anticipated to have a one level subterranean below the proposed building footprint. The proposed building is located within an interior portion of the property with a frontage entry way consisting of a historic portion of building that is to be preserved and converted to the ingress and egress driveways to the proposed development.

At this time, parking is proposed primarily as stacked parking at the street grade level. The subterranean level will primarily consist of equipment rooms and fire department related tank room.

Column loads provided by the project structural engineer range between 800 to 1500 kips.

As part of the proposed development's stormwater mitigation requirements, our scope of work also included performing percolation testing of the subsurface soils to evaluate the potential for stormwater infiltration at the site.

The proposed building location is shown on Plate 2, Plot Plan.

3.0 - FIELD EXPLORATION AND LABORATORY TESTING

The subsurface soil conditions at the site were explored by performing three hollow-stem-auger borings within the site. The borings were performed to depths of between approximately 11½ to 41½ feet below existing grade. Our field representative supervised the fieldwork, logged the borings, and collected relatively undisturbed and disturbed samples for further evaluation and laboratory testing. The borings were performed at the locations indicated on Plate 2, Plot Plan. Details of the field investigation and the Log of Borings are presented in Appendix A, Field Exploration.

Following the completion of the drilling for Boring B-1, the boring was converted into a percolation well. The results of the percolation testing are discussed later in the report. The piping was removed and the borings backfilled at the completion of the testing.

At the time of the original investigation, a basement level was not anticipated in the preliminary conceptual design. Subsequent to our fieldwork however, a basement level was considered and added to the design, primarily brought about by the requirements of the fire department and equipment rooms. Accessibility to the site for drilling and fieldwork was very difficult and complex given the very limited access to the site of the proposed building. The existing buildings were in place and the only portion of the existing buildings where drilling could be performed was located entirely within a group of existing buildings. As such, additional fieldwork for deeper borings within the project site was not easily feasible and coordinated. However, a review of adjoining sites on Selma Avenue and Wilcox Avenue, in very close proximity, and essentially across the street from our site, indicate relatively recent geotechnical investigations performed by Geoconcepts, Inc. dated November 25, 2014 for 6421-6429 Selma Avenue, and January 13, 2016 for 1523-1541 Wilcox Avenue. Both the reports include deep borings performed in each of those two site, providing results relatively consistent with the findings from our borings. Accordingly, we have selected to include the results of the borings from those projects for reference purposes and as representative of the deeper soil conditions at our site as well, especially since the two site effectively bracket our site. We have reviewed, concur with, and accept the results of the field investigation performed by Geoconcepts for those two project site. The location of the borings and log of borings from those reports are presented on Appendix F of this report.

Laboratory testing was performed on selected relatively undisturbed and disturbed samples collected during the investigation to aid in the classification of the soils and to determine pertinent engineering properties used for the development of geotechnical recommendations. The following tests were performed:

- In situ moisture and dry density determination
- Direct shear test
- Consolidation

- Percent Passing No.200 Sieve
- Preliminary corrosivity test

Laboratory testing was performed by AP Engineering and Testing, Inc. of Pomona, California. All testing was performed in accordance with the latest versions of applicable ASTM methods. We have reviewed, approve, and concur with the results of the laboratory testing. Details of the laboratory testing and test results are presented in Appendix B, Laboratory Testing.

4.0 - SITE CONDITIONS

The site is located at 6422 Selma Avenue, in Los Angeles, California. The site is currently occupied by two existing buildings adjoining one another. The building along Selma Avenue consists of a narrow rectangular shaped structure in plan view, extending southward from Selma Avenue, that is considered historic in nature, or at least has a historic façade along Selma Avenue that will require preservation as part of the proposed development. This building is currently occupied and serves as an office building. We understand that as part of the development, a portion of this building, specifically near the north portion, will be preserved due to its historic nature.

The second building is an L-shaped building located south and west of the first building described above. It is currently vacant however the building is located wholly interior to the block of buildings, surrounded entirely by existing adjoining buildings, with very limited accessibility from the property to the west. Both buildings are single story. We understand that the scope of the project will include the demolition of the second building and potential partial demolition of the first building, to allow space for the construction of the proposed development. The overall site is surrounded on the south, east, west, and a portion of the north by existing office buildings. The first historic building described above is bounded by Selma Avenue to the north. The site is relatively flat and various utilities are anticipated to cross the site.

5.0 – SUBSURFACE SOIL CONDITIONS

Fill soils to a depth of approximately 2 to 3 feet below grade were encountered within our borings. Deeper fill soils may be present beyond and between our borings. The onsite fill soils consist of silty sand soils.

The native soils encountered at the site generally consist of medium dense silty and clayey sands and stiff sandy and clayey silts. Insitu moisture contents vary between 2.4 and 14.6 percent and the dry density was 98.0 to 112.0 pounds per cubic foot.

Following the review of our borings as well as the borings performed by Geoconcepts, Inc in their reports from 2014 and 2016 mentioned above, it appears that the deeper onsite soils remain granular in nature, consisting of clayey and silty sands, and increase appreciably in density with depth, especially beyond a depth of 40 to 45 feet below grade.

Groundwater was not encountered in our borings to the depth explored. According to the State (CGS, 1998), historical high groundwater is anticipated to be at a depth of approximately 80 feet below grade.

6.0 - LIQUEFACTION AND SEISMIC SETTLEMENT EVALUATION

Liquefaction is a phenomenon associated with shallow groundwater combined with the presence of loose, fine sands and/or silts within a depth of 50 feet below grade or less. Liquefaction occurs when saturated, loose, fine sands and/or silts are subjected to strong ground shaking resulting from an earthquake event. Liquefaction has the potential to result in the soil temporarily losing part or all of its shear strength. Part of this strength may return sometime after shaking ceases. Liquefaction potential decreases with an increase in grain size, and clay and gravel content. Increasing duration of the ground shaking during a seismic event can also increase the potential for liquefaction.

As previously stated, groundwater was not encountered in our borings to the depth explored. Historical high groundwater at the site is reported to be on the order of 80 feet below grade. The site is not located within a State of California designated liquefaction hazard zone. Due to the relative densities, the nature of the onsite soil materials encountered within our borings, and the depth of historical groundwater, the potential for liquefaction occurrence is considered low.

We have selected the estimated magnitude and acceleration for the site in accordance with the National Earthquake Source Database provided on the USGS website. The ground acceleration used was estimated at two-thirds of the PGA_M for the site. The result of the evaluation is attached, and is summarized as a Magnitude 6.69 and a ground acceleration of 0.66g. Our analysis has been performed using these values.

Seismically induced settlement of the non-saturated soils due to seismic ground shaking has been evaluated based on field data and using the Tokimatsu and Seed (1987) procedures. We estimate the seismically induced dry settlements to be on the order of ½-inch. Differential settlements are estimated to be less than ¼-inch.The results of our analyses are presented in Appendix C, Seismic Settlement Analysis.

The settlements presented herein are in addition to the static settlements presented in this report.

7.0 - CONCLUSIONS AND RECOMMENDATIONS

7.1 - GENERAL

Based on our field exploration, the results of our laboratory testing, and our geotechnical analyses, it is our professional opinion that the proposed project may be constructed and is feasible from a geotechnical perspective. The recommendations presented in this report should be incorporated into the design and construction aspects of the proposed project.

As discussed earlier, fill soils were encountered within our borings to a depth of approximately 2 to 3 feet below existing grade. Deeper fill soils may be present between and beyond our borings. The onsite fill soils are not considered suitable for support of structures. The native soils generally consist of medium dense silty and clayey sands and stiff sandy and clayey silts and increase in densities with depth, specifically below 40 to 45 feet below grade.

As discussed earlier, under the current design, a one level basement is anticipated under the majority of the proposed building footprint. Given the loading associated with the proposed building, and following discussions with the project structural engineer, we recommend that the proposed building be supported on a mat foundation system. At this time, based on the design configuration of the building, we understand the bottom of the mat foundation, approximately 8 feet in thickness, to be established at approximately 16 feet below grade.

Given the nature of the proposed building, and although the onsite soils at the level of the proposed mat foundation consist of the natural onsite soils, to provide a more uniform support for the proposed mat, we recommend that the upper 2 feet below the bottom of the mat be overexcavated to recompacted as properly compacted engineered fill. An allowable bearing capacity of 10,000 pounds per square foot may be used for the soils at least 16 feet below the existing grade where the mat is to be established.

Temporary shoring consisting of soldier pile and lagging will be required for the basement excavation. Recommendations for shoring are presented herein. With the inclusion of the overexcavation below the bottom of the proposed mat foundations, the depth of excavation within may extend to approximately 18 feet below existing grade. Temporary shoring may be designed as a cantilevered system for excavations up to 18 feet in depth. Excavations extending deeper, although not anticipated, will require additional support such as tie back anchors or raker bracing. Excavation should not extent deeper that 18 feet below existing grade unless the shoring design is confirmed by the shoring engineer and deemed acceptable without additional support such as tie back anchors or raker bracing. Additional support recommendations for tie backs or rakers may be provided if required.

Slabs on grade are not anticipated for the proposed building since it will be supported on a mat foundation system. Smaller structure foundations constructed at grade level, flatwork at grade, or other slabs on grade may be supported on properly compacted engineered fill soils. Pavement recommendations are also presented later in this report.

At-grade foundations structurally separate from the proposed building may be supported on shallow spread footings established on at least 3 feet of engineered fill soils, or deepened to the firm and unyielding native soils.

Walls below grade recommendations for the basement level of the proposed building are also presented herein.

7.2 - EARTHWORK

7.2.1 - Site Preparation

As discussed earlier, the proposed building may be supported on a mat foundation established at approximately 16 feet below the existing grade. Following the excavation of the basement and proposed mat foundation, we recommend that to provide a uniform support for the proposed mat, the upper 2 feet below the bottom of the mat be overexcavated and recompacted as properly compacted fill.

For support of smaller structure foundations and slabs on grade near existing grade, as well as pavement support, we recommend the upper fill soils be overexcavated to the firm and unyielding native soils and recompacted as properly compacted engineered fill. Alternately, proposed foundations may be deepened to the firm and unyielding native soils.

Following the overxcavation of the existing soils as recommended above, the exposed subgrade should be observed by a Garcrest representative for unsuitable soils and debris and the excavation deepened as necessary. In areas at grade where deeper fill is encountered, the excavation should be deepened to the firm and unyielding native soils locally. The excavation at grade should extend at least 2 feet laterally, where feasible. Excavation within the basement and mat foundation excavation should extend up to the shoring line but should not extend deeper than 18 feet below grade without review and approval by the shoring designer.

The exposed subgrade should then be scarified to a depth of 6-inches, brought to within 3 percent above the optimum moisture and compacted to a minimum of 90 percent relative compaction as obtainable by ASTM Designation D-1557.

7.2.2 – Excavation Conditions

The borings were performed using a truck mounted hollow stem auger drilling equipment. Drilling was completed using moderate effort through the onsite soils. Conventional earthmoving equipment should be capable of performing the anticipated excavations required. The onsite soils consist of silty and clayey sand and sandy and clayey silt soils.

7.2.3 - Compaction

Engineered fill soils should be placed in loose lifts of no more than 8-inches, brought to a moisture content of within 3 percent above the optimum moisture content, and mechanically compacted using heavy roller and/or vibratory equipment. The fill soils should be compacted to at least 95 percent of maximum dry density.

7.2.4 - Material for Fill

The onsite soils less any debris or organic matter, may be used as fill soils. Import soils should be granular in nature and be relatively non-expansive. Import fill, if required, soils should have a minimum sand equivalent of 30, and an expansion index of less than 35. The import soils should contain sufficient fines to provide a stable subgrade and maintain low to medium permeability. All import materials should be approved by our personnel prior to import onto the site.

7.2.5 - Trench Backfill

All required trench backfill should be mechanically compacted to a minimum of 90 percent relative compaction. Trench backfill should be placed in loose lifts of 8-inches or less, brought to within 3 percent above the optimum moisture content, and compacted with mechanical equipment. Jetting or flooding is not permitted. Some settlement of the backfill may occur and utilities within the trench should be designed to accept some differential settlement.

7.2.6 - Excavation and Temporary Slopes

Excavations deeper than 4 feet should be slopped back at 1:1 (H:V) or be shored for safety. Unshored excavations should not extend below a 1½:1 (H:V) plane drawn downward from the bottom of adjacent existing foundations.

As recommended above, overexcavation is recommended below the proposed mat foundation at the basement level to a depth of 2 feet below the bottom of the proposed mat foundation. Excavation should not extent deeper that 18 feet below existing grade unless the shoring design is confirmed by the shoring engineer and deemed acceptable without additional support such as tie back anchors or raker bracing.

Earthen berms or other methods should be used during wet weather construction in order to prevent runoff water from entering the excavations. All runoff water should be collected and disposed of outside the construction limits.

Excavations should be observed by a representative from our firm so that modifications as a result of varying soil conditions may be facilitated.

All excavations must comply with applicable local, state, and federal safety regulations including the current OSHA Excavation and Trench Safety Standards. Construction site safety is the sole responsibility of the Contractor, who shall also be solely responsible for the means, methods, and sequencing of construction operations. Excavations and temporary slopes should be protected from surficial erosion and the effects of inclement weather by the project contractor. Protective measures such as plastic or jute mesh may be used to protect against the potential for surficial sloughing.

7.3 – TEMPORARY SHORING

Excavations should be shored where there is insufficient room to make a safe sloped excavation. In the case of the current development, shoring will be required for the excavation of the proposed building basement and the mat foundation.

Shoring should be designed to prevent significant lateral or vertical movement of the shored grade and settlement of the existing improvements or foundations. One method of shoring would consist of using timber laggings placed between steel soldier piles placed in drilled holes that are backfilled with concrete. The following recommendations may be used for the structural design of soldier pile and lagging shoring systems.

Some difficulty may be anticipated during the drilling of the soldier piles because of caving and raveling potential anticipated in the sandy soils. Special techniques may be necessary to permit the proper installation of the soldier piles, such as the use of drilling fluid, casing, or reduced drilling speeds. Soldier piles should be drilled and installed alternately.

7.3.1 – Lateral Pressure

For the design of cantilever shoring up to 18 feet in height, an active equivalent fluid pressure of 25 pounds per cubic foot may be used for temporary shoring. Where the surface of the retained earth slopes up away from the shoring, a greater pressure may be required. Deeper excavations will require the use of bracing or tie backs. These recommendations may be provided if required.

The design of shoring should include the surcharge imposed by the footings of adjacent structure within a distance of 15 feet behind the wall. Conservatively, this surcharge pressure may consist of a uniform lateral pressure equivalent to one-third of the existing vertical pressure. Alternately, foundations adjacent to the shoring may be specifically analyzed by the geotechnical engineer once the existing vertical pressures are available. This surcharge may be neglected if foundations are beyond 15 feet behind the wall.

In addition to the recommended earth pressures, the upper 10 feet of shoring adjacent to streets or vehicle traffic should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pounds per square foot surcharge behind the wall due to normal street traffic. If the traffic is kept back at least 10 feet, the surcharge may be neglected. We should be advised if special loading conditions such as cranes or heavy trucks are planned to operate directly adjacent to the shoring as these may impose additional lateral surcharge pressures on the shoring.

7.3.2 – Design of Soldier Pile

For the design of solider piles spaced at least two diameters on centers, the allowable lateral bearing value (passive value) of the subsurface soils at the site may be assumed to be an equivalent fluid pressure of 600 pounds per square foot per foot of depth, up to maximum values of 6,000 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 materials. 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 excavated level should be of sufficient strength to adequately transfer the imposed loads to the surrounding materials.

7.3.3 - Lagging

The lagging may be either timber or gunite. Timber lagging may not remain in place unless it has been treated.

The soldier piles should be designed for the full-anticipated pressure. However, the pressure on the lagging will be less due to arching in the soils. The lagging should be designed for the recommended earth pressure but limited to a maximum value of 400 pounds per square foot.

7.3.4 - Deflection

It is difficult to accurately predict the amount of deflection of a shored embankment. It shall be realized, however, that some deflection will occur. We estimate that this deflection could be on the order of one inch at the top of a 20-foot shored embankment. If greater deflection occurs during construction, additional bracing may be necessary to reduce settlement of the adjacent building. If it is desired to reduce the deflection of the shoring, a greater active pressure or at-rest pressure could be used in the shoring design.

Some deflection of the shored embankments shall be anticipated during the planned excavation. Shoring adjacent to existing structures shall be designed and constructed so as to reduce the potential movement of the adjacent structures. We suggest that proper documentation (including photographs) of existing structures be made, and that the existing structures be surveyed and monitored during construction to record any movements for use in the event of a dispute.

7.4 - FOUNDATIONS

The proposed building may be supported on a mat foundation established at a depth of approximately 16 feet below grade. The proposed mat foundation is anticipated to encompass the proposed building footprint and preliminary discussions with the structural engineer indicate that the proposed mat is anticipated to be approximately 8 feet in thickness. To provide a uniform support for the proposed mat foundation, we recommend that the upper 2 feet below the bottom of the mat be overexcavated to recompacted as properly compacted engineered fill prepared as recommended in the Earthwork section above.

Foundations for smaller at-grade structures, structurally separate from the proposed building may be supported on shallow spread footings established on at least 3 feet of engineered fill soils, or deepened to the firm and unyielding native soils. Foundation systems may not be established in a combination of engineered fill and native, or straddle cut/fill transitions.

Prior to placement of steel reinforcement, the foundation excavations should be cleaned of debris and loose soils and water. The footing excavations should be observed by a Garcrest representative just prior to steel and concrete placement to verify the implementation of the recommendations made herein.

7.4.1 - Bearing Value

An allowable bearing capacity of 10,000 pounds per square foot may be used for the soils at least 16 feet below the existing grade where the mat is to be established.

Proposed structures established at grade and structurally separate from the proposed building may be supported on shallow spread footings at least 24-inches wide and established at least 24-inches below the lowest adjacent grade. For the soils supporting these near grade foundations, an allowable bearing capacity of 3,500 pounds per square foot may be used.

If required, a modulus of subgrade reaction of 120 pounds per cubic inch may also be used for the design of the proposed mat.

A one-third increase may be used for wind and seismic loading conditions.

The recommended bearing value is a net value. The weight of the concrete in the footing may be taken as 50 pounds per cubic foot and the weight of the soil backfill may be neglected when determining the downward loads.

Footings may experience an overall loss in bearing capacity or an increased potential to settle where located above and in close proximity to existing or future utility trenches. Furthermore, stresses imposed by the footings on the utility lines may cause the utilities to crack, collapse and/or lose serviceability. To reduce this risk, footings should extend below a 1:1 plane projected upward from the closest bottom corner of utility trenches.

7.4.2 - Settlement

Loading information was provided to us by the project structural engineer in the form of a pressure diagram imposed by the proposed mat given the loading from the proposed building. The pressure diagram is attached to this report (Plate 3) and indicates pressures imposed on the order of 3,000 to 5,000 pounds per square foot. Based on these anticipated foundation pressures and dimensions as provided, we estimate the total static settlement of the proposed mat foundations to be on the order of 2-inches. Differential settlements are anticipated to be on the order of 1-inch. If pressures or loading conditions should vary from those provided to us, our office should be consulted and additional settlement evaluations and recommendations may be required.

Total and differential settlement for proposed smaller structures at-grade are anticipated to be on the order of ¾-inch and ½-inch, respectively. Static settlement of all foundations is expected to be primarily elastic and should be essentially completed shortly after initial application of structural loads.

The seismically induced settlements estimated earlier are in addition to the static settlements discussed above.

7.4.3 - Lateral Resistance

Resistance to lateral loads may be provided by friction between the soil and the foundation, and by the passive resistance of the soil against the vertical face of the foundation. A coefficient of friction of 0.4 may be used between the foundation and underlying soil. The passive resistance of the soil may be taken as equivalent to the pressure developed by a fluid with a density of 300 pounds per cubic foot. A one-third increase may be used for wind and seismic loading conditions and the passive and sliding values may be combined without reduction.

Sloughing, caving, or overwidening of trench sidewalls during or following excavations may reduce or eliminate the passive resistance of the subgrade soils against foundations. In the event such conditions are encountered, our firm should be notified to review the condition and provide remedial recommendations, if necessary.

7.4.4 - Minor Foundations

Footings for minor structures, such as small retaining walls, that are structurally separate from buildings may be supported on shallow spread footings, established at least 18-inches below the lowest adjacent grade, and be designed for a bearing capacity of 1,500 pounds per square foot. Such footings may be supported on properly compacted engineered fill or undisturbed native soils.

7.5 - SEISMIC CONSIDERATIONS AND GROUND MOTION STUDY

We have provided both code-based seismic design parameters, and as requested by the project structural engineer, site-specific seismic design response spectrum that incorporates probabilistic seismic hazard analysis methods. Our analyses are further described below and figures and tables are presented in Appendix E, Ground Motion Study.

7.5.1 - Code-Based Seismic Design Parameters

The site is located within the seismically active Southern California region. As a minimum, we recommend that the proposed buildings be designed in accordance with the requirements of the latest edition of the California Building Code (CBC).

The structure may be designed to resist earthquake forces following the 2019 edition of California Building Code (CBC), which is based on the 2018 edition of the International Building Code (IBC). The Site Classification, as defined in Section 1613.2.2 of the CBC, may be assumed to be a Site Class D, Stiff Soil Profile.

The mapped maximum considered earthquake spectral response accelerations, Ss and S1, are obtained from Figures 1613.2.1(1) and 1613.2.1(2) from the CBC and are evaluated as 2.116 and 0.745 respectively. Site coefficients Fa and Fv of 1.0 and 1.7 respectively, may be used for the calculation of the spectral response accelerations, however given that S1 is greater than 0.2, based on ASCE 7-16 (Section 11.4.8), a site response analysis may be required. With the above coefficients however, spectral response accelerations $\mathbf{S_{MS}}$ and $\mathbf{S_{M1}}$ of 2.116g and 1.267g and $\mathbf{S_{DS}}$ and $\mathbf{S_{D1}}$ of 1.411g and 0.844g may be used for a Site Class D.

7.5.2 - Probabilistic Seismic Hazard Analysis

We have estimated the Maximum Rotated values of the Risk-Targeted, Maximum Considered Earthquake (MCE_R) design response spectrum based on ASCE 7-16, Sections 11.4, and 21.2 through 21.3. The latest version of the program OpenSHA (version 1.5.2) was used to calculate the probabilistic hazard spectra. The following data briefly summarizes the parameters used in calculating the probabilistic hazards spectrum.

Site Coordinates: 34.099˚ N, -118.33˚ W

V_s30: 350m/s

Fault Model: Mean UCERF 3, Both Branches Averaged

Probability Level: 2% Probability of Exceedance in 50 Years (2,475-year recurrence)

Structural Damping: 5%

Ground Motion Prediction Equations (GMPEs):

- Abrahamson, Silva & Kamal, 2014 NGA West 2
- Boore, Stewart, Seyhan & Atkinson, 2014 NGA West 2
- Campbell & Bozorgnia, 2014 NGA West 2

- Chiou & Youngs, 2014 NGA West 2

The V_s30 value was estimated based on our review of the USGS Global V_s30 Map Viewer (https://usgs.maps.arcgis.com/apps/webappviewer/index.html?id=8ac19bc334f747e486550f32837578e1/). The value estimated from the Map corresponds well with blow counts obtained in our borings, and with blow counts in borings at other sites in the general area.

The 5% damped, mean horizontal acceleration response spectrum was obtained for each of the above GMPEs. The resulting spectra were equally weighted and averaged for the purposes of developing the MCE. The MCE spectrum was converted to a spectrum that is expected to have a 1 percent chance of causing collapse in the subject structure within a 50-year period by employing Method 1 of section 21.2.1.1 in ASCE 7-16. The spectrum was further scaled to estimate the maximum rotated acceleration response spectrum by applying factors from Shahi and Baker 2014.

7.5.3 - Deaggregation of Hazards

According to the project structural engineer, the period of interest for the proposed building is 1.27 seconds.

Using the USGS Unified Hazard Tool (https://earthquake.usgs.gov/hazards/interactive/), we disaggregated the earthquake hazard with respect to earthquake moment magnitude (M), site-source distance (r), and epsilon (ε) for the seismic event having a 2% probability of exceedance in 50 years, and at the PGA period, and at periods of 1, 2 and 3 seconds to evaluate the period range that may affect the proposed building.

For comparison purposes, we performed estimates of the deterministic scenarios that contributed at least 10% to the probabilistic hazard. The faults identified for these deterministic events were the Santa Monica and Hollywood Faults, which contributed approximately 32% and 13% to the probabilistic hazard near the period of interest, respectively. We used the PEER NGA West 2 GMPE Spreadsheets (v5.7_041415) to estimate the deterministic hazard spectra associated with these faults. We plotted the deterministic spectrum associated with the Santa Monica fault, since the intensity magnitude levels were generally greater for this fault.

7.5.4 - Design Response Spectrum

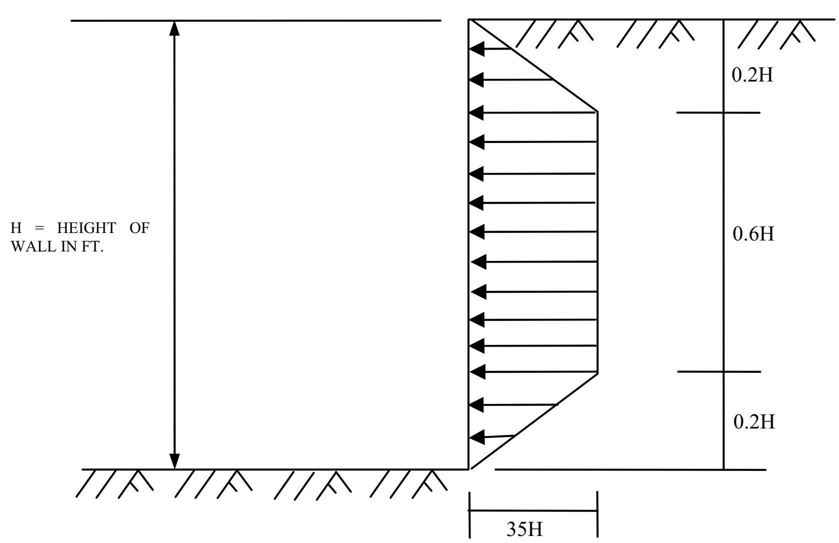
Seismic design criteria for the Risk-Targeted Maximum Considered Earthquake (MCE_R) are based on the ASCE 7-16 procedures using 21.2.1.1 (Method 1) through 21.3. According to ASCE 7-16, the MCE_R is defined as the lesser of the (1) probabilistic response motions developed assuming a 2 percent probability of being exceeded in 50 years (return period of about 2,475 years) and (2) the deterministic spectral response from section 21.2.2. The Design Response Spectrum was developed according to ASCE 7-16 section 21.3. Figure E-1 presents the acceleration response spectra that were used in developing the Design Response Spectrum for this project. The recommended Design Response Spectrum is presented on Figure E-2. Tabulated values for the Design Response Spectrum are presented on Figure E-3.

7.6 - WALLS BELOW GRADE

7.6.1 - Lateral Earth Pressures

For design of cantilevered walls below grade where the surface of the backfill is level, it may be assumed that the retained soils exert a drained lateral earth pressure equal to that developed by a fluid with a unit weight of 35 pounds per cubic foot in the active condition. For basement or walls with restricted movement at the top, an at-rest pressure may be used by the structural engineer. A lateral earth pressure equal to a fluid pressure of 60 pounds per cubic foot may be used for an at-rest condition.

For design of braced walls below grade, a trapezoidal distribution of earth pressure should be used in design as shown in the sketch below.



Where the surface of the backfill is level, a maximum lateral pressure of 35H pounds per square foot should be used in design, where H is the height of the retained earth in feet. The above values assume non-expansive backfill and free-draining conditions.

For seismic purposes, an additional lateral earth pressure may be used where a difference in retained grade greater than 6 feet exists across the site. The pressure distribution may be considered to be an inverted triangle with the maximum pressure at the top and zero on the bottom. The resultant of this force may be assumed to be at 2/3 the height of the wall from the bottom of the wall. For a level backfill condition, a maximum pressure of 20H pounds per square foot may be used, where H is the difference in height of retained grade across the site in feet. This pressure is in addition to the static pressures presented above and may be considered as an ultimate load in design.

The design of walls below grade should include the surcharge imposed by the footings of adjacent structure within a distance of 15 feet behind the wall. Conservatively, this surcharge pressure may consist of a uniform lateral pressure equivalent to one-third of the existing vertical pressure. Alternately, foundations adjacent to walls below grade may be specifically analyzed by the geotechnical engineer once the existing vertical pressures are available. This surcharge may be neglected if foundations are beyond 15 feet behind the wall.

In addition to the recommended earth pressures, the upper 10 feet of walls below grade adjacent to streets or vehicle traffic should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pounds per square foot surcharge behind the wall due to normal street traffic. If the traffic is kept back at least 10 feet, the surcharge may be neglected.

7.6.2 - Backfill

All required backfill should be placed and compacted in accordance with the recommended presented in the Earthwork Section above.

7.6.3 - Drainage

Walls below grade should be designed to resist hydrostatic pressures or be provided with a backdrain to reduce the accumulation of hydrostatic pressures. The recommended lateral earth pressures assume that drainage is provided behind the walls to prevent accumulation of hydrostatic pressures. Backdrains may consist of a 2-foot wide zone of Caltrans Class 2 permeable material located immediately behind the wall, extending to within 1 foot of the ground surface. If Class 2 base is not available, ¾-inch crushed rock with less than 5 percent passing the No. 200 Sieve may be used. The backdrain should be separated from the adjacent soils using a non-woven filter fabric, such as Mirafi 140N or equivalent. Weep holes should be provided or a perforated pipe (Schedule 40 PVC) should be installed at the base of the backdrain and sloped to discharge to a suitable collection facility such as a sump in the basement level that will pump out the collected seepage along with other drainage flow from the building. A slope of at least 2-feet per 100 feet should be used for the drain lines.

As an alternative, proprietary prefabricated drainage systems such as Miradrain 6000 or equivalent, placed behind the wall and connected to a suitable collection and discharge system may also be used.

Prior to construction of the wall backdrain system, the back of walls below grade should be adequately waterproofed to reduce the potential for migration of water through the retaining wall.

7.7 – PERCOLATION TESTING

It is our understanding that in order to control the stormwater flow of the proposed development, stormwater infiltration devices may be considered for the subject site depending on feasibility. Percolation testing was performed at the site to provide subsurface soil percolation potential and to assist in the design of the infiltration devices.

At this time and following discussions with the project civil engineer, it is likely that adequate room may not be available for the use of an infiltration system. As such, alternative treatment methods may likely be pursued, however, the overall stormwater design and associated devise location and setbacks are the responsibility of the project civil engineer and will be presented on their drawings. Our office does not have access to those designs at the time of this report.

Percolation testing was performed in one boring at the site (Boring B-1). The percolation boring was drilled to depth of approximately 10 feet below grade and percolation testing was performed between 5 to 10 feet below existing grade. The percolation testing was performed by drilling an 8-inch diameter boring, installing a 4-inch diameter perforated PVC pipe with openings within the abovementioned depths. Pea gravel was used as backfill around the pipe and water was filled into the pipe to saturate the medium prior to performing the testing. Depth readings were taken every 15 minutes for a period of approximately 1½ hours or until at least three virtually even consecutive readings, the water being replenished subsequent to each reading interval.

The measured percolation rate was reduced based on a factor for bottom of hole percolation only in accordance with the County of Los Angeles guidelines (GS-200.2). Reduction factors of 10.75 was calculated.

The results of the tests are presented in Appendix D, Percolation Testing and summarized in the following table.

Boring/Well No.	Field Percolation Rate (inch/hr)	Reduced Percolation Rate (inch/hr)
B-1	120	11.2

7.7.1 – Infiltration Devices

Based on the results summarized above, some variability may be anticipated in the subsurface soils, due to the test depth as well as localized soil variability or increase in siltier zones within the subsurface materials. It is also likely that the rate of percolation may vary at different locations across the site, however, based on our field investigation, the subsurface soils appear to be relatively uniform and we anticipate this variability to be generally minor. Please refer further to the liquefaction potential discussion below for additional recommendations for stormwater infiltration.

It is our professional opinion that percolation rates as measured in our borings and later adjusted of approximately 11.2 inch/hr may be considered relatively representative of the overall conditions at the site. These rates have not been factored for design purposes but include sidewall reductions for borehole testing.

Groundwater was not encountered within our borings performed at the site to the depth explored. Based on the information obtained from the historic ground water of the Hollywood Quadrangle, historic high groundwater is anticipated to be on the order of 80 feet below existing grade.

Infiltration devices may consist of excavated pits or trenches to depths and size as needed for design capacity. The devices may be backfilled with granular material conforming to the requirements of Class 2 Permeable Base Material as defined by the most current State Specifications or crushed rock material between ¾- to 1-inch open graded material. The use of recycled material is not permitted. The base or rock materials should be surrounded by non-woven filter fabric to reduce the potential of fines migration into the device. Prefabricated devices should also be surrounded by base or rock material wrapped in filter fabric. Adequate overflow capacities should be incorporated into the design of the proposed devices. Infiltration devices considered for the proposed project should be installed a distance of at least 10 feet from proposed or existing foundations

7.7.2 – Additional Discussions

Liquefaction Potential Discussion

As discussed earlier, the site is not located within a State designated liquefaction hazard zone. The depth to historical high groundwater at the site is on the order of approximately 80 feet below grade. Based on the depth to historical groundwater and the nature of the onsite soils, the potential for seismically induced liquefaction settlement is considered low. Regardless however, to reduce the potential for adverse effects from water for the proposed improvements and existing building, we recommend that if infiltration devices are considered for the site, that the devices be kept away from existing or proposed foundations by a distance of at least 15 feet. The design of the proposed devices should include consideration for flexible connections in the event of localized settlement.

Perched Water Conditions

Based on the results of our field investigation, groundwater was not encountered within our borings to the depth explored. Typical infiltration requirements limit the depth of a device such as to maintain a separation of at least 10 feet from groundwater, including historical levels.

The onsite soils are generally sandy in nature and are considered relatively uniform across the site from the ground surface. Given the nature of the material and that substantial layer permeability and material variation with depth were not encountered at the site, it is our opinion that the potential for perched water or mounding is considered low.

Collapsible Soils

Collapsible soils are defined as soils with a potential for a significant decrease in strength and increase in compressibility when wet or saturated (hydro-collapse). Collapsible soils typically consist of relatively sandy soils that exhibit a degree of cementation.

Based on the results of our laboratory testing, the onsite soils do not exhibit a significant collapse potential.

7.8 - FLOOR SLAB SUPPORT

Although floor slabs on grade are not anticipated for the proposed building, smaller structures with floor slabs or slabs on grade at the grade level may be required at the site. Following the preparation of the subgrade as recommended above for at grade structures, concrete floor slabs and walks may be supported on grade. The concrete slab on grade should have a minimum thickness of 5-inches and a structural engineer should design the minimum reinforcement requirements. We recommend minimum reinforcement of No.4 at 18-inches on center for the design of the slab.

Construction activities and exposure to the elements may cause deterioration of the prepared subgrade. We recommend that the exposed subgrade be inspected by our representative and that the subgrade be moisture conditioned and compacted, if necessary, prior to placement of the concrete floor slab.

The proposed floor slab on grade may be designed for a modulus of subgrade reaction of 120 pounds per cubic inch.

To reduce the impact of subsurface moisture and upward moisture migration on vinyl or other moisture sensitive flooring where such floor covering is planned, we recommend that the floor slab be underlain by a vapor retarder and a layer of compacted crushed rock, as is the current industry standard. The rock typically consists of a minimum of 4 inches of crushed rock or aggregate base material compacted to a minimum of 95 percent relative compaction. The vapor retarding membrane should consist of visqueen or poly-vinyl sheeting with a thickness of at least 10 mils. We recommend a low slump concrete with a slump not exceeding 3-inches be used to reduce possible curling of the slab.

It should be noted that these vapor barriers, although currently the industry standard, may not completely inhibit the upward migration of subsurface moisture. Other factors such as the moisture transmission rates to meet for specific floor coverings and interior humidity levels that could induce mold growth may still be beyond the prevention capabilities of the current standard. The effectiveness of the industry standard system is highly dependent on the ultimate use and design of the proposed building, its ventilation, and the indoor moisture levels.

Various factors such as surface grades, the presence of adjacent planters, the quality of the concrete placed, and permeability of the supporting soils will affect future performance. We recommend that the manufacturer for the specific flooring used be contacted for additional consultation specific to their product. The quality of the concrete slab, including the water/cement ratio and curing practices can also affect the ultimate performance of the slab. All concrete placement and curing should be performed in accordance with applicable American Concrete Institute (ACI) methods.

We are not moisture proofing experts and therefore make no guarantees or provide assurances that the use of a capillary break/vapor retarding system will reduce infiltration of subsurface moisture through the floor slab in accordance with any specific flooring material performance specifications.

7.9 - PAVEMENT DESIGN

To provide support for pavement, the subgrade soils should be prepared as recommended in the Earthwork Section of this report. Our pavement recommendations are based on our findings and observations during our field investigation. For design purposes, we have assumed an R-value of 20 for the design of asphalt and Portland cement concrete pavements.

The required pavement thicknesses are based on expected wheel loads and the volume of traffic (TI or Traffic Index). Anticipated traffic indices of 4 through 7 have been used to develop pavement recommendations as presented in the tables below.

Asphalt Concrete Pavement

Traffic Usage	Traffic Index	Asphaltic Concrete (inches)	Base Course (inches)
Automobile Parking Areas	4	3	6
Automobile Traffic	5	3	7
Truck Traffic	6	3½	10
Heavy Truck Traffic	7	4	12

Portland Cement Concrete Pavement

Traffic Usage	Traffic Index	Portland Cement Concrete (inches)	Base Course (inches)
Automobile Parking Areas	4	6½	4
Automobile Traffic	5	6½	4
Truck Traffic	6	7	4
Heavy Truck Traffic	7	7½	4

The above sections have been derived based on the following assumptions.

- The subgrade soils below pavements should be overexcavated to a depth of at least 2 feet below the pavement section, brought to within 3 percent above the optimum moisture content, and compacted to a minimum of 95 percent relative compaction in accordance with the recommendations in the Earthwork section of this report

- The upper 6-inches of the prepared subgrade should be compacted to a minimum of 95 percent relative compaction.

- The aggregate base is brought to within 2 percent of the optimum moisture content and compacted to a minimum of 95 percent relative compaction.

- The subgrade is stable and non-pumping.

- Adequate drainage is provided to reduce the potential of water migration and ponding under the pavement section.

- Planter curbs and gutters extend at least 4-inches into the subgrade level and below the base course to reduce the migration of water into the pavement base course.

- Minimum portland cement concrete compressive strengths of 4,000 pounds per square inch have been used for design.

- Base courses should conform to Caltrans or Standard Specification for Public Works Construction (Green Book) specifications.

- Asphalt pavement materials and placement methods should be in accordance with Caltrans methods.

7.10 - SITE DRAINAGE

Ponding and saturation of the soils in the vicinity of the proposed foundations should be avoided. To reduce this potential, we recommend that positive drainage be provided for the site, in both improvement and landscaping areas, to carry surface water away from the building foundations and slabs on grade and towards appropriate drop inlets or other surface drainage devices. Site grading adjacent to structures and foundations should be sloped away a minimum of 5 percent for a minimum distance of 10 feet away from the face of wall. Impervious surfaces within 10 feet of structures should be sloped a minimum of 2 percent away from the building. These grades should be maintained for the life of the structure. We also recommend that roof runoff be connected to a suitable collection and discharge system to avoid surface discharge and potential saturating the soils near foundations. Poor perimeter and surface drainage may result in water migration beneath building foundations, and may result in potential distress to the proposed improvements.

Planter areas adjacent to the building and foundations should be lined to reduce the infiltration of irrigation water beneath the building. Care should also be taken to maintain a leak-free irrigation system.

7.11 - EXPANSIVE SOILS

Soils that have the potential for volume change (shrinkage and swelling) caused by moisture variations or drying and wetting cycles are classified as expansive soils. Soil moisture variations are typically a result of rainfall, irrigation, poor drainage, roof drains discharging surficially, and exposure to heat and drought conditions. This shrinkage and swelling action can potentially result in distress to pavements, floor slabs-on-grade, and foundations and grade beams.

Based on the results of our field investigation, the site is underlain by relatively granular soils that are anticipated to have very low to negligible expansion potentials.

7.12 - CORROSIVITY

Selected samples of the near surface soils were collected and tested for corrosivity potential. The samples were tested for pH, resistivity, soluble chlorides, and soluble sulfates in general accordance with California Test Methods 643, 422, and 417 respectively. The results of the tests are presented in Appendix B. Preliminary corrosivity testing indicates that the soils have a moderate potential to buried ferrous metals and a mild potential to buried concrete structures. Based on the preliminary corrosivity results, concrete structures should comply with cement type, minimum compressive strength, and minimum water/cement ratio requirements as specified in ACI guidelines 318, Section 4.3.

These tests are only an indicator of the soil corrosivity at the site. A competent corrosion engineer should be consulted to further evaluate the corrosion potential for the onsite soils, suggest additional testing if needed, and to provide further recommendations for corrosion mitigation as applicable to the specific project and improvements.

8.0 - ADDITIONAL SERVICES

We recommend that Garcrest perform a review of the project specifications and plans to evaluate the correct interpretation and incorporation of the recommendations presented in this report into the project design. We will assume no responsibility for incorrect or inadequate interpretation of the recommendations herein should we not be retained for the review of the project plans and specifications.

We also recommend that our firm be retained to perform the geotechnical observation and testing services for the earthwork operations at the site. The services may include the following:

- Observation of soldier pile and lagging installation,
- Observation of cleaning and excavating operations,
- Observation and inspection of the exposed subgrades to receive fill,
- Evaluation of the suitability of import soils,
- Observation of subdrain system,
- Observation and testing of fill placed,
- Observation and probing of foundation excavations prior to placement of concrete.

This service allows us the opportunity to evaluate the applicability of the recommendations presented herein during the construction phase and allows us to make additional recommendations, if necessary. If another firm is retained to provide geotechnical observation services, our professional liability and responsibility would be limited to the extent that we would no longer be the geotechnical engineer of record.

9.0 - LIMITATIONS

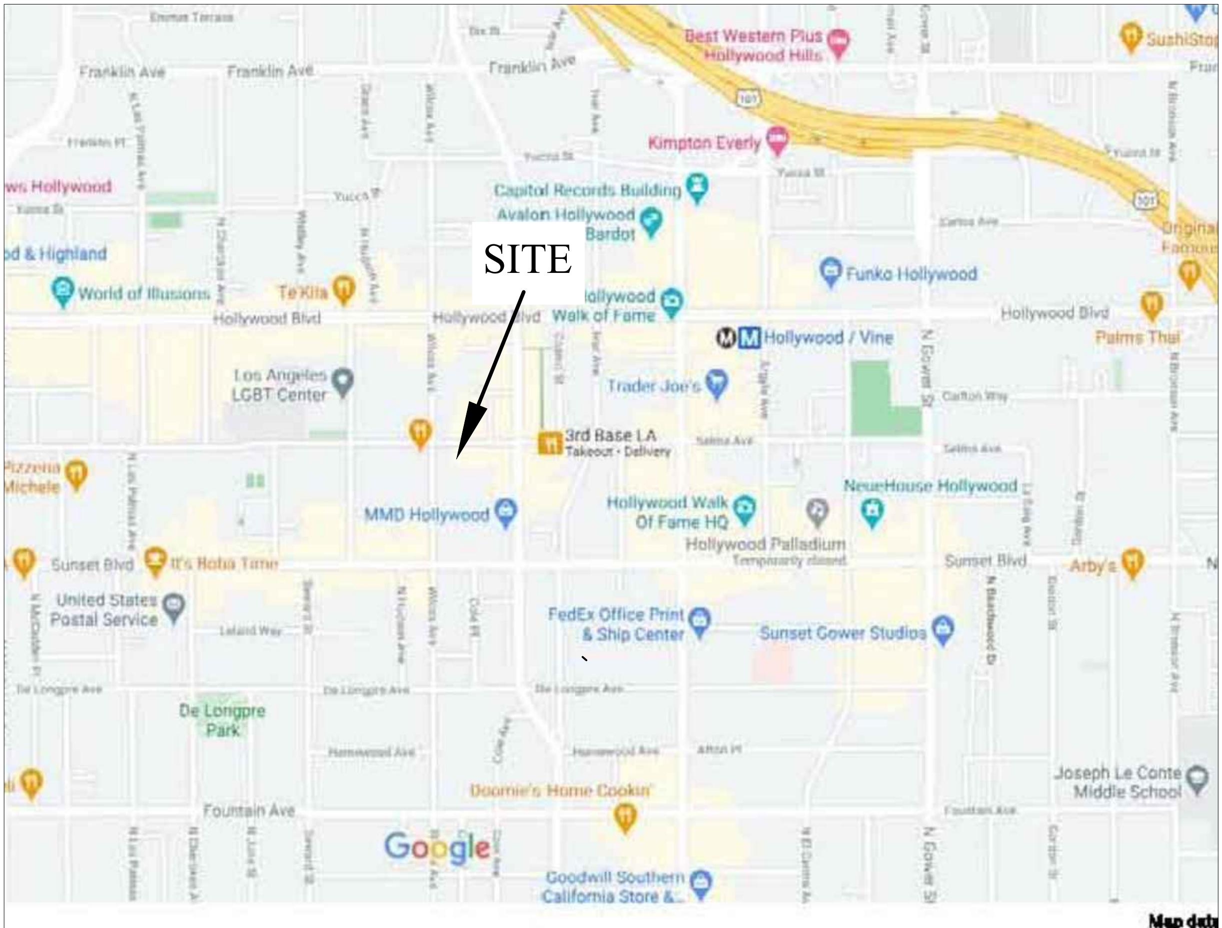
The recommendations presented herein are based on our understanding of the described project information and our interpretation of the data collected during our field investigation. The findings, conclusions, and recommendations presented in this report have been prepared in accordance with the accepted geotechnical practices. Our services have been performed using that degree of care and skill ordinarily exercised, under similar circumstances, by geotechnical consultants practicing in this or similar localities. No other warranty, expressed or implied, is made to the professional advice included in this report.

This report has been prepared exclusively for NA and Associates, Inc. and 6422 Selma Owner, LLC. and their design consultants for the specific application of their project located at 6422 Selma Avenue in Los Angeles, California. This report has not been prepared for other parties and may contain insufficient information for the purpose of other parties and other uses.

The client is responsible for the distribution of this report to all parties associated with the project, including design consultants, contractors, subcontractors. This report may be used to prepare project specifications but is not intended to be used as a specification document.

This report is intended for the sole use of the Client for this specific project within a reasonable time from its issuance. Regulatory and site condition changes may result in the additional information to be incorporated into the report and additional work to be performed by Garcrest prior to the issuance of an update. Non-compliance with these limitations releases Garcrest from any liability resulting from the use of this report by other unauthorized parties

PLATES

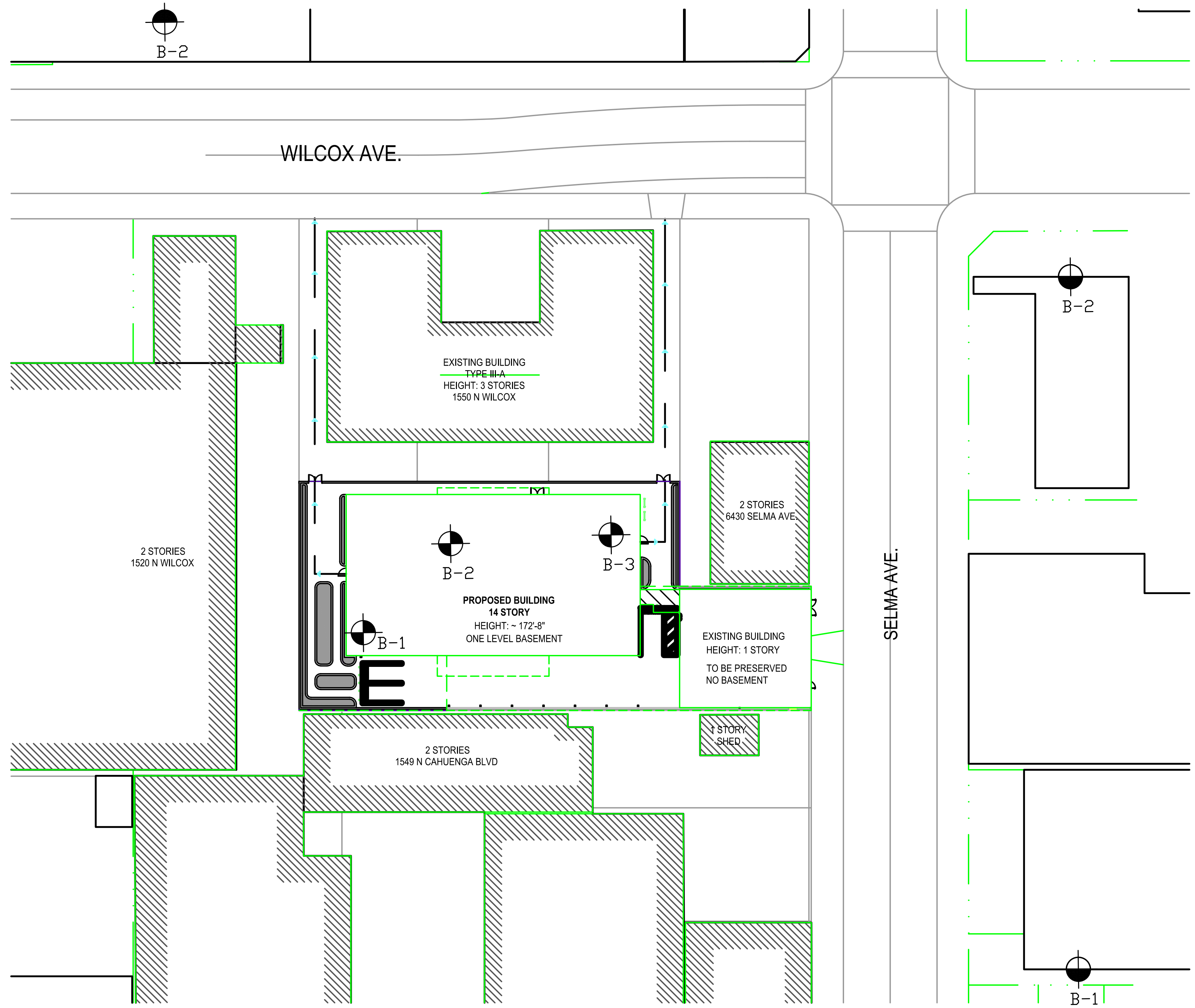


SCALE: Not to Scale

REFERENCE: GoogleMaps (2021)

NORTH

GARCREST ENGINEERING AND CONSTRUCTION, INC.	**SITE LOCATION MAP** Proposed Building 6422 Selma Avenue, Los Angeles, California Project No. G21-003/1	PLATE 1



KEY

- Approximate Boring Locations (Garcrest 2021)
- Approximate Boring Locations (Geoconcepts 2016)
- Approximate Boring Locations (Geoconcepts 2014)

NORTH

SCALE

1 50

REFERENCE: Base plan provided by NA and Associates, Inc.

GARCREST

ENGINEERING AND CONSRUCTION, INC.

PLOT PLAN

Proposed Building
6422 Selma Avenue, Los Angeles, California
Project No. G21-003/1

PLATE

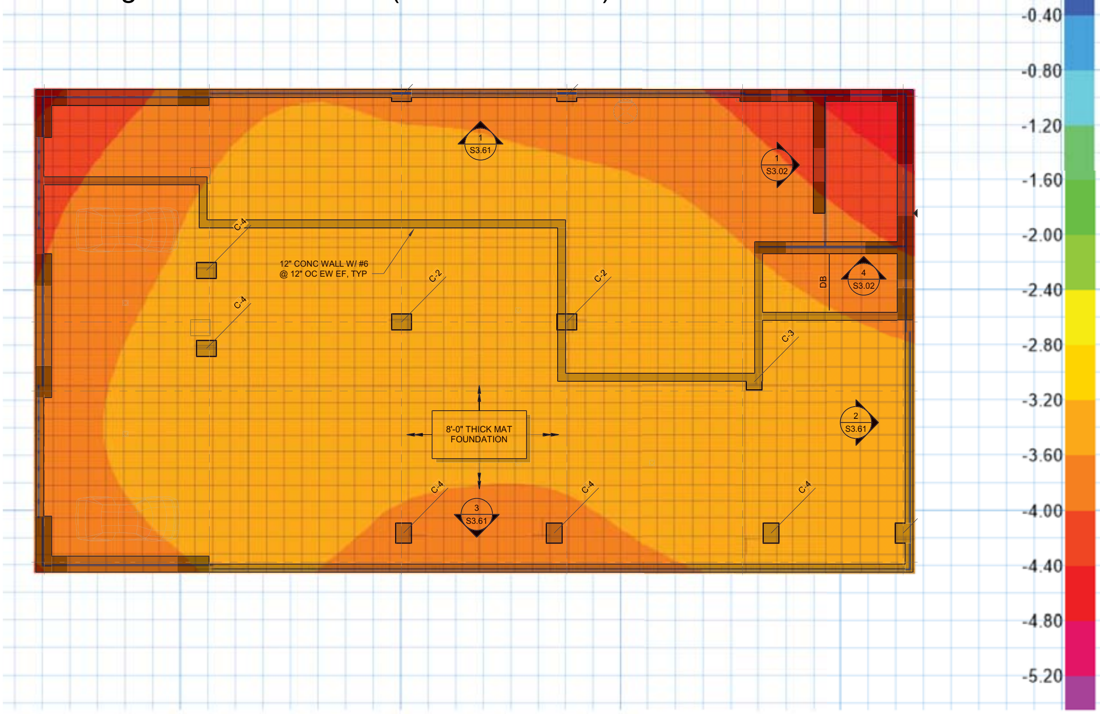
2

kpff 1601 5th Avenue, Suite 1600 Seattle, WA 98101 206 622-5822	project: Selma Residential Tower	by: IF	sheet no.
	location: Hollywood, CA	date: 04-13-2021	
	client: DLR Group		job no.
	Max Bearing Pressure		

Design Values: 8'-0" THICK MAT Subgrade Modulus = 120 psi

GRAVITY:

Max Bearing Pressure: 4.8 ksf (ASD LC 2: D+L)



APPENDIX A – FIELD EXPLORATION

APPENDIX A

FIELD EXPLORATION

The soil conditions at the site were explored by drilling three borings using a track-mounted hollow stem auger type drilling equipment provided by Choice Drilling of Pacoima, California. The borings were performed on January 21, 2021. The borings were advanced to a depth of 41½ feet below the existing grade. The boring locations are shown on Plate 2, Plot Plan. The borings were backfilled using the excavated cuttings and tamped.

The soils encountered were logged by our field engineer and relatively undisturbed and bulk samples were collected for laboratory inspection and testing. The logs of our borings are presented on Figure A-1 through A-3, Log of Borings. The samples were classified in accordance with the Uniform Soil Classification Method (USCS).

A California-type ring sampler was used to collect the relatively undisturbed samples. The sampler was driven a total of 18-inches. The number of blows required to drive the sampler the final 12-inches was recorded on the borings logs. The hammer weight and drop height are also indicated on the boring logs.

Disturbed samples were also collected using a Standard Penetration Test (SPT) sampler. The sampler was driven a total of 18-inches and a number of blows required to drive the final 12-inches were recorder and are presented on the boring logs. The SPT was driven using a 140-pound automatic trip hammer falling a drop height of 30 inches.

Garcrest Engineering & Construction, Inc.
LOG OF BORING

PROJECT NO.:	G21-003/1	**DRILLER:**	Choice Drilling	**LOGGED BY:**	AG
PROJECT NAME:	Selma and Wilcox	**DRILL METHOD:**	8" Hollow Stem Auger	**OPERATOR:**	
LOCATION:	6422 Selma Ave, Los Angeles, CA	**HAMMER:**	140 pound Auto/30 inches	**RIG TYPE:**	CME75 LAR
ELEVATION:				**DATE:**	1/21/2021

BORING NO.: *B-1*

Depth (ft)	Sample Type	Blows/ 6"	Blows/Foot	Sample Number	Graphical Log	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Moisture Content (%)	Dry Density (pcf)	Others
							Concrete Slab - 5" (no rebar only mesh) no Base			
							FILL **SILTY SAND -** brown, very fine, moist			
5	Ring	6 8 8	16	1			**ALLUVIUM** **SILTY SAND -** brown, fine to coarse, moist, medium dense	2.4	104	
10	Ring	7 9 7	16	2			**SANDY SILT -** brown, fine, moist, stiff			
15 20 25							**NOTES:** BORING TERMINATED AT 11½ feet. No Groundwater Encountered Boring convereted to percolation boring. Tested between 5 to 10 feet. Boring backfilled at completion of test with excavated cuttings.			

Legend: --Ring --SPT ---Bulk ---No Recovery ---Water Table

chk: AG 03/22/21

PLATE A-1

Garcrest Engineering & Construction, Inc.
LOG OF BORING

PROJECT NO.:	G21-003/1	**DRILLER:**	Choice Drilling	**LOGGED BY:**	AG
PROJECT NAME:	Selma and Wilcox	**DRILL METHOD:**	8" Hollow Stem Auger	**OPERATOR:**	
LOCATION:	6422 Selma Ave, Los Angeles, CA	**HAMMER:**	140 pound Auto/30 inches	**RIG TYPE:**	CME75 LAR
ELEVATION:				**DATE:**	1/21/2021

BORING NO.: *B-2*

Depth (ft)	SAMPLES: Sample Type	Blows/ 6"	Blows/Foot	Sample Number	Graphical Log	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Moisture Content (%)	Dry Density (pcf)	Others
							Concrete Slab - 5" (no rebar only mesh) no Base			
							FILL **SILTY SAND -** brown. verv fine. moist			CORR
							ALLUVIUM **SILTY SAND -** brown. fine to coarse. moist. medium dense			
5	Ring	5 8 6	14	2				3.0	98	DS
							SANDY SILT - brown. fine. moist. stiff			
10	Ring	5 8 10	18	3				8.6	110	CS
							CLAYEY SAND - brown. fine. moist. medium dense			
15	SPT	3 7 8	15	4			-- Sandy -- 9.7 percent passing No. 200 Sieve.			
							-- trace gravel			
							SANDY SILT - brown, fine, moist, stiff			
20	Ring	6 9 8	17	5				12.8	103	DS
							CLAYEY SAND - brown. fine. moist. medium dense			
25	SPT	4 8 8	16	6			-- 28 percent passing No. 200 Sieve			

Legend: --Ring --SPT ---Bulk ---No Recovery ---Water Table

chk: AG 03/22/21

PLATE A-2a

Garcrest Engineering & Construction, Inc.
LOG OF BORING

PROJECT NO.:	G21-003/1	**DRILLER:**	2R Drilling	**LOGGED BY:**	JD
PROJECT NAME:	Selma and Wilcox	**DRILL METHOD:**	8" Hollow Stem Auger	**OPERATOR:**	
LOCATION:	6422 Selma Ave, Los Angeles, CA	**HAMMER:**	140 pound Auto/30 inches	**RIG TYPE:**	CME55
ELEVATION:				**DATE:**	6/8/2020

BORING NO.: *B-2*

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6"	SAMPLES: Blows/Foot	SAMPLES: Sample Number	Graphical Log	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Moisture Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
30	Ring	7 12 13	25	7				14.6	111	DS CS
33							**CLAYEY SILT -** brown, moist to very moist, stiff to very stiff			
35	SPT	4 8 9	17	8			-- 56 percent passing No. 200 Sieve			
40	Ring	10 22 30	52	9			-- 45 percent passing No. 200 Sieve			

NOTES:
BORING TERMINATED AT 41½ feet.
No Groundwater Encountered
Boring backfilled with cuttings and tamped

Legend: --Ring --SPT ---Bulk ---No Recovery ---Water Table

chk: AG 03/22/21

PLATE A-2b

Garcrest Engineering & Construction, Inc.
LOG OF BORING

PROJECT NO.:	G21-003/1	**DRILLER:**	Choice Drilling	**LOGGED BY:**	AG
PROJECT NAME:	Selma and Wilcox	**DRILL METHOD:**	8" Hollow Stem Auger	**OPERATOR:**	
LOCATION:	6422 Selma Ave, Los Angeles, CA	**HAMMER:**	140 pound Auto/30 inches	**RIG TYPE:**	CME75 LAR
ELEVATION:				**DATE:**	1/21/2021

BORING NO.: *B-3*

Depth (ft)	SAMPLES: Sample Type	Blows/ 6"	Blows/Foot	Sample Number	Graphical Log	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Moisture Content (%)	Dry Density (pcf)	Others
							Concrete Slab - 5" (no rebar only mesh) no Base			
							FILL **SILTY SAND -** brown. verv fine. moist			
							ALLUVIUM **SILTY SAND -** brown. fine to coarse. moist. medium dense			
5		5 7 8	15	1			-- 20 percent passing No. 200 Sieve	6.2	109	
10		3 8 8	16	2			**CLAYEY SAND** - brown, fine, moist, medium dense -- 31 percent passing No. 200 Sieve			
15		10 13 20	33	3			-- dense	4.8	112	
20		2 7 8	15	4			**CLAYEY SILT -** brown, moist to very moist, stiff	14.3		
25		9 16 23	39	5			**NOTES:** BORING TERMINATED AT 21½ feet. No Groundwater Encountered Boring backfilled with cuttings and tamped -- very stiff			
		5 10 10	20	6			**SILTY SAND** - brown, fine to coarse, some gravel,moist, medium dense			

Legend: --Ring --SPT ---Bulk ---No Recovery ---Water Table

chk: AG 03/22/21

PLATE A-3

APPENDIX B – LABORATORY TESTING

APPENDIX B

LABORATORY TESTS

Laboratory tests were performed on selected samples to aid in the classification of the soils encountered and to determine engineering properties for the onsite soils. The laboratory tests were performed by AP Engineering and Testing, Inc. of Pomona, California.

Field moisture content and dry densities of the soils were determined by performing tests on relatively undisturbed samples collected. The results are presented on the boring logs and Figure B-1, Moisture and Density Test Results.

Direct Shear tests were performed on selected samples to evaluate the strength parameters of the soils. The tests were conducted on samples after soaking to near-saturated moisture content at various surcharges. The tests were performed in general accordance with ASTM Standard Test Method D-3080. The tests were performed at a strain rate of 0.005 inches per minute under soaked conditions. The results of the tests are shown on Figure B-2, Direct Shear Test Results.

A Consolidation test was performed on a selected sample to evaluate the compressibility of the soils. The test was conducted in general accordance with ASTM Standard Test Method D-2435. Water was added to the sample to illustrate the effect of moisture on compressibility. The results are presented on Figure B-3, Consolidation Curve.

The percent passing the No. 200 sieve of selected samples was performed by wash sieving in accordance with ASTM Standard Test Method D-1140. The results are presented on Figure B-4, Percent Passing No. 200 Sieve.

A series of corrosivity tests were performed on selected samples of the soils encountered at the site. The tests included pH, resistivity, soluble chlorides and soluble sulfates. The tests were performed in general accordance with California Test Methods 643, 422, and 417 respectively. The results are presented on Figure B-5, Corrosion Test Results

AP Engineering & Testing, Inc.

A Certified DBE/MBE/SBE Company

February 1, 2021

To: Garcrest Engineering and Construction, Inc.
126 S. Jackson Street, Suite 300
Glendale, California 91205

Attention: Armen Gaprelian, P.E., G.E.

Subject: Laboratory Test Report
Project Name: Selma & Wilcox
Project No.: G21-003/1

Dear Armen,

This letter is to certify that AP Engineering and Testing has performed laboratory soil tests for the subject project. The laboratory testing program as requested by you consisted of:

- 3 Moisture Content & Density (ASTM D 2216 & D 2937)
- 1 Moisture Content Only (ASTM D 2216)
- 6 Percent Passing #200 Sieve (ASTM D 1140)
- 1 Corrosion Suite (CTM 417, 422 & 643)
- 3 Direct Shear (ASTM D 3080)
- 2 Consolidation (ASTM D 2435)

All tests were performed in accordance with the applicable standards as indicated above under the supervision of a registered geotechnical engineer. Attached please find the test results.

We appreciate the opportunity to be of service to you. Should you have any questions, please call our office at your convenience.

Respectfully submitted,

AP Engineering and Testing, Inc.
Certificate No. 10130

Apichart Phukunhaphan, P.E., G.E.
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
APICHART PHUKUNHAPHAN
No. 002156
Exp. 12/31/21
GEOTECHNICAL
STATE OF CALIFORNIA

Distribution: 1 Addressee

Attachments: Laboratory Test Results

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MOISTURE AND DENSITY TEST RESULTS

ASTM D2216 and ASTM D7263 (Method B)

Client: Garcrest Engineering
Project Name: Selma & Wilcox
Project No.: G21-003/1

AP Lab No.: 21-0145
Test Date: 01/26/21

Boring No.	Sample No.	Sample Depth (ft.)	Moisture Content (%)	Dry Density (pcf)
B1	-	5	2.4	104.1
B3	-	5	6.2	108.9
B3	-	15	4.8	111.6
B3	-	20	14.3	NA

GARCREST
ENGINEERING AND CONSTRUCTION, INC.

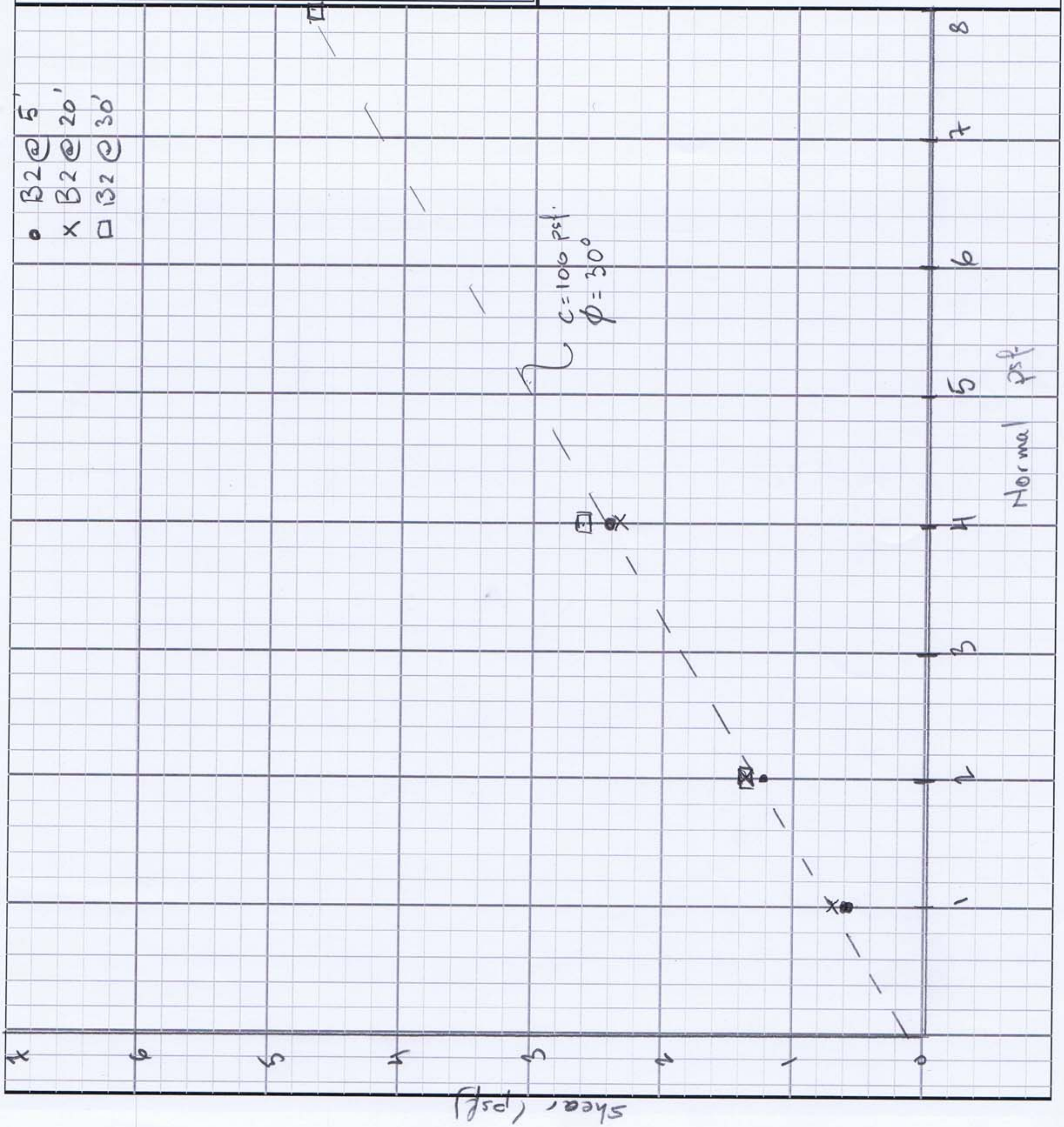
JOB NO: G21-003/1 SHEET ___ OF ___

JOB NAME: G422 Selma

BY: AG DATE: 3/24/21

CHECKED BY: ______ DATE: ______

SUBJECT: SHEAR SUMMARY



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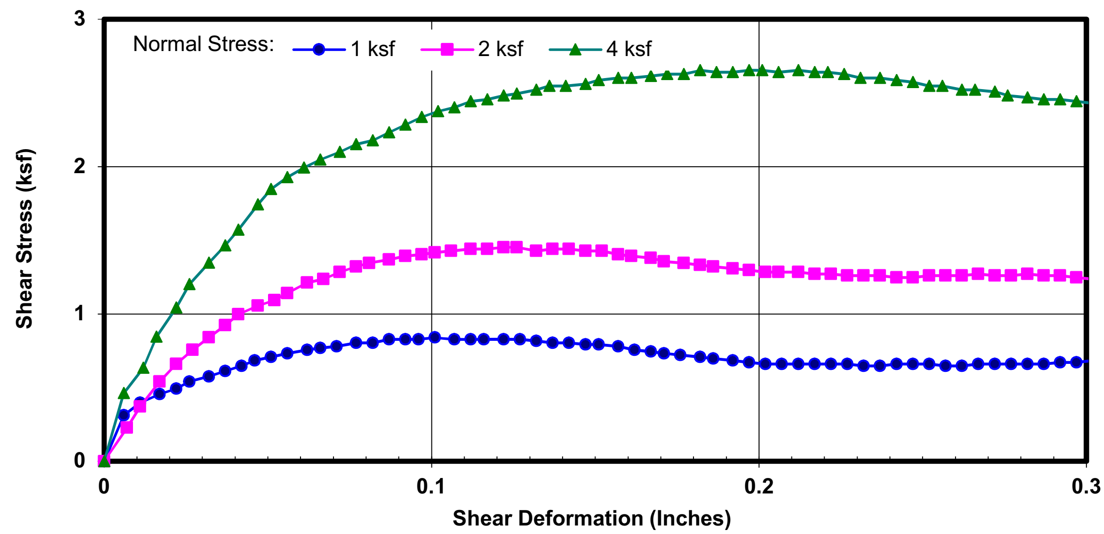
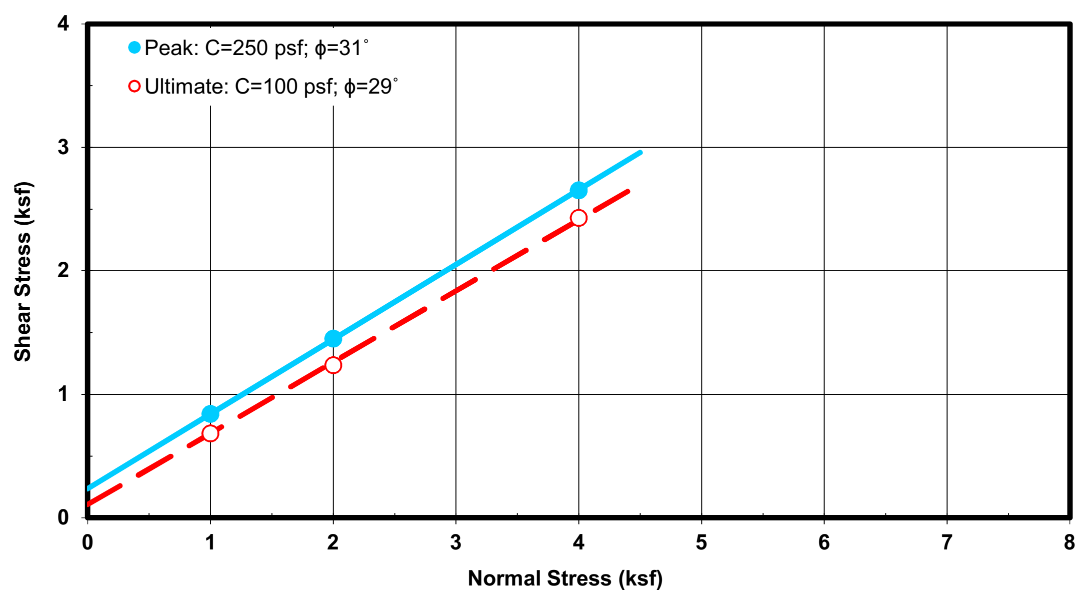
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Client:	Garcrest Engineering	**Tested By:** NG	**Date:** 01/27/21
Project Name:	Selma & Wilcox	**Computed By:** NR	**Date:** 01/29/21
Project No.:	G21-003/1	**Checked by:** AP	**Date:** 02/01/21
Boring No.:	B2		
Sample No.:	-	**Depth (ft):** 5	
Sample Type:	Mod. Cal.		
Soil Description:	Silty Sand		
Test Condition:	Inundated	**Shear Type:** Regular	

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
101.0	98.0	3.0	23.8	11	90	1	0.840	0.684
						2	1.452	1.236
						4	2.653	2.429





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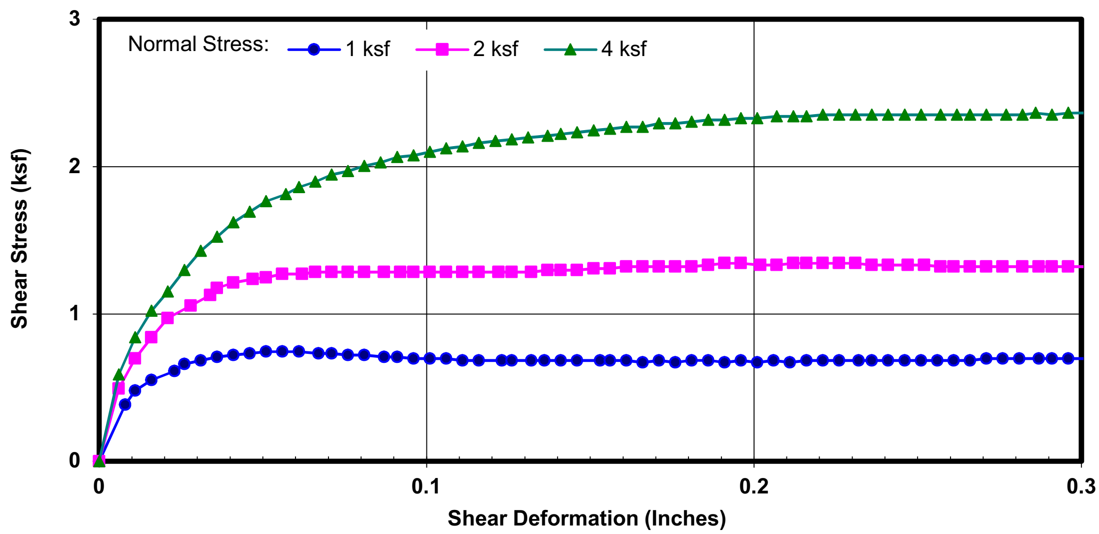
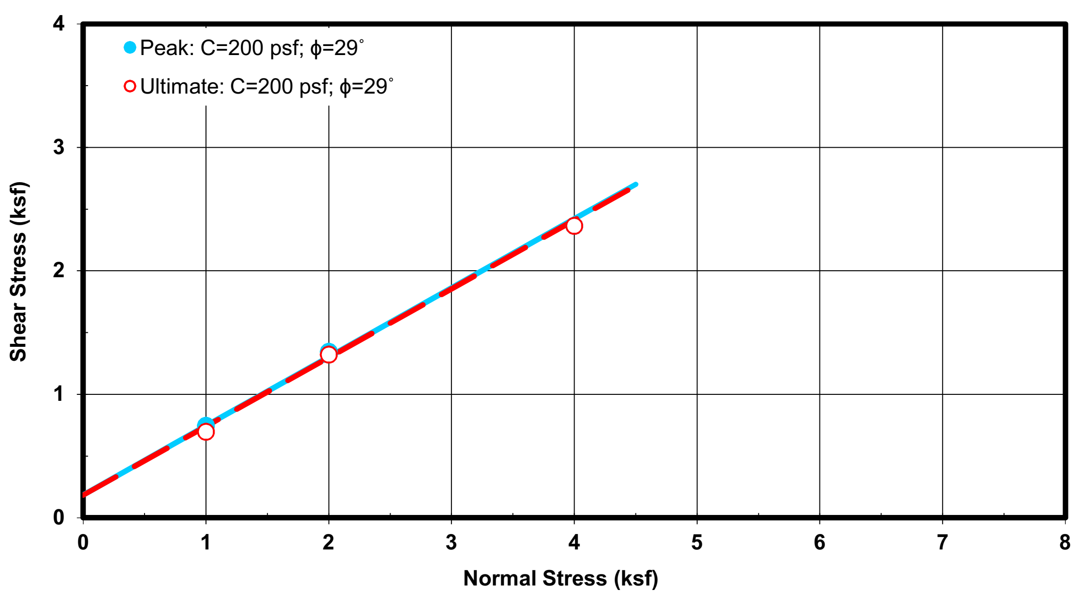
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Client: Garcrest Engineering
Project Name: Selma & Wilcox
Project No.: G21-003/1
Boring No.: B2
Sample No.: - **Depth (ft):** 20
Sample Type: Mod. Cal.
Soil Description: Sandy Silt
Test Condition: Inundated **Shear Type:** Regular

Tested By: ST **Date:** 01/28/21
Computed By: NR **Date:** 02/01/21
Checked by: AP **Date:** 02/01/21

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
116.5	103.3	12.8	21.4	55	91	1	0.744	0.696
						2	1.344	1.320
						4	2.364	2.364





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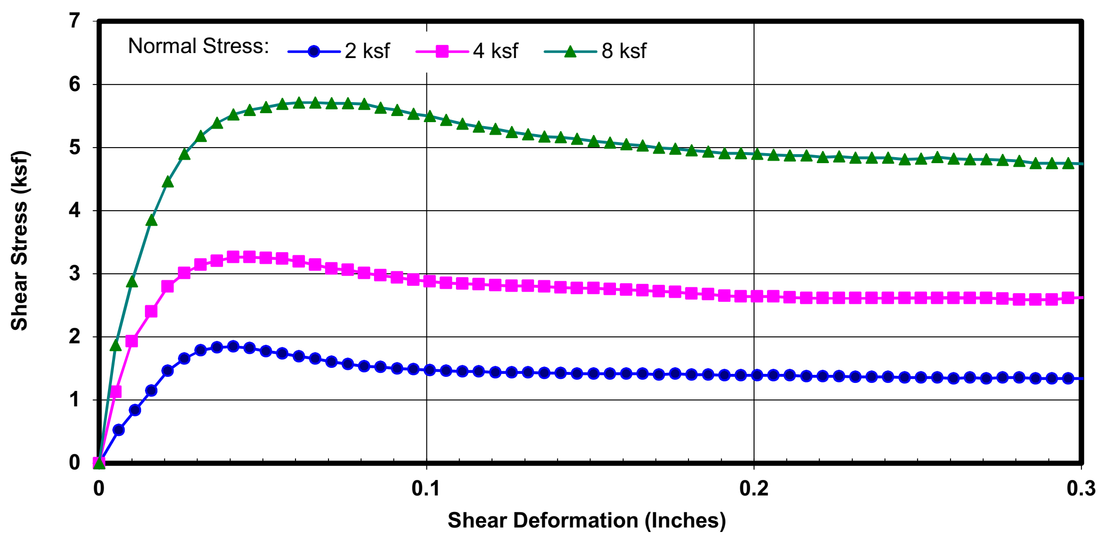
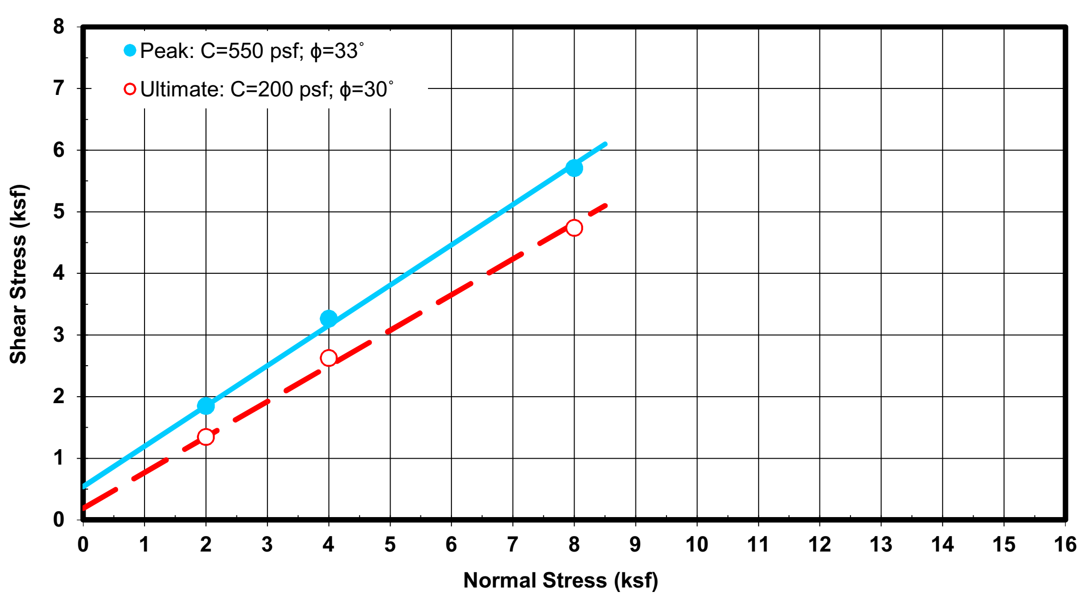
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Client:	Garcrest Engineering	**Tested By:**	NG	**Date:**	01/27/21
Project Name:	Selma & Wilcox	**Computed By:**	NR	**Date:**	01/29/21
Project No.:	G21-003/1	**Checked by:**	AP	**Date:**	02/01/21
Boring No.:	B2				
Sample No.:	- **Depth (ft):** 30				
Sample Type:	Mod. Cal.				
Soil Description:	Clayey Sand				
Test Condition:	Inundated **Shear Type:** Regular				

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
127.4	111.2	14.6	17.1	76	90	2	1.848	1.344
						4	3.264	2.628
						8	5.712	4.740





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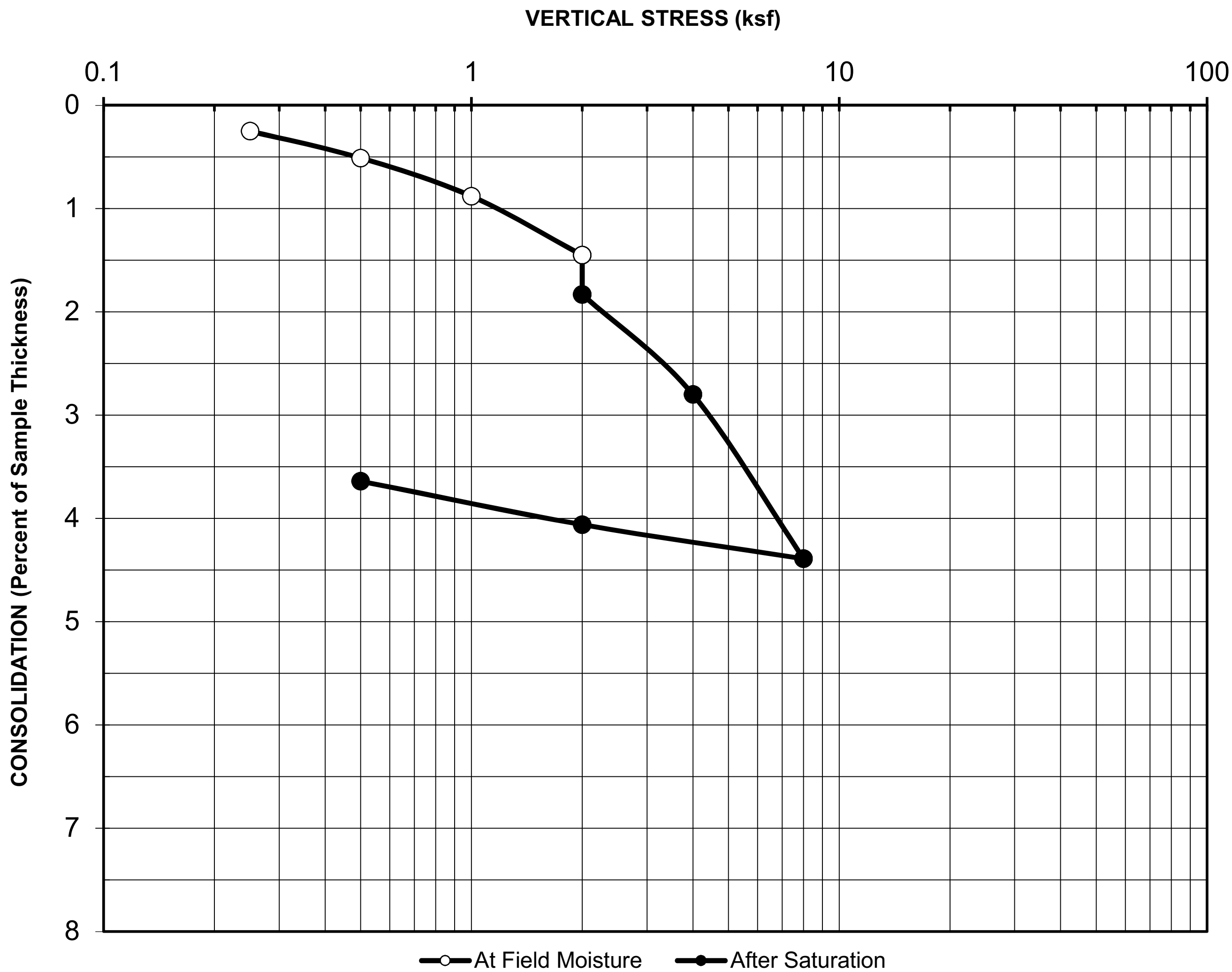

Boring No. :	B2	Initial Dry Unit Weight (pcf):	109.7
Sample No.:	-	Initial Moisture Content (%):	8.6
Depth (feet):	10	Final Moisture Content (%):	16.1
Sample Type:	Mod Cal	Assumed Specific Gravity:	2.7
Soil Description:	Clayey Sand	Initial Void Ratio:	0.54
Remarks:	Collapse= 0.38% upon inundation		

CONSOLIDATION CURVE
ASTM D 2435

Project Name: Selma & Wilcox
Project No.: G21-003/1
Date: 1/26/2021
AP No: 21-0145 **Sheet No:** 1

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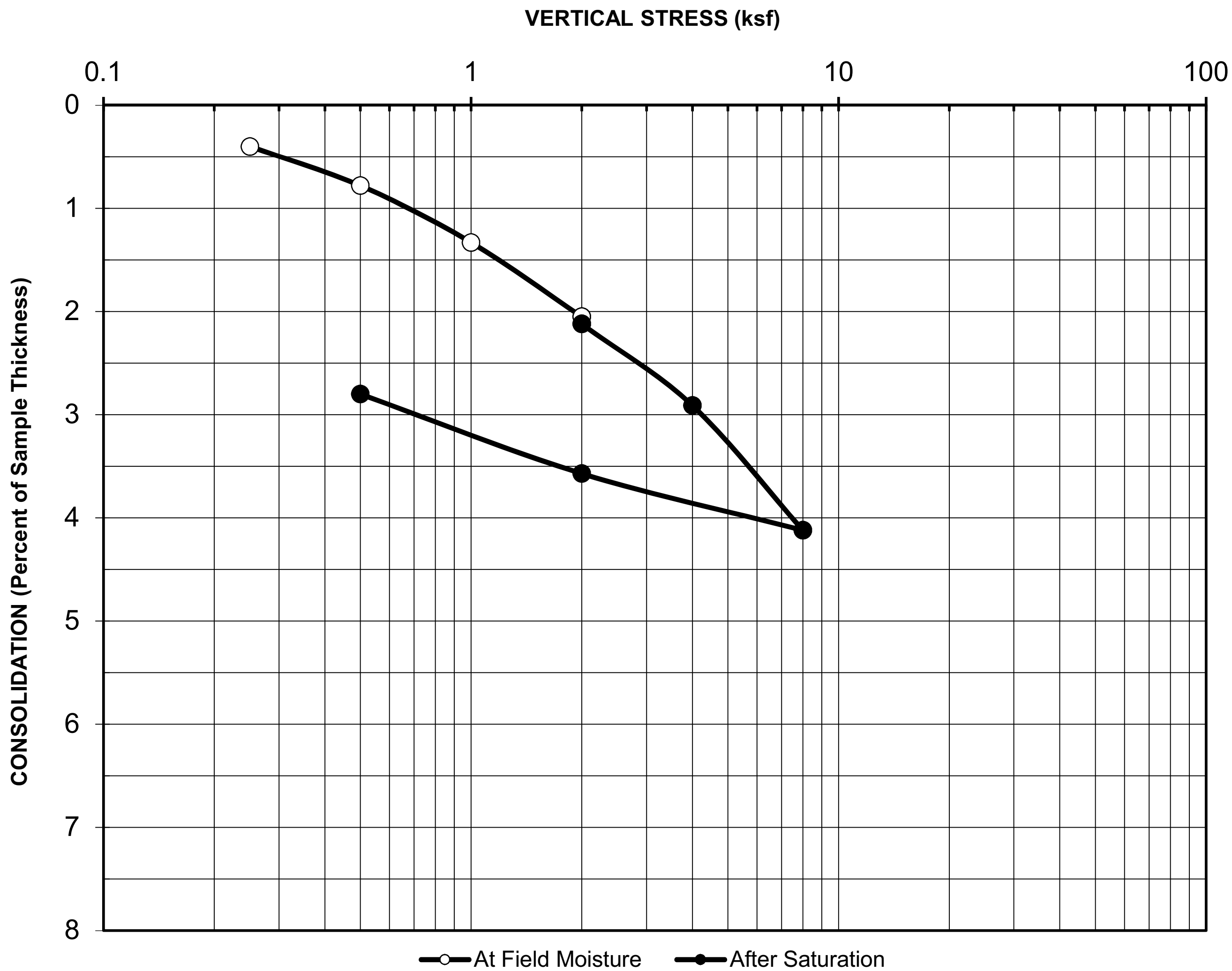

Boring No. :	B2	Initial Dry Unit Weight (pcf):	106.5
Sample No.:	-	Initial Moisture Content (%):	14.6
Depth (feet):	30	Final Moisture Content (%):	18.6
Sample Type:	Mod Cal	Assumed Specific Gravity:	2.7
Soil Description:	Clayey Sand	Initial Void Ratio:	0.58
Remarks:	Collapse= 0.07% upon inundation		

CONSOLIDATION CURVE
ASTM D 2435

Project Name:	Selma & Wilcox
Project No.:	G21-003/1
Date:	1/26/2021
AP No:	21-0145 **Sheet No:** 1

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PERCENT PASSING NO. 200 SIEVE

ASTM D1140

Client: Garcrest Engineering
AP Lab No.: 21-0145
Project Name: Selma & Wilcox
Test Date: 01/26/21
Project Number: G21-003/1

Boring No.	Sample No.	Depth (ft)	Percent Fines (%)
B2	-	15	9.7
B2	-	25	28.1
B2	-	35	56.2
B2	-	40	44.6
B3	-	5	20.0
B3	-	10	30.8

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CORROSION TEST RESULTS

Client Name: Garcrest Engineering
Project Name: Selma & Wilcox
Project No.: G21-003/1

AP Job No.: 21-0145
Date: 01/27/21

Boring No.	Sample No.	Depth (feet)	Soil Description	Minimum Resistivity (ohm-cm)	pH	Sulfate Content (ppm)	Chloride Content (ppm)
B2	-	1-4	Silty Sand	5,107	7.7	124	96

NOTES:
Resistivity Test and pH: California Test Method 643
Sulfate Content : California Test Method 417
Chloride Content : California Test Method 422
ND = Not Detectable
NA = Not Sufficient Sample
NR = Not Requested

APPENDIX C– SEISMIC SETTLEMENT ANALYSIS

Garcrest Engineering and Construction, Inc.

Seismically Induced Dry Settlement (Tokimatsu Seed, 1987/Pradel, 1998)

Project Name	Selma
Project No.	G21-003/1
Location	6422 Selma Los Angeles, CA
Boring	B-2
Design mag	6.69
Design Accel	0.66
Hammer Energy Ce	1.25
Borehole diameter Cb	1.15
Sampling Method Cs	1.25

Depth to top (feet)	Depth to bottom (feet)	Unit Weight (pcf)	SPT	Fine Content (%)		Layer Thickness (feet)	Effective Depth (feet)	Total Stress (psf)	Overburden Correction factor Cn	Rod Length Correction factor Cr	N160	Stress reduction Coefficient rd	K2max	Sig m (psf)	tau (psf)	Gmax (psf)	geff(Geff/Gmax)	geff (%)	e15 (%)	Nc	enc (%)	Delta S (inches)
15	18	110	25	10		3	16.5	1815	1.07	0.85	40.82	0.965	68.87	1210.0	751.570	2395475	0.0003137	0.0009	0.0004	8.562	0.00028	0.020
18	24	110	15	50		6	21	2310	0.96	0.95	24.58	0.954	58.15	1540.0	945.606	2282044	0.0004144	0.0010	0.0008	8.562	0.00061	0.087
24	28	120	16	28		4	26	3120	0.82	0.95	22.47	0.938	56.43	2080.0	1255.634	2573811	0.0004879	0.0015	0.0013	8.562	0.00101	0.097
28	34	120	16	28		6	31	3720	0.74	1	21.38	0.915	55.51	2480.0	1460.603	2764447	0.0005284	0.0016	0.0015	8.562	0.00115	0.165
34	37	120	17	56		3	35.5	4260	0.68	1	20.91	0.887	55.10	2840.0	1620.881	2936556	0.0005520	0.0016	0.0015	8.562	0.00118	0.085
37	40	120	17	45		3	38.5	4620	0.65	1	19.86	0.864	54.16	3080.0	1711.759	3005976	0.0005695	0.0016	0.0016	8.562	0.00125	0.090
																					Total	0.545

APPENDIX D– PERCOLATION TESTING

Well B-1

Diameter (in) =	8	Depth of Hole (ft) =	10	Effi. =	1
Length of Pipe (ft) =	10	casing diameter (in) =	3	Perc. Zone	5 ft to 10 ft

	Time	Time Difference (min)	Depth to Top of Water (ft)	Change in Depth (ft)	Change in Depth (in)	Depth of water above bott. of screen (ft)	Avg. Head (ft)	Percolation Rate "R" (min/in.)	Percolation Rate "R" (in/min)
1	9:30		5.00	-		5.0			
	9:45	15	7.80	2.80	33.6	2.2	3.6	0.45	2.24
2	9:45		5.30			4.7			
	10:00	15	7.80	2.50	30	2.2	3.5	0.50	2.00
3	10:05		5.40			4.6			
	10:20	15	7.90	2.50	30	2.1	3.4	0.50	2.00
4	10:25		5.50			4.5			
	10:40	15	8.00	2.50	30	2.0	3.3	0.50	2.00
5	10:45		5.50			4.5			
	11:00	15	8.00	2.50	30	2.0	3.3	0.50	2.00

Percolation Rate (in/hr) 2.00 x60= 120 in/hr

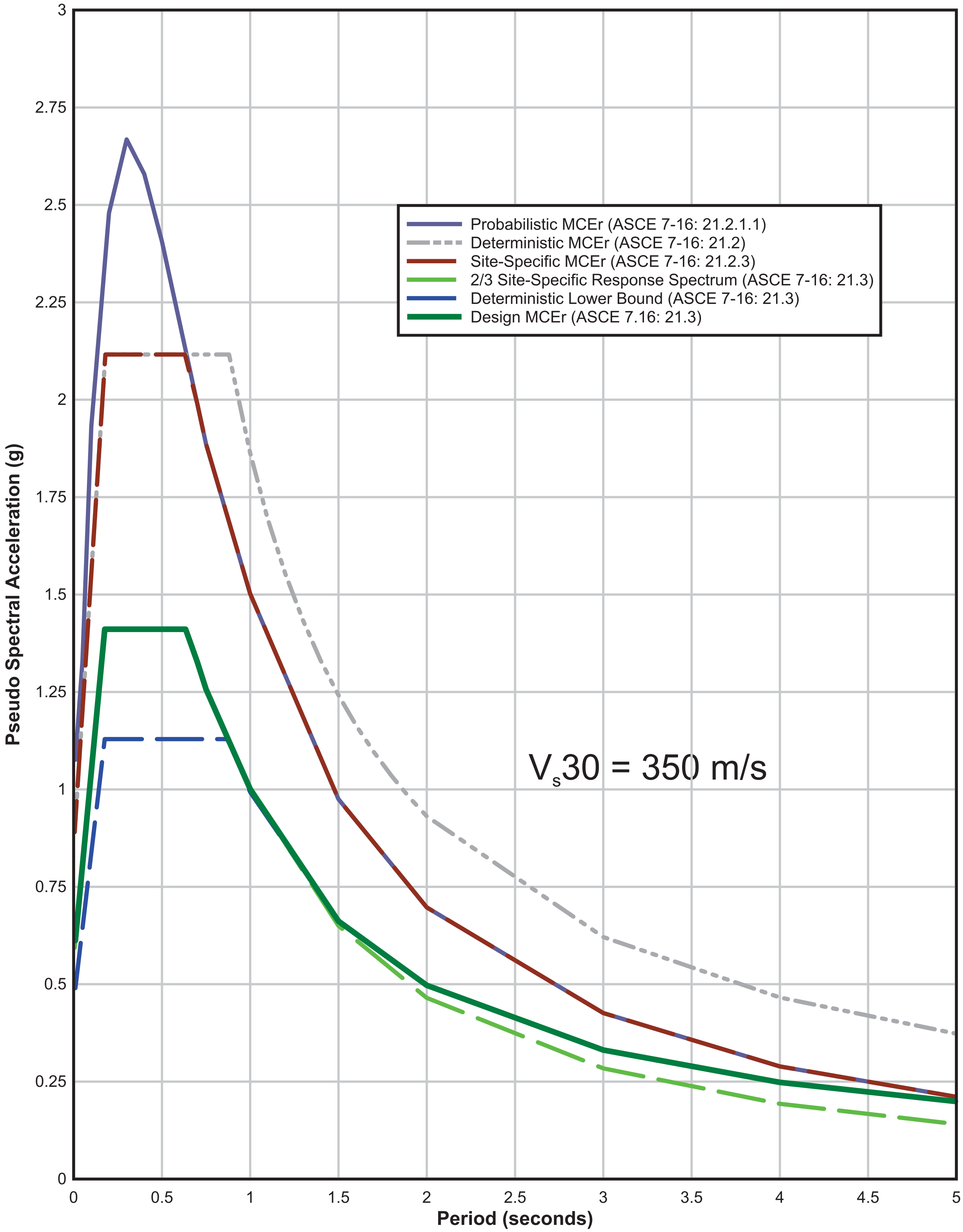
GS200.1 Rf= ((2xd1-Dd)/diam)+1

d1= depth of hole-initial depth = 4.50 ft 54 in
Dd= Change in depth= 2.50 ft 30 in

Rf= 10.75

Bottom only perc rate= 11.2 in/hr

APPENDIX E – GROUND MOTION STUDY



Job No. G21-003/1

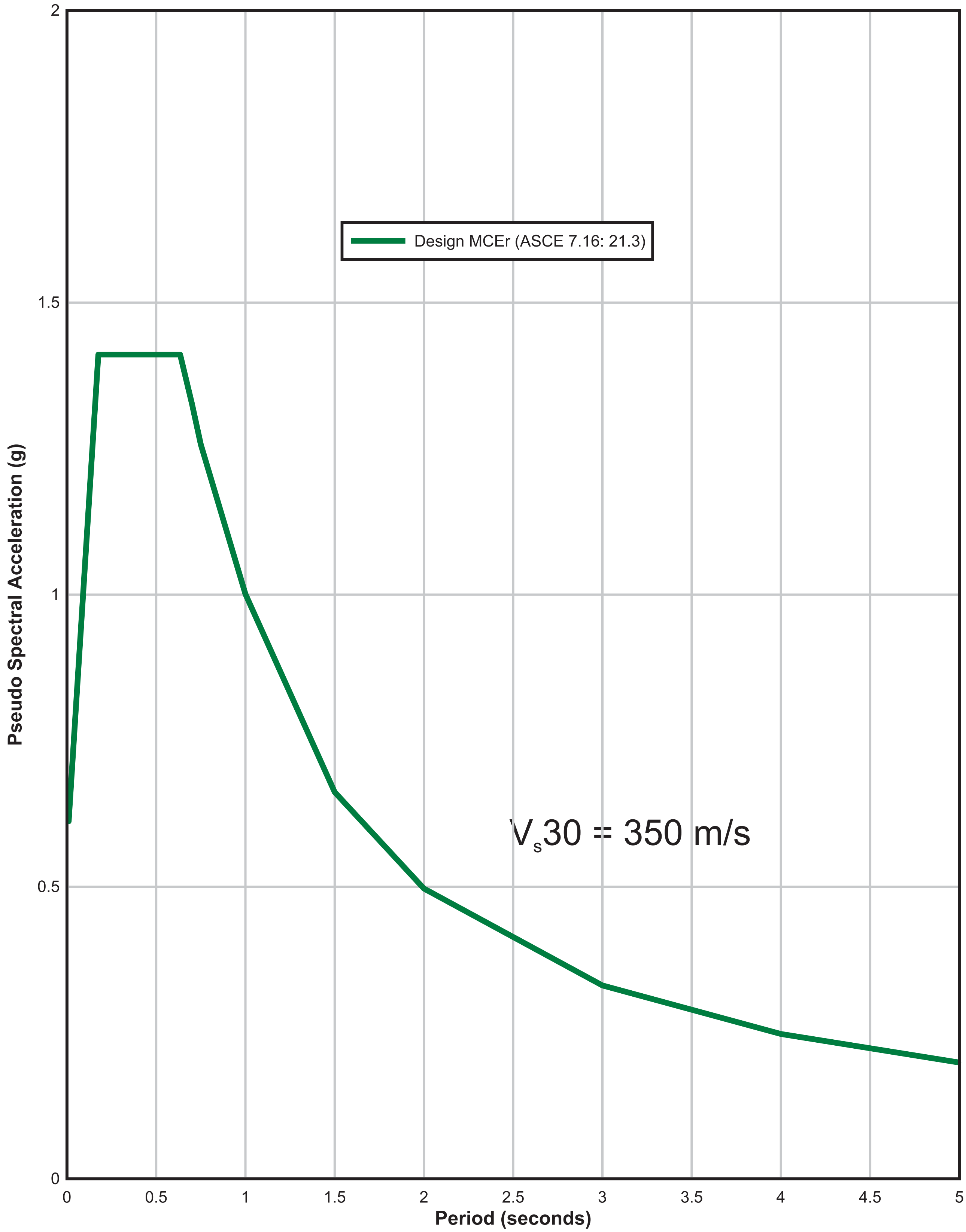
Date: March 2021

By: BDS

Risk-Targeted Spectra
2% Probability of Exceedandce in 50 Years
5% Damping

6422 Selma Avenue

FIGURE E-1



Job No. G21-003/1

Date: March 2021

By: BDS

Recommended Risk-Targeted Spectra (MCE_R)
2% Probability of Exceedandce in 50 Years
5% Damping
6422 Selma Avenue

FIGURE E-2

Pseudo Spectral Acceleration

ASCE 7-16 Site-Specific PSHA Procedure

Probabilistic MCE_R Sec. 21.2.1.1		**Deterministic MCE_R Sec. 21.2.2**		**Site-Specific MCE_R Sec. 21.2.3**		**2/3 Site-Specific MCE_R Sec. 21.3**		**Deterministic Lower Bound Sec. 21.3**		**Design MCE_R Sec. 21.3**	
T (sec)	PSA (g)	T (sec)	PSA (g)	T (sec)	PSA (g)	T (sec)	PSA (g)	T (sec)	PSA (g)	T (sec)	PSA (g)
0.010	1.076	0.010	0.919	0.001	0.854	0.001	0.569	0.010	0.490	0.010	0.612
0.050	1.328	0.176	2.116	0.010	0.919	0.010	0.612	0.176	1.129	0.176	1.411
0.100	1.932	0.880	2.116	0.176	2.116	0.176	1.411	0.200	1.129	0.200	1.411
0.200	2.479	0.900	2.069	0.200	2.116	0.200	1.411	0.300	1.129	0.300	1.411
0.300	2.668	1.000	1.863	0.300	2.116	0.300	1.411	0.500	1.129	0.500	1.411
0.400	2.579	1.200	1.552	0.634	2.116	0.634	1.411	0.700	1.129	0.634	1.411
0.500	2.406	1.300	1.433	0.700	1.990	0.700	1.327	0.868	1.129	0.700	1.327
0.750	1.886	1.500	1.242	0.750	1.886	0.750	1.257	0.900	1.104	0.750	1.257
1.000	1.502	1.600	1.164	1.000	1.502	1.000	1.001	1.000	0.993	1.000	1.001
1.500	0.975	1.700	1.096	1.500	0.975	1.500	0.650	1.500	0.662	1.500	0.662
2.000	0.697	2.000	0.931	2.000	0.697	2.000	0.465	2.000	0.497	2.000	0.497
3.000	0.426	3.000	0.621	3.000	0.426	3.000	0.284	3.000	0.331	3.000	0.331
4.000	0.289	4.000	0.466	4.000	0.289	4.000	0.193	4.000	0.248	4.000	0.248
5.000	0.211	5.000	0.373	5.000	0.211	5.000	0.141	5.000	0.199	5.000	0.199

FIGURE E-3

APPENDIX F – GEOCONCEPTS, INC BORINGS

BORING: B-1

ADDRESS: Selma Ave & Wilcox Ave — PROJECT NO.: 4718

DATE LOGGED: February 6, 2014 — LOGGED BY: KC

ATTITUDES: b - bedding, j - joint, s - shear, f - fault

WATER CONTENT, %	UNIT DRY WEIGHT, PCF	BLOWS/FOOT	DEPTH, FT	DESCRIPTION
				0.0' - 2.0" **ASPHALT**
				2.0" - 6.0" **CONCRETE and BASE**
6	106	11		0.5' - 3.0' **ARTIFICIAL FILL; Af,** silty sand, medium brown, slightly moist, fine to coarse grained, gravels up to 1/2" in length
9	110	10	5	@ 2.5' silty sand, medium brown, slightly moist, fine to coarse grained, rock fragments up to 1/2" in length
				3.0' - 81' **QUATERNARY ALLUVIUM; Qal,**
				@ 5.0' silty sand, brown, slightly moist, fine to medium grained, slightly porous
15	110	25	10	@ 10.0' clayey sand, medium brown, slightly moist, fine to medium grained
8	114	17	15	@ 15.0' sand, medium brown to brown, slightly moist, fine to coarse grained, gravels up to 1/4" in length
8	119	17	20	@ 20.0' sand, medium brown, slightly moist, fine to coarse grained, gravels up to 3/4" in length
7	111	18	25	@ 25.0' silty sand, medium brown, slightly moist, fine to medium grained
8	118	29	30	@ 30.0' silty sand, medium brown, slightly moist, fine to coarse grained
7	117	29	35	@ 35.0' silty sand, medium brown, slightly moist, fine to coarse grained, gravels up to 1/2" in length
8	115	40	40	@ 40.0' silty sand, medium brown, slightly moist, fine to coarse grained, gravels up to 1/2" in length

BORING: B-1

ADDRESS: Selma Ave & Wilcox Ave

PROJECT NO.: 4718

DATE LOGGED: February 6, 2014

LOGGED BY: KC

ATTITUDES (b - bedding, j - joint, s - shear, f - fault)	WATER CONTENT, %	UNIT DRY WEIGHT, PCF	BLOWS/FOOT	SAMPLES	DEPTH, FT	GRAPHIC LOG	DESCRIPTION
	7	112	43				@ 45.0' silty sand, medium brown, slightly moist, fine to medium grained, gravels up to 3/4" in length
	7	117	50		50		@ 50.0' silty sand, medium brown, slightly moist, fine to medium grained, gravels up to 3/4" in length
	15	119	40		55		@ 55.0' clayey sand, medium brown, slightly moist, fine to medium grained, gravels between 1/4" to 1/2" in length
	13	118	47		60		@ 60.0' clayey sand, medium brown, slightly moist, fine to medium grained
	5	121	51		65		@ 65.0' silty sand, brown, slightly moist, fine to coarse grained, gravels up to 1/2" in length
	7	123	49		70		@ 70.0' silty sand, brown, slightly moist, fine to coarse grained, gravels up to 1" in length
	9	121	56		75		@ 75.0' silty sand, light brown, slightly moist, fine to coarse grained
	13	124	64		80		@ 80.0' silty sand, light brown, slightly moist, fine to coarse grained
					85		Total Depth: 81.0 Feet No Groundwater Hollow Stem Auger

BORING: B-2

ADDRESS: Selma Ave & Wilcox Ave — PROJECT NO.: 4718

DATE LOGGED: February 6, 2014 — LOGGED BY: KC

ATTITUDES: b - bedding, j - joint, s - shear, f - fault

WATER CONTENT, %	UNIT DRY WEIGHT, PCF	BLOWS/FOOT	SAMPLES	DEPTH, FT	DESCRIPTION
					0.0' - 2.0" **ASPHALT**
					2.0" - 6.0" **CONCRETE and BASE**
6	111	17			0.5' - 3.0' **ARTIFICIAL FILL; Af,** silty sand, light brown, slightly moist, fine to coarse grained, rock fragments up to 1/2" in length
3	111	19		5	@ 2.5' silty sand, medium brown, slightly moist, fine to medium grained
					3.0' - 86.0' **QUATERNARY ALLUVIUM; Qal,** @ 5.0' silty sand, medium brown, slightly moist, fine to coarse grained, gravels 1/4" to 1" in length
16	111	17		10	@ 10.0' silty sand, medium brown, slightly moist, fine to medium grained, slightly porous
7	111	22		15	@ 15.0' silty sand, medium brown, slightly moist, fine to medium grained, slightly porous, few gravels up to 1/4" in length
10	111	15		20	@ 20.0' silty sand, light brown, slightly moist, fine to medium grained
5	112	26		25	@ 25.0' silty sand, medium brown, slightly moist, fine to medium grained, gravels up to 1" in length, fine to coarse grained
10	115	28		30	@ 30.0' silty sand, medium brown, slightly moist, fine to coarse grained
9	121	31		35	@ 35.0' silty sand, medium brown, slightly moist, fine to medium grained
7	123	38		40	@ 40.0' silty sand, medium brown, slightly moist, fine to medium grained

BORING: B-2

ADDRESS: Selma Ave & Wilcox Ave | PROJECT NO.: 4718

DATE LOGGED: February 6, 2014 | LOGGED BY: KC

ATTITUDES (b - bedding, j - joint, s - shear, f - fault)	WATER CONTENT, %	UNIT DRY WEIGHT, PCF	BLOWS/FOOT	SAMPLES	DEPTH, FT	GRAPHIC LOG	DESCRIPTION
	14	120	32				@ 45.0' clayey sand, medium brown to reddish brown, slightly moist, fine to medium grained, gravels 1/4" to 3/4" in length
	13	120	28		50		@ 50.0' clayey sand, medium brown to reddish brown, slightly moist, fine to medium grained
	14	122	46		55		@ 55.0' clayey sand, reddish brown, slightly moist to moist, fine to coarse grained, gravels up to 1/2" in length
	4	126	53		60		@ 60.0' clayey sand, reddish brown, slightly moist to moist, fine to coarse grained, gravels up to 1/4" in length
	6	124	64		65		@ 65.0' sand, orange brown to medium brown, slightly moist, fine to coarse grained, gravels 1/4" to 3/4" in length
	13	124	57		70		@ 70.0' sand, orange brown to medium brown, slightly moist, fine to coarse grained, gravels 1/4" to 3/4" in length
	5	124	60		75		@ 75.0' clayey sand, medium brown, slightly moist, fine to coarse grained
	12	121	64		80		@ 80.0' clayey sand, medium brown, slightly moist, fine to coarse grained
	9	126	67		85		@ 85.0' clayey sand, medium brown, slightly moist, fine to coarse grained

Total Depth: 86.0 Feet
No Groundwater
Hollow Stem Auger

BORING: B - 2

ADDRESS: 1523 - 1541 Wilcox Avenue

PROJECT NO.: 4755

DATE LOGGED: April 9, 2014

LOGGED BY: KNC

ATTITUDES: b - bedding, j - joint, s - shear, f - fault

WATER CONTENT, %	UNIT DRY WEIGHT, PCF	BLOWS/FOOT	DEPTH, FT	DESCRIPTION
			0	0.0' - 0.5" **ASPHALT**
8	96	18	5	0.5" - 2.0' **ARTIFICIAL FILL; Af,** silty sand, dark brown, moist, fine to coarse grained, concrete fragments
				2.0' - 81.0' **QUATERNARY ALLUVIUM; Qal**
				@ 5.0' sand (SW), orangish brown, slightly moist, fine to coarse grained, gravels up to 1" in length
11	110	20	10	@ 10.0' sand (SW) to silty sand (SM), medium brown, slightly moist, fine to medium grained
3	106	25	15	@ 15.0' silty sand (SM), orangish brown, slightly moist, fine to coarse grained, gravels up to 1/2" in length
7	120	33	20	@ 20.0' silty sand (SM), medium brown, slightly moist, fine to medium grained
15	105	25	25	@ 25.0' sandy silt (ML) to silty sand (SM), medium brown, slightly moist, fine to medium grained
8	116	24	30	@ 30.0' silty sand (SM), medium brown, slightly moist, fine to medium grained
10	116	28	35	@ 35.0' clayey sand (SC), light to medium brown, slightly moist, fine to medium grained
13	115	30	40	@ 40.0' clayey sand (SC), medium brown, slightly moist, fine to medium grained
10	115	33	45	@ 45.0' silty sand (SM), medium brown, slightly moist, fine grained
4	122	37	50	@ 50.0' sand (SW), light orangish brown, slightly moist, fine to coarse grained, abundant gravels up to 1" in length
4	123	35	55	@ 55.0' sand (SW), light orangish brown, slightly moist, fine to coarse grained, abundant gravels up to 1" in length
17	114	48	60	@ 60.0' silty sand (SM), medium brown, moist, fine to medium grained
9	115	61	65	@ 65.0' clayey sand (SC), medium brown, moist, fine to medium grained
13	120	62	70	@ 70.0' silty sand (SM), medium brown, moist, fine to medium grained
				@ 72.0' **GROUNDWATER**
13	118	67	75	@ 75.0' sand (SW), orangish brown, wet, fine to coarse grained, gravels up to 1" in length
13	120	85	80	@ 80.0' sand (SW) with gravel, orangish brown, wet, fine to coarse grained, gravels up to 1" in length
			85	Total Depth 81.0 Feet
				Groundwater @ 72.0 Feet
			90	8" Hollow stem Auger