GARCREST
ENGINEERING AND CONSTRUCTION, INC

REPORT OF GEOTECHNICAL INVESTIGATION

PROPOSED BUILDINGS

15101 Paramount Boulevard

Paramount, California

Prepared for:

NA CIVIL, INC.

Lake Forest, California

June 26, 2023

G23-009/1

GARCREST
ENGINEERING AND CONSTRUCTION, INC

126 S. Jackson Street, Suite 300 - Glendale, California 91205
Telephone: 818-241-2408 - Facsimile: 818-241-2282

June 26, 2023

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

Subject: **Report for Geotechnical Investigation**
Proposed Buildings
15101 Paramount Blvd
Paramount, California, 90723
Project No.: G23-009/1

Dear Mr. Ayoub:

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

Based on the results of our analysis, the site is susceptible to seismically induced liquefaction settlement. Accordingly, we recommend that the proposed smaller building pad be supported on a mat foundation underlain by at least 4 feet of properly compacted engineered fill. The area of the larger retail building proposed for the site should undergo ground improvement and modification techniques such as stone columns or rammed aggregate piers to a depth of at least 20 feet, following the improvements, the proposed building may be supported on shallow spread foundations, underlain by at least 4 feet of properly compacted engineered fill. If ground improvements are proposed for the smaller pad at the southeast corner, then that building may also be supported on spread foundations in lieu of a mat as recommended herein. At this time, recommendations for the small pad at the northwest are to be provided by others and are beyond the scope of 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
No. 2668
Exp. 12/31/24
GEOTECHNICAL
STATE OF CALIFORNIA

Armen Gaprelian, PE, GE
Principal Engineer

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1.0 - SCOPE

This report provides foundation design recommendations for the proposed buildings located at 15101 Paramount Blvd in Paramount, California. The site location is shown on Plate 1, Site Location Map. The proposed building footprints are 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 buildings. Our scope of services was performed in general accordance with our proposal dated January 25, 2023 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;
- Results of liquefaction evaluation;
- 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;
- Recommendations for deep and shallow 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 following the demolition of the existing structures at the site, three new buildings along with associated parking areas will be proposed for the subject site. The proposed buildings will consist of two smaller food services type pads that will include associated drive-thru lanes. The smaller pads will be located near the northwest corner of the site (Chick-fil-a) and the southeast corner of the site (Starbucks). The two smaller pads are anticipated to be approximately 3,000 square feet in size. In addition to the smaller building pads, one larger main retail building pad approximately 23,000 square feet in size is proposed near the central and western portion of the subject site.

At this time, we understand that the northwest corner pad will be investigated, designed, and constructed by the tenant, Chick-fil-a, and consequently the recommendations presented in our report will not be specific to that pad and finding in that area will be for information purposes only.

Recommendations presented in the current report are primarily focused on the Starbucks pad as well as the large main retail pad at the subject site.

We anticipate the structures to consist of single story type, wood framed construction. Subterranean construction is not anticipated. Structural loads are not yet available but are anticipated to be relatively light.

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 locations are shown on Plate 2, Plot Plan.

3.0 - FIELD EXPLORATION AND LABORATORY TESTING

The subsurface soil conditions at the site were explored by performing six hollow-stem-auger borings within the site. The borings were performed to depths of between approximately 11½ to 51½ 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 Borings B-5 and B-6, the borings were converted into percolation wells. 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.

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 test
- Atterberg Limits
- Percent Passing No.200 Sieve
- Maximum Density and Optimum Moisture Content
- 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, approved, 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 15101 Paramount Blvd in Paramount, California. The site is currently occupied by an Ace Hardware that occupies the majority of the eastern third of the site. The remaining portion of the site is occupied by a lumberyard; with a large L-shaped storage canopy in the north and west corner of the site, as well as a saw mill near the center of the site and several smaller warehouse style buildings near the south and southeast portions of the site. The remainder of the site consists of surface parking and drive aisles, with light landscaping mostly in the north east. The northwest section of the site is paved with asphalt, whereas the rest of the site is paved with concrete. A sewer main line in the center of the site runs north to south along what appears to be the continuation of an alley from the south. We anticipate numerous other utilities to cross the site.

5.0 – SUBSURFACE SOIL CONDITIONS

Fill soils to a depth of approximately 3 to 4 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, silt, and sand soils.

The native soils encountered at the site generally consist of medium stiff to very stiff sandy silt and medium dense to dense silty sand and sand soils. Insitu moisture contents vary between 1.2 and 26.6 percent and the dry density was between 88 and 120 pounds per cubic foot.

Groundwater was encountered in our borings at a depth of approximately 44 to 48 feet below grade. According to the State (CGS, 1998), historical high groundwater is anticipated to be at a depth of approximately 8 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 encountered in our borings at a depth of approximately 44 to 48 feet below grade. Historical high groundwater at the site is reported to be on the order of 8 feet below grade.

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 for the Design Level Earthquake (DBE). The result of the evaluation is attached, and is summarized as a Magnitude 6.6 and a ground acceleration of 0.55g. Our liquefaction analysis has been performed using these values.

Liquefaction analyses based on the simplified procedures developed by Seed and Idriss (1971), with modifications suggested by NCEER (1997) were performed for the design basis earthquake (DBE). 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. The results of our analyses are presented in Appendix C, Liquefaction Analysis.

Based on the results of our field investigation and laboratory testing, certain layers of the subsurface materials were evaluated for susceptibility for liquefaction using the requirements from Special Publication 117A - Guidelines for Evaluating and Mitigating Seismic Hazards in California dated 2008 (Page 35). Using these requirements and the results of laboratory testing, we evaluated for layer that might have been considered “borderline”, in order to determine whether to maintain or dismiss them from the analysis.

Given that the intent of the California Building Code is to maintain "Life Safety" during the Maximum Considered Earthquake (MCE) level event, additional analysis was performed for the evaluation of liquefaction potential. The structural engineer should evaluate the proposed structure for the anticipated MCE liquefaction induced settlement and verify that anticipated settlements will not cause adverse effects to the stability of the proposed foundations systems and cause collapse. The ground acceleration used was estimated at the PGA_M for the site. The result of the evaluation is attached, and is summarized as a Magnitude 6.8 and a ground acceleration of 0.83g.

The results of our liquefaction analyses estimate the seismically induced liquefaction settlements at the site following the site improvements as recommended in our report to be on the order of 1 to 1½-inch for the DBE level and on the order of 1½ to 2½-inch for the MCE level analysis. Differential settlements are estimated to be on the order ¾ to 1-inch for the DBE event and 1 to 1½-inch for the MCE event.

It should be noted that a portion of the total seismic settlement, approximately 1 to 1½ inch, is estimated to occur between approximately 35 to 50 feet below grade.

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 ¼-inch. Differential settlements are estimated to be less than ¼-inch.

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 4 feet below existing grade. Deeper fill soils may be present between and beyond our borings. The native soils generally consist of stiff sandy silts and medium dense to dense silty sands.

The onsite fill soils are not considered suitable for the support of proposed buildings and should be overexcavated to the firm and unyielding native soils and recompacted as properly compacted engineered fill.

As mentioned earlier, groundwater was encountered in our borings at a depth of approximately 44 to 48 feet below grade. According to the state historic high groundwater is mapped at a depth of approximately 8 feet below grade.

Based on the results of our field investigation and our liquefaction evaluation, some of the onsite soils may have susceptibility to seismically induced liquefaction settlement. The analysis evaluated seismically induced settlements on the order of 1½ to 2½-inch for the site, especially for the MCE level earthquake. Additionally, a portion of the total settlement, between 1 to 1½ inch, is estimated to occur between approximately 35 to 50 feet below grade.

Based on the liquefaction analysis, the estimated settlements are considered high and exceed tolerable limits, in their current state, for conventional spread foundations and may require support from alternative foundation systems, or though ground modification techniques.

It is important to note that based on our analysis and as mentioned above, part of the total seismic settlement occurs between approximately 35 to 50 feet in depth. Given this depth range, removal and recompaction alternatives are not feasible for mitigation. Additionally, although we have recommended drilled pier foundation alternatives, we also recommend that should this option be considered for implementation, additional investigation will need to be performed to determine the behavior of the soils below approximately 50 feet. The estimate degree and depth of liquefaction potential was not initially anticipated and our original scope did not include deep borings below 50 feet.

Given the above, pier foundations extending above 35 feet will be subject to potential seismic settlements below the tips of the piers. If piers are deepened below at least 50 feet below grade, additional downdrag forces as well as deeper investigation for potential settlement below the pier tips will be required. At this time, given the more limited information, we recommend that pier alternatives be carefully evaluated in comparison to other options presented below and if considered, additional investigation and analysis will be required.

As an alternative to pier foundations, portion of the upper onsite soils extending to at least 20 feet below grade may be densified by ground modification techniques such as stone columns or Rammed Aggregate Piers. With this approach, proposed buildings may be supported on shallow spread foundations system. Additional exploration and evaluation may be recommended after treatment of the subsurface to determine the level of improvement and revised seismic settlement potential at the site. These improvements will result in the densification of the soils below the proposed structures and can also assist with the dissipation of excess pore pressures developed during liquefaction, as well as reduce the liquefaction-induced seismic settlement. We recommend that prior to the installation of the stone columns, the soil beneath the proposed structures considered be overexcavated to a depth of at least 6 feet below the existing grade. The stone columns may be installed as specified by the specialty contractor, into the soils in a grid pattern extending at least 10 feet beyond the edge of the proposed structures, to a depth of at least 20 feet below. As this method is typically a design-build technique, further details for the methods and costing of ground improvements may be obtained from ground improvement contractors.

After installation of the stone columns, the upper 6 feet may be backfilled and compacted as properly compacted engineered fill to allow for the construction of the proposed spread foundations without interference from the stone columns.

Using the above ground modification alternatives, we estimate the total and differential seismically induced liquefaction settlements at the site may be reduced to approximately 1½-inch and ½-inch, respectively.

The smaller structure proposed in the southeastern corner of the site may be supported using either the ground modification techniques and spread foundation system recommended above, or using a mat foundation system.

To provide a uniform support, we recommend that for the support of the mat foundations, if considered, the upper at least 6 feet of the onsite soils or at least 4 feet below the bottom of the mat, whichever is deeper, be overexcavated and recompacted as properly compacted, engineered fill. The proposed excavation bottom should also extend laterally a distance of at least 5 feet beyond the edge of the proposed mat foundations, where feasible.

Mat foundations may be designed for an allowable bearing value of 1,500 pounds per square foot.

Slabs on grade may be supported on the properly compacted soils as recommended herein.

7.2 - EARTHWORK

7.2.1 - Site Preparation

As discussed earlier, the proposed structures may be supported on spread foundations following ground modification techniques, or a mat foundations for the smaller building pad, established in the properly compacted engineered fill. Pier foundations if considered, may require additional investigation to mitigate deeper settlement potential.

The onsite fill soils are not considered suitable for the support of the proposed foundations and slabs on grade and should be overexcavated and recompacted as properly compacted engineered fill.

As recommended above, depending on the foundation system and the ground modification procedure, following the overexcavation of the existing onsite soils to a depth of at least 6 feet below grade, or at least 4 feet below the bottom of the proposed mat foundations for the smaller building, whichever is deeper, the exposed subgrade should be observed by a Garcrest representative for unsuitable soils and debris and the excavation deepened as necessary. The excavation should extend at least 10 feet beyond the edge of the building if ground modification techniques are considered. If the mat foundation system is considered, the overexcavation should extend at least 5 feet laterally beyond the edge of the proposed mat foundations, where feasible. In areas where deeper fill is encountered, the excavation should be deepened to the firm and unyielding native soils locally.

The extent of removal and recompaction below the proposed pavement areas may be reduced to approximately 2 feet below existing grade.

The exposed subgrade should then be scarified to a depth of 8-inches, brought to within 2 to 4 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 sand, silt, and sand soils.

7.2.3 - Compaction

Engineered fill soils should be placed in loose lifts of no more than 8-inches, brought to 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 90 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 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.

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 - FOUNDATIONS

As mentioned above, at this time, we understand that the northwest corner pad will be investigated, designed, and constructed by the tenant, Chick-fil-a, and consequently the recommendations presented in our report will not be specific to that pad and finding in that area will be for information purposes only.

Given the liquefaction potential at the site, shallow spread foundations are not recommended for the proposed buildings without site ground modification and improvement.

The larger retail building pad may be supported on drilled piers, although, we recommend that this approach be more carefully evaluated, and that additional investigation will be required and are recommended prior to the finalization of this design approach.

To provide support for the larger retail pad, we recommend as an alternative to deep foundations, the use of ground improvement and modification techniques such as stone columns or Rammed Aggregate Piers, established as recommended above, to a depth of at least 20 feet below grade.

Following the overexcavation, ground improvements, and recompaction, as discussed above, the proposed large retail building may be supported on shallow spread foundations established in the properly compacted engineered fill soils. Proposed foundations should be underlain by at least 4 feet of properly compacted engineered fill soils.

The smaller structure proposed in the southeastern corners of the site may be supported on either spread foundations following ground improvements, or if not ground improvement if desired here, a mat foundations established in and underlain by at least 4 feet of properly compacted engineered fill soils prepared as recommended in the Earthwork section above.

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.

7.3.1 - Bearing Value

Spread foundations at least 18-inches in width and established at least 18-inches below the lowest adjacent grade, may be designed for a net dead-plus-live allowable pressure of 2,500 pounds per square feet.

Mat foundations established at least 18-inches below the lowest adjacent grade may be designed for a net dead-plus-live allowable pressure of 1,500 pounds per square feet.

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.

7.3.2 - Settlement

Based on the anticipated foundation loads and dimensions, we anticipate the total static settlement of the proposed foundations established on spread foundations, in the properly compacted engineered fill, and following ground improvements recommended above, to be on the order of ½- to ¾-inch. Differential settlements are anticipated to be less than ½-inch.

Anticipated settlement for the smaller building at the southeast corner, if opted to be supported on a mat foundation and no ground improvement, at estimated to be on the order of 1- to 1½-inch and ¾-inchfor total and differential settlements, respectively.

In general, static settlement of 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.

7.3.3 - 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 250 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.

7.3.4 - 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 at least 3 feet of properly compacted engineered fill or undisturbed native soils.

7.3.5 - Drilled Piers

At this time, drilled pier recommendations are provided, however, we recommend that if this approach is considered for the support of the larger building pad, additional field investigation and analysis, extending below 50 feet be performed to determine the settlement potential of the soils below that depth. At the time of this report, the seismic settlements and depths involved were not anticipated, and accordingly deeper investigation was not included or considered necessary as part of our original scope of work.

Additionally, should drilled pier foundations be considered, piers above approximately 35 feet are anticipated to be subject to deeper seismically induced liquefaction settlements. Piers should therefore extend to at least 50 feet below grade, in which case additional downdrag loading as well as deeper settlement potential is possible and will need to be evaluated through the additional exploration and analysis recommended.

Allowable downward capacities for 18-, and 24-inch diameter drilled cast-in-place concrete friction piers are presented in the following table:

Allowable Downward Drilled Pier Capacity

Pier Length below grade beam (ft)	18-inch Diam. Allowable Downward Capacity (kips)	24-inch Diam. Allowable Downward Capacity (kips)
5	4	5
10	9	12
15	16	21
20	24	32
25	32	43
30	42	56
35	52	70

The downward capacities are based on frictional resistance only. Uplift capacities may be taken as one-half the downward capacity. The drilled piers should have a minimum embedment of 5 feet into the onsite soils. A one-third increase in capacity may be used for wind and seismic loading.

The capacities are based on the strength of the soils; the compressive and tensile strength of the pier section itself should be checked to verify the structural capacity of the piers.

7.3.6 - Lateral Loads for Drilled Piers

Lateral loads may be resisted by the piers and by the passive resistance of the soils.

For computing the lateral resistance of drilled piers, an acceptable pole formula such as the one in the California Building Code may be used. When using the pole formula for individual piers, a lateral resistance of 500 pounds per cubic foot, up to a maximum of 5,000 pound per square foot may be used. The lateral resistance may be assumed to develop at a depth of one-foot below the surface level. The proposed piers should be embedded into the onsite soils to a depth of at least 5 feet. The effective depth may be taken as the depth of reinforcement in the pier but not more than 20 feet.

7.3.7 - Drilled Pier Installation

All drilled pier excavations should be visually observed by the geotechnical engineer or his representative. Appropriate drilling and excavation equipment should be used for the construction of the project and piers. Precautions should be taken during the installation of the piers to minimize caving and raveling. Among other precautions, the drilling speed may be reduced as necessary to reduce vibration and sloughing. If desired, casings may also be used during installation and slowly removed during placement of concrete. The level of the concrete should remain about 5 feet above the bottom of the casing during this operation. Drilling fluid may also be used to reduce the caving potential of the sidewalls of the pier excavation.

The pier excavations should be filled with concrete as soon after drilling and inspection as possible; the hole should not be left open overnight. We recommend that the adjacent piers be drilled and concreted alternately. The concrete should be placed with special equipment so that the concrete is not allowed to fall freely more than five feet and to prevent concrete from striking the walls of the excavation. The concrete must be capable of propagating between the reinforcing bars to come in contact with the soil and to avoid arching.

A concrete mix with a water to cement ratio of less than 0.50 should be used in the construction of the piers to reduce shrinkage of the concrete. An increased slump may be desirable to increase the fluidity of the mix for improved consolidation and bond with reinforcing steel. With a low water/cement ratio, slump during concrete placement should be increased using a plasticizer as opposed to adding water to the mix.

7.3.8 - Pier Group Reduction

Pile in groups should be spaced at least 3 diameter on center. Pile so spaced need not consider group action reduction for axial capacity.

Group effect for lateral behavior may be evaluated based on the table below. Lateral spacing of 8 diameters on center or greater need not consider lateral group effect. The recommendations presented in the table below were obtained from the California Amendment to the AASHTO LRFD Bridge design specifications, Section 10.

Pile P-Multiplier			
Pile Center to Center Spacing (in load direction)	Row 1	Row 2	Row 3 and above
2D	0.6	0.35	0.25
3D	0.75	0.55	0.40
5D	1.0	0.85	0.70
7D	1.0	1.0	0.90

D=Diameter of Pile

7.3.9 - Settlement

As discussed above, seismically induced liquefaction settlements are anticipated at the site between approximately 35 to 50 feet below grade. Drilled pier foundations, if considered, and established above 35 feet, are anticipated to be subject to seismically induced settlements estimated to be on the order of 1- to 1½-inch. Differential settlements are anticipated to be on the order of ½-inch. Static total and differential settlements are estimated to be on the order of ½-inch and ½-inch, respectively, and will be in addition to the above seismically induced settlements.

Pier foundations extending below 50 feet will be subject to downdrag loading as well as settlement potential below 50 feet. Given the maximum exploration depth of 50 feet in our current investigation, settlement of soils below that depth may not be estimated at this time, and we recommend that additional field exploration as analysis be performed should this alternative be considered for design.

7.4 - SEISMIC CONSIDERATIONS

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 1.627 and 0.583 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 **SMS** and **SM1** of 1.953g and 0.991g and **SDS** and **SD1** of 1.302g and 0.661g may be used for a Site Class D.

7.5 – 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.

Percolation testing was performed in two borings at the site. Borings B-5 and B-6 were both drilled to a depth of 10 feet and percolation tests were performed directly in the borings. The 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 3-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 5 minutes for a period of approximately 30 minutes or until at least three virtually even consecutive readings, the water being replenished subsequent to each reading interval. The results of the tests are presented in Appendix D, Percolation Testing and summarized in the following table.

The measured percolation rate is based on the small diameter boring shallow infiltration test setup in accordance with the County of Los Angeles guidelines (GS-200.1).

Boring/Well No.	Adjusted Percolation Rate (inch/hr)
B-5	5.38
B-6	4.29

7.5.1 – 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 of approximately 4.29 to 5.38 inch/hr may be considered relatively representative of the overall conditions at the site although some siltier or sandier zones may affect the rate. These rates have not been factored for design purposes but include sidewall reductions for borehole testing.

Groundwater was encountered within our borings performed at a depth of approximately 45 feet. According to the State (CGS, 1998), historical high groundwater is anticipated to be on the order of 8 feet below 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 materials. 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 20 feet from proposed or existing foundations

7.5.2 – Additional Discussions

Liquefaction Potential Discussion

As discussed earlier, the site is located within a State designated liquefaction hazard zone. Further, the depth to historical high groundwater is anticipated to be at approximately 8 below the existing grade. Detailed liquefaction evaluation was performed and discussed earlier in this report. The results of our analysis indicate that the site has a potential for seismically induced liquefaction settlements, with estimated settlements on the order of 1- to 2½- inches, depending on the level of seismic event. To reduce the potential for adverse effects from water for the proposed structures, we recommend that if infiltration devices are considered for the site, that the devices be kept away from existing or proposed buildings foundations by a distance of at least 20 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 encountered within our borings at a depth of approximately 44 to 48 feet below grade. Typical infiltration requirements limit the depth of a device such as to maintain a separation of at least 10 feet from groundwater.

The upper onsite soils are generally silty and 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.

7.6 - FLOOR SLAB SUPPORT

Following the preparation of the subgrade as recommended above, 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 16-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 110 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.

7.7 - PAVEMENT DESIGN

To provide support for paving, 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 the purposes of design, we have assumed an R-value representative of the onsite soils. Confirmatory testing may be required during the grading and earthwork. We have assumed an R-value of 20 for design.

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	8
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	7	4
Truck Traffic	6	7½	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 2 feet below the pavement section, brought to within 3 percent above the optimum moisture content, and compacted to a minimum of 90 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.

7.8 - 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 slopped 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.

7.9 - 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.

7.10 - 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 severe potential to buried ferrous metals and a moderate 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 cleaning and excavating operations,
- Observation and inspection of the exposed subgrades to receive fill,
- Evaluation of the suitability of import soils,
- 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.

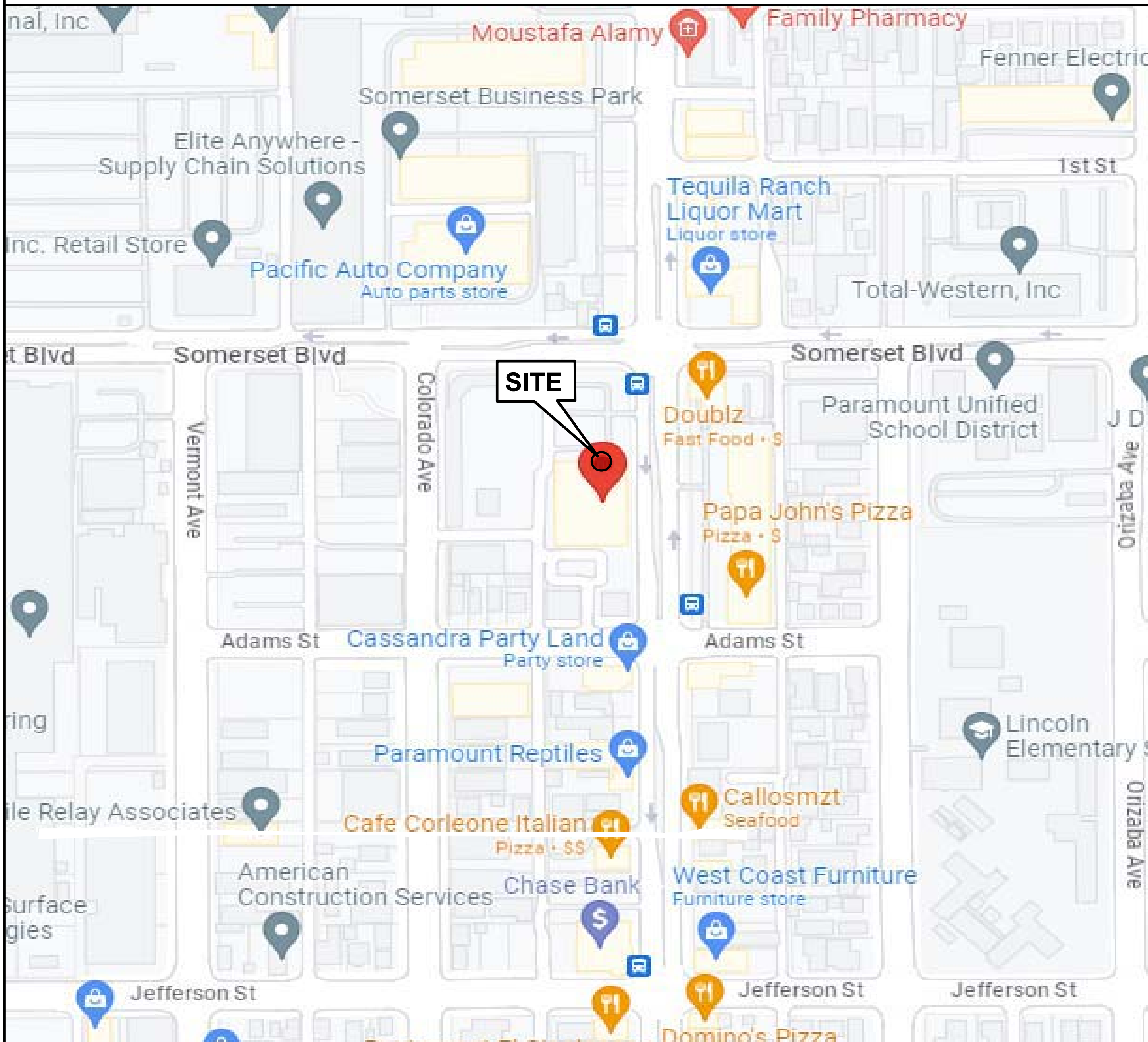
This report has been prepared exclusively for NA Civil, Inc. as the project civil engineer, their client, and other associated design consultants for the specific application of their project located at 15101 Paramount Blvd, Paramount, California. This report has not been prepared for other parties and 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, and 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

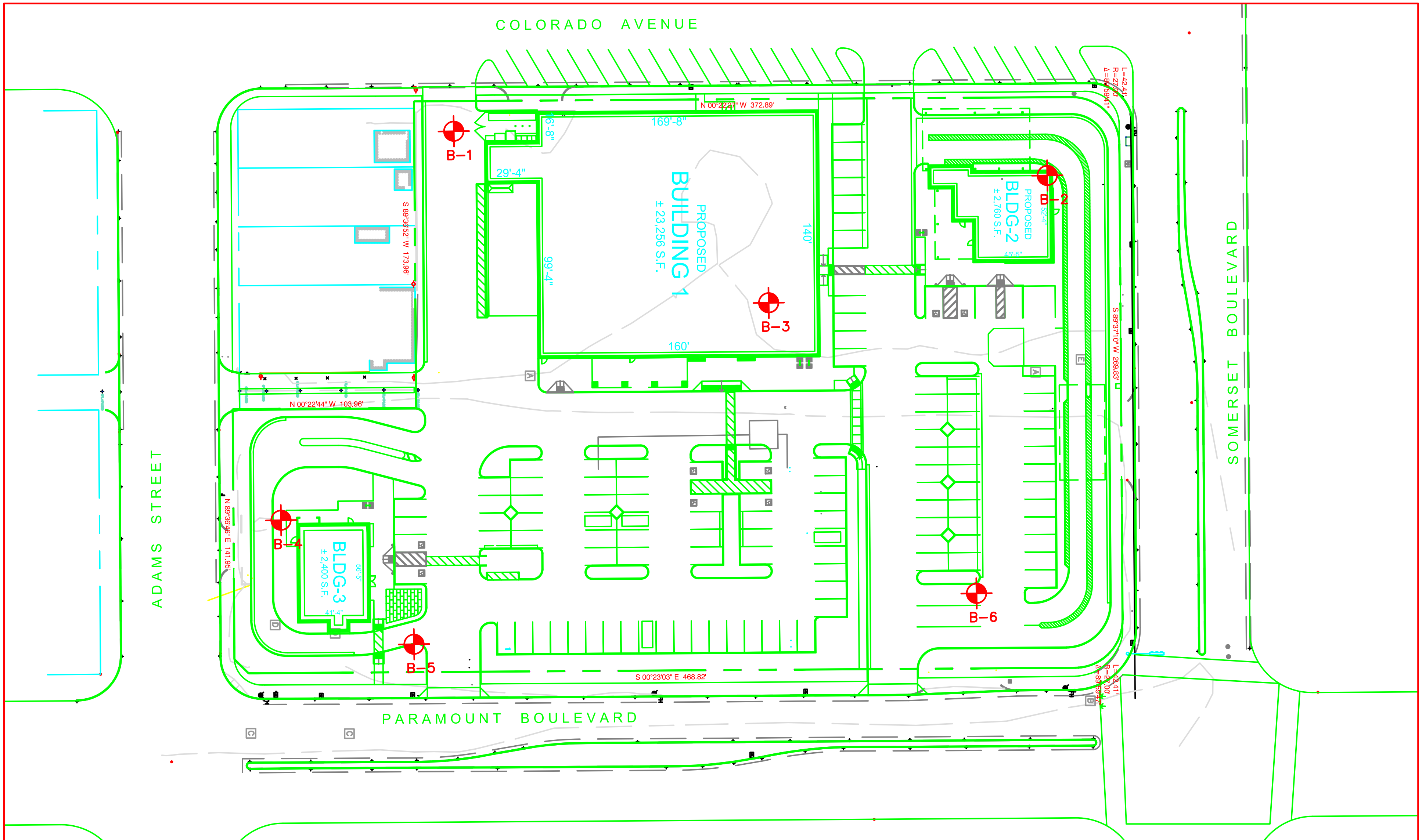
PLATES

SITE LOCATION MAP



Satellite image retrieved from https://www.google.com/maps/

PROJECT: 15101 Paramount Blvd, Paramount, CA	PROJECT NO: G23-009/1	DATE: June 2023
GARCREST ENGINEERING AND CONSTRUCTION, INC.		PLATE 1



KEY Approximate Boring Location

REFERENCE: Base plan provided by NA Civil

SCALE

1 80

NORTH

GARCREST

ENGINEERING AND CONSTRUCTION, INC.

PLOT PLAN

Proposed Buildings
15101 Paramount Blvd, Paramount, California
Project No. G23-009/1

PLATE

2

APPENDIX A – FIELD EXPLORATION

APPENDIX A

FIELD EXPLORATION

The soil conditions at the site were explored by drilling three borings using a truck-mounted hollow stem auger type drilling equipment provided by 2R Drilling of Chino, California. The borings were performed on March 17, 2023. The borings were advanced to a depth of 51½ feet below the existing grade. The boring locations are shown on Plate 2, Plot Plan. The borings were backfilled using the excavated cuttings and patched.

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-6, 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.:	G23-009/1	**DRILLER:**	2R	**LOGGED BY:**	JD
PROJECT NAME:	Paramount	**DRILL METHOD:**	8" Hollow Stem Auger	**OPERATOR:**	George
LOCATION:	15101 Paramount Blvd, Paramount. CA	**HAMMER:**	140 pound Auto/30 inches	**RIG TYPE:**	CME 75
ELEVATION:				**DATE:**	3/17/2023

BORING NO.: *B-1*

Depth (ft)	Sample Type	Blows/ 6"	Blows/Foot	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Moisture Content (%)	Dry Density (pcf)	Others
0						**3-inch asphalt, 6-inch base, 5-inch asphalt**			
	Bulk			1	SM	**FILL** **SILTY SAND -** Dark Brown. fine. moist			
					SM	**ALLUVIUM** **SILTY SAND -** Dark Brown. fine. moist. medium dense			
5	Ring	7 10 11	21	2		-- medium dense	11.3	106	DS
	SPT	2 5 5	10	3	ML	**SANDY SILT -** Mottled brown, moist, stiif			
	Ring	4 5 13	18	4		-- Dark brown			
10	SPT	4 5 6	11	5	SM	**SILTY SAND -** Dark brown. fine. moist. medium dense			
15	Ring	8 15 18	33	6			13.3	104	
20	SPT	4 8 9	17	7		-- Brown, 21.9 percent passing No. 200 sieve			WASH
25	SPT	4 9 10	19	8		-- Dark grey, more silt, 55.1 percent passing No. 200 sieve			WASH

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

chk: AG 03/15/23

PLATE A-1a

Garcrest Engineering & Construction, Inc.
LOG OF BORING

PROJECT NO.:	G23-009/1	**DRILLER:**	2R	**LOGGED BY:**	JD
PROJECT NAME:	Paramount	**DRILL METHOD:**	8" Hollow Stem Auger	**OPERATOR:**	George
LOCATION:	15101 Paramount Blvd, Paramount. CA	**HAMMER:**	140 pound Auto/30 inches	**RIG TYPE:**	CME 75
ELEVATION:				**DATE:**	3/17/2023

BORING NO.: *B-1 (cont.)*

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
30		5 9 9	18	9		**SM**	**SILTY SAND -** Dark grey, fine, moist, medium dense (continued)			
35		7 11 11	22	10		**SP**	**SAND -** Grey, fine to medium, moist, medium dense -- 12.2 percent passing No. 200 sieve			WASH
40		3 5 12	17	11		**SM**	**SILTY SAND -** Dark grey, fine, wet, medium dense			
45		3 5 6	11	12		**ML**	**SANDY SILT -** with Clay, Grey, wet, stiff -- 51.5 percent passing No. 200 sieve	22.6		WASH PI
						SM	**SILTY SAND** - Dark grey, fine, wet, medium dense			
50		4 10 17	27	13						
55							<u>**NOTES:**</u> BORING TERMINATED AT 50 feet. Groundwater Encountered at 44-Feet Boring backfilled with cuttings and patched			

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

chk: AG 03/15/23

PLATE A-1b

Garcrest Engineering & Construction, Inc.
LOG OF BORING

PROJECT NO.:	G23-009/1	**DRILLER:**	2R	**LOGGED BY:**	JD
PROJECT NAME:	Paramount	**DRILL METHOD:**	8" Hollow Stem Auger	**OPERATOR:**	George
LOCATION:	15101 Paramount Blvd, Paramount. CA	**HAMMER:**	140 pound Auto/30 inches	**RIG TYPE:**	CME 75
ELEVATION:				**DATE:**	3/17/2023

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
							2.5-inch asphalt, 9-inch base			
	Bulk			1		SM	**FILL** **SILTY SAND -** Dark brown. moist	14.7	112	CORR MAX REM CS REM DS
						SM	**ALLUVIUM** **SILTY SAND -** Dark brown, moist, medium dense			
5	Ring	3 5 8	13	2		ML	**SANDY SILT** - Brown, moist, stiff			
	Ring	4 6 8	14	3				12.7	91	CONS
10	Ring	7 9 16	25	4			-- mottled brown, 58.4 percent passing No. 200 sieve			WASH
						SM	**SILTY SAND -** Light brown, fine to very fine, moist, dense			
15	Ring	10 18 26	44	5				4.4	102	
20	SPT	5 5 5	10	6			-- more silt, medium dense, 54.1 percent passing No. 200 sieve			WASH
25	SPT	6 7 8	15	7						

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

chk: AG 03/15/23

PLATE A-2a

Garcrest Engineering & Construction, Inc.
LOG OF BORING

PROJECT NO.:	G23-009/1	**DRILLER:**	2R	**LOGGED BY:**	JD
PROJECT NAME:	Paramount	**DRILL METHOD:**	8" Hollow Stem Auger	**OPERATOR:**	George
LOCATION:	15101 Paramount Blvd, Paramount. CA	**HAMMER:**	140 pound Auto/30 inches	**RIG TYPE:**	CME 75
ELEVATION:				**DATE:**	3/17/2023

BORING NO.: *B-2 (cont.)*

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
30						SM	**SILTY SAND -** Dark grey, fine, moist, medium dense (continued)			
		5 9 11	20	8			-- more silt, 54.2 percent passing No. 200 sieve			WASH
35		4 7 10	17	9			-- 46.8 percent passing No. 200 sieve			WASH
						ML	**SANDY SILT -** Dark grey, moist, trace clay, very stiff			
40		7 13 15	28	10			-- 55.5 percent passing No. 200 sieve			WASH
45		2 6 8	14	11			-- wet, stiff	26.6		PI
						SM	**SILTY SAND -** Dark grey, fine, wet, medium dense			
50		3 8 12	20	12						
55							**NOTES:** BORING TERMINATED AT 50 feet. Groundwater Encountered at 48-Feet Boring backfilled with cuttings and patched			

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

chk: AG 03/15/23

PLATE A-2b

Garcrest Engineering & Construction, Inc.
LOG OF BORING

PROJECT NO.:	G23-009/1	**DRILLER:**	2R	**LOGGED BY:**	JD
PROJECT NAME:	Paramount	**DRILL METHOD:**	8" Hollow Stem Auger	**OPERATOR:**	George
LOCATION:	15101 Paramount Blvd, Paramount. CA	**HAMMER:**	140 pound Auto/30 inches	**RIG TYPE:**	CME 75
ELEVATION:				**DATE:**	3/17/2023

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
							3-inch asphalt, 4-inch base, 4-inch asphalt			
						SM	**FILL** **SILTY SAND -** Brown. fine. moist			
						ML	**ALLUVIUM** **SANDY SILT-** Dark brown. very fine. moist. stiff			
5		4 6 10	16	1						
		5 7 10	17	2			-- gray brown, fine	25.5	94	CONS
10		4 9 11	20	3		SM	**SILTY SAND** - Brown. fine. moist. medium dense	5.4	120	
15		6 10 12	22	4						
20		8 7 6	13	5			-- 29 percent passing No. 200 sieve	12.6	99	WASH
25		7 10 11	21	6						
							NOTES: BORING TERMINATED AT 26½ feet. No Groundwater Encountered Boring backfilled with cuttings and patched.			

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

chk: AG 03/15/23

PLATE A-3

Garcrest Engineering & Construction, Inc.
LOG OF BORING

PROJECT NO.:	G23-009/1	**DRILLER:**	2R	**LOGGED BY:**	JD
PROJECT NAME:	Paramount	**DRILL METHOD:**	8" Hollow Stem Auger	**OPERATOR:**	George
LOCATION:	15101 Paramount Blvd, Paramount. CA	**HAMMER:**	140 pound Auto/30 inches	**RIG TYPE:**	CME 75
ELEVATION:				**DATE:**	3/17/2023

BORING NO.: *B-4*

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
							7-inch concrete, no base			
						SP	**FILL** **SAND -** Brown. fine. moist			
						SP	**ALLUVIUM** **SAND -** Brown. fine to medium. slightly moist. medium dense			
5	Ring	7 11 11	22	1			-- with silt	1.2	88	DS
	Ring	6 10 11	21	2						
10	Ring	9 12 14	26	3				2.0	94	CONS
						SM	**SILTY SAND -** Drak brown. fine. moist. dense			
15	Ring	14 23 24	47	4				10.0	110	
20	Ring	10 18 18	36	5			-- brown			
25	Ring	16 25 28	53	6			-- 44.4 percent passing No. 200 sieve	4.8	111	WASH
							NOTES: BORING TERMINATED AT 26½ feet. No Groundwater Encountered Boring backfilled with cuttings and patched.			

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

chk: AG 03/15/23

PLATE A-4

Garcrest Engineering & Construction, Inc.
LOG OF BORING

PROJECT NO.:	G23-009/1	**DRILLER:**	2R	**LOGGED BY:**	JD
PROJECT NAME:	Paramount	**DRILL METHOD:**	8" Hollow Stem Auger	**OPERATOR:**	George
LOCATION:	15101 Paramount Blvd, Paramount. CA	**HAMMER:**	140 pound Auto/30 inches	**RIG TYPE:**	CME 75
ELEVATION:				**DATE:**	3/17/2023

BORING NO.: *B-5*

Depth (ft)	Sample Type	Blows/ 6"	Blows/Foot	Sample Number	Graphical Log	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Moisture Content (%)	Dry Density (pcf)	Others
							4.5-inch concrete, no base			
						SM	**FILL** **SILTY SAND -** Dark brown. fine. moist trace gravel			
						SM	**ALLUVIUM** **SILTY SAND -** Brown. fine to coarse. moist. medium dense			
5		5 7 12	19	1			**--** mottled brown, fine			
						SP	**SAND -** light brown, fine to medium, moist, medium dense			
10		8 10 14	24	2				4.7	93	
							NOTES: BORING TERMINATED AT 11½ feet. No Groundwater Encountered Boring converted to percolation hole. Following testing, Boring backfilled with cuttings and patched.			
15										
20										
25										

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

chk: AG 03/15/23

PLATE A-5

Garcrest Engineering & Construction, Inc.
LOG OF BORING

PROJECT NO.:	G23-009/1	**DRILLER:**	2R	**LOGGED BY:**	JD
PROJECT NAME:	Paramount	**DRILL METHOD:**	8" Hollow Stem Auger	**OPERATOR:**	George
LOCATION:	15101 Paramount Blvd, Paramount. CA	**HAMMER:**	140 pound Auto/30 inches	**RIG TYPE:**	CME 75
ELEVATION:				**DATE:**	3/17/2023

BORING NO.: *B-6*

Depth (ft)	Sample Type	Blows/ 6"	Blows/Foot	Sample Number	Graphical Log	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Moisture Content (%)	Dry Density (pcf)	Others
							5-inch concrete, no base			
						SM	**FILL** **SILTY SAND -** Dark brown. fine. moist trace gravel			
						SM	**ALLUVIUM** **SILTY SAND -** Light brown. fine. moist. medium dense			
5		6 10 12	22	1		SP	**SAND -** light brown. fine to medium. moist. medium dense	5.0	99	
10		3 3 5	8	2		ML	**SANDY SILT** - Dark grey. fine. moist. stiff			
15 20 25							**NOTES:** BORING TERMINATED AT 11½ feet. No Groundwater Encountered Boring converted to percolation hole. Following testing, Boring backfilled with cuttings and patched.			

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

chk: AG 03/15/23

PLATE A-6

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.

Plasticity index testing was performed on selected samples of the soils to evaluate the plasticity characteristics and to aid in classification. The tests were performed in general accordance with ASTM Standard Test Method D 4318. The results are presented on Figure B-5, Atterberg Limits.

Maximum density and optimum moisture testing was performed on selected bulk samples of the onsite soils to determine optimum compaction characteristics. The test was performed in general accordance with ASTM Standard Method D-1557-91. The test results are presented on Figure B-6, Compaction Test.

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-7, Corrosion Test Results

AP Engineering & Testing, Inc.

A Certified DBE/MBE/SBE Company

March 31, 2023

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: Paramount Bldg
Project No.: G23-009/1

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:

- 8 Moisture Content & Density (ASTM D 2216 & D 7263)
- 2 Moisture Content Only (ASTM D 2216)
- 2 Atterberg Limits (ASTM D 4318)
- 5 Direct Shear (ASTM D 3080)
- 4 Consolidation (ASTM D 2435)
- 1 Corrosion Suite (CTM 417, 422 & 643)
- 1 Modified Proctor Compaction (ASTM D 1557)
- 11 Percent Passing #200 Sieve (ASTM D 1140)

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

REGISTERED PROFESSIONAL ENGINEER
APICHART PHUKUNHAPHAN
No. 002156
Exp. 12/31/23
GEOTECHNICAL
STATE OF CALIFORNIA

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

Distribution: 1 Addressee

Attachments: Laboratory Test Results

2607 Pomona Boulevard, Pomona, CA 91768 ◆ Tel 909.869.6316 ◆ Fax 909.869.6318 ◆ www.aplaboratory.com

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MOISTURE AND DENSITY TEST RESULTS

ASTM D2216 and ASTM D7263 (Method B)

Client: Garcrest Engineering
Project Name: Paramount Bldg
Project No.: G23-009/1

AP Lab No.: 23-0355
Test Date: 03/27/23

Boring No.	Sample No.	Sample Depth (ft.)	Moisture Content (%)	Dry Density (pcf)
B1	-	15	13.3	103.8
B1	-	45	22.6	NA
B2	-	15	4.4	102.1
B2	-	45	26.6	NA
B3	-	10	5.4	119.8
B3	-	20	12.6	99.0
B4	-	15	10.0	110.3
B4	-	25	4.8	110.5
B5	-	10	4.7	92.6
B6	-	5	5.0	99.3

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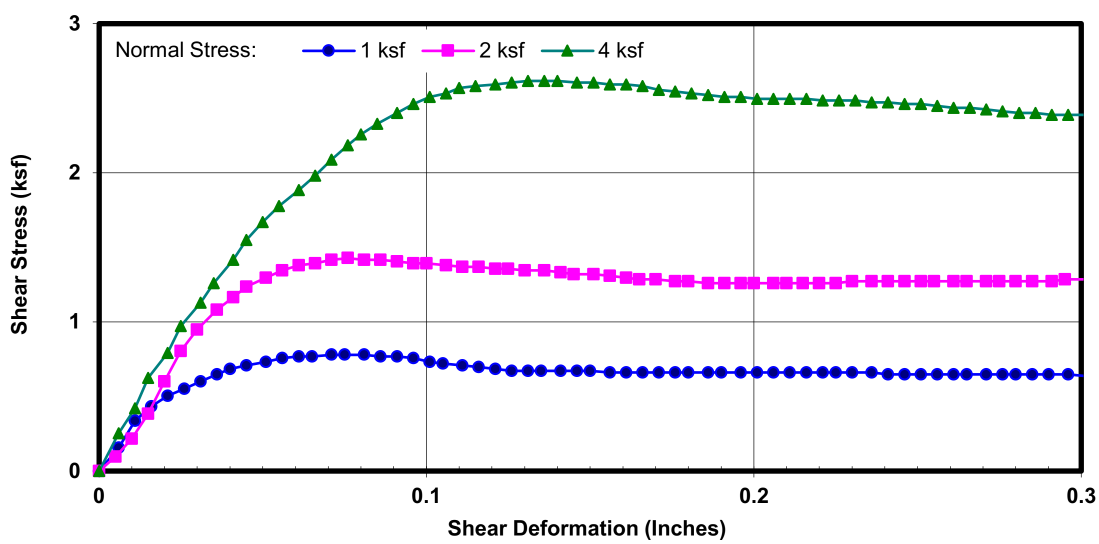
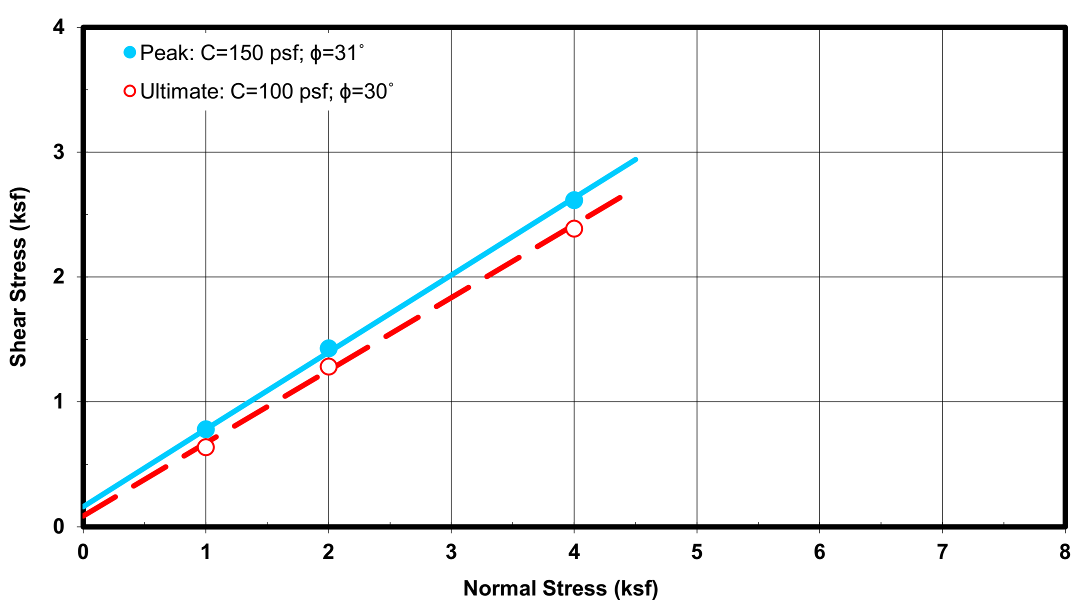
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name: Paramount Bldg
Project No.: G23-009/1
Boring No.: B1
Sample No.: - **Depth (ft):** 5
Sample Type: Mod. Cal.
Soil Description: Silty Sand, fine-grained
Test Condition: Inundated **Shear Type:** Regular

Tested By: AP **Date:** 03/27/23
Computed By: NR **Date:** 03/29/23
Checked by: AP **Date:** 03/29/23

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)
118.1	106.1	11.3	20.1	52	92	1	0.780	0.636
						2	1.428	1.284
						4	2.616	2.388





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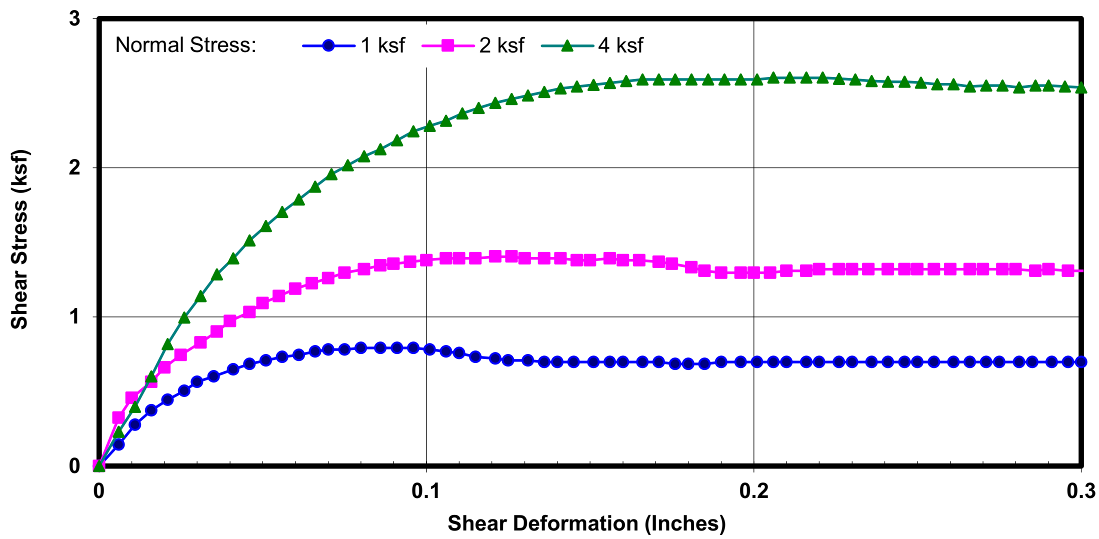
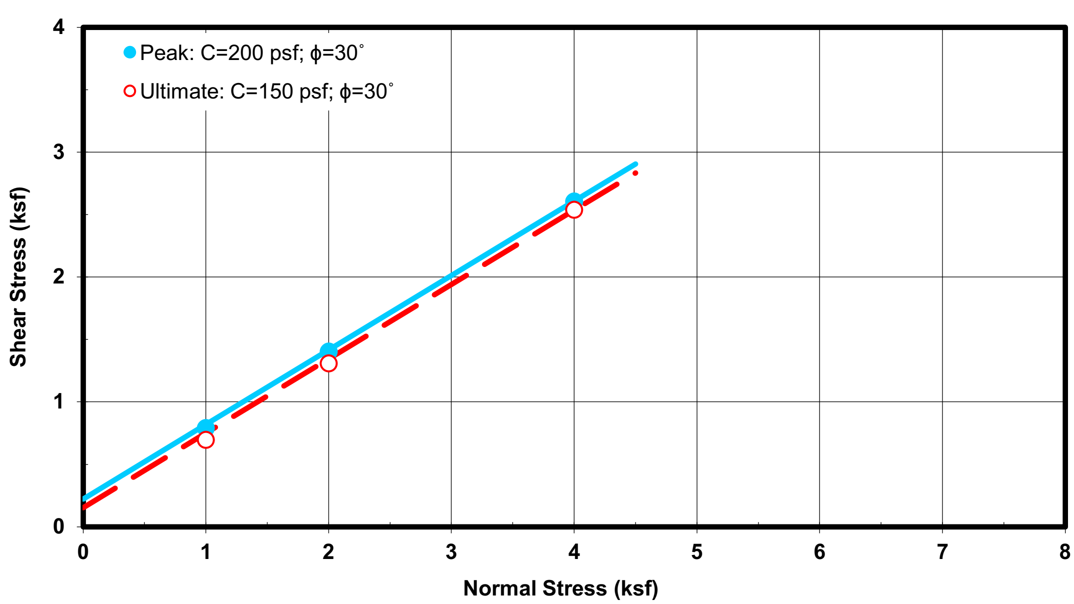
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name: Paramount Bldg
Project No.: G23-009/1
Boring No.: B1
Sample No.: - **Depth (ft):** 15
Sample Type: Mod. Cal.
Soil Description: Silty Sand, fine-grained
Test Condition: Inundated **Shear Type:** Regular

Tested By: AP **Date:** 05/19/23
Computed By: JP **Date:** 05/22/23
Checked by: AP **Date:** 05/22/23

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)
110.7	98.5	12.3	26.0	47	99	1	0.792	0.696
						2	1.404	1.308
						4	2.604	2.539





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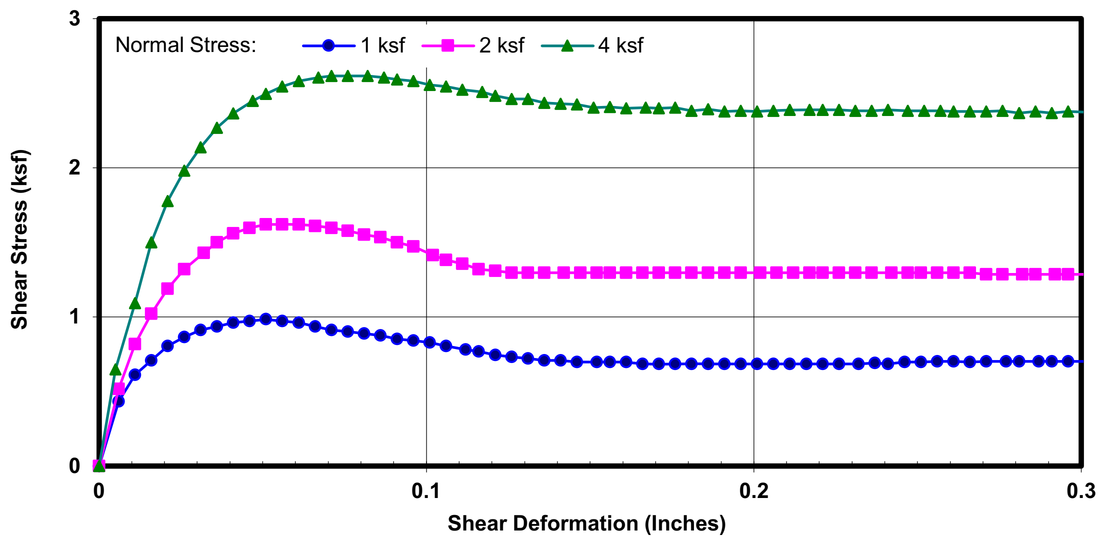
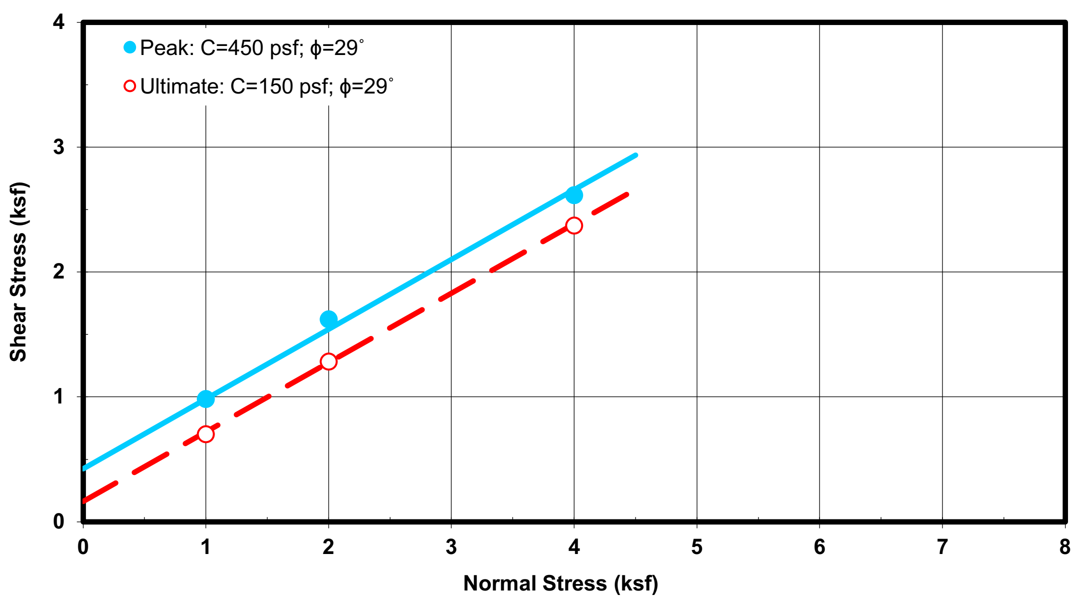
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name: Paramount Bldg
Project No.: G23-009/1
Boring No.: B2
Sample Type: Bulk **Depth (ft):** 0-5
Remold Cond.: Remolded to 95% RC at OMC+2%
Soil Description: Sandy Silt
Test Condition: Inundated **Shear Type:** Regular

Tested By: AP **Date:** 03/27/23
Computed By: NR **Date:** 03/29/23
Checked by: AP **Date:** 03/29/23

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)
128.2	111.7	14.7	18.7	78	100	1	0.984	0.702
						2	1.620	1.284
						4	2.616	2.372





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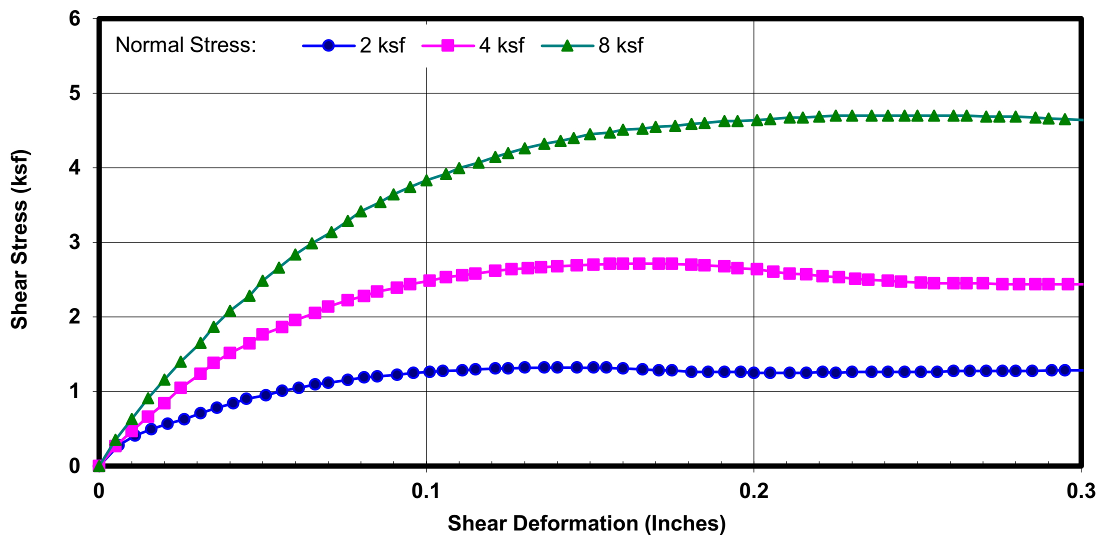
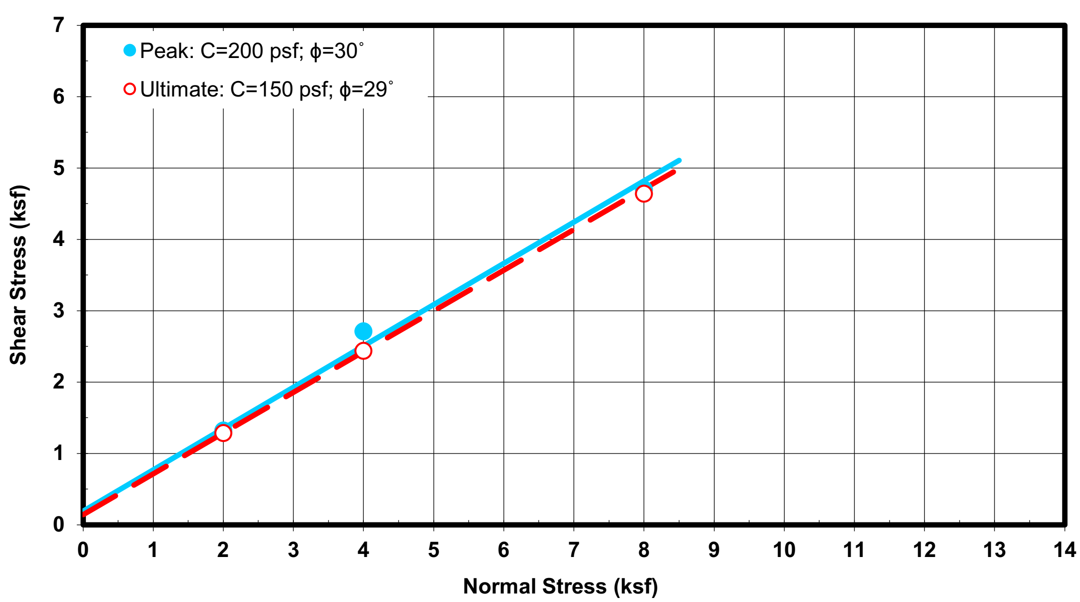
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name: Paramount Bldg
Project No.: G23-009/1
Boring No.: B3
Sample No.: - **Depth (ft):** 20
Sample Type: Mod. Cal.
Soil Description: Silty Sand
Test Condition: Inundated **Shear Type:** Regular

Tested By: AP **Date:** 05/19/23
Computed By: JP **Date:** 05/22/23
Checked by: AP **Date:** 05/22/23

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)
103.4	91.3	13.3	27.5	43	88	2	1.320	1.284
						4	2.712	2.436
						8	4.700	4.638





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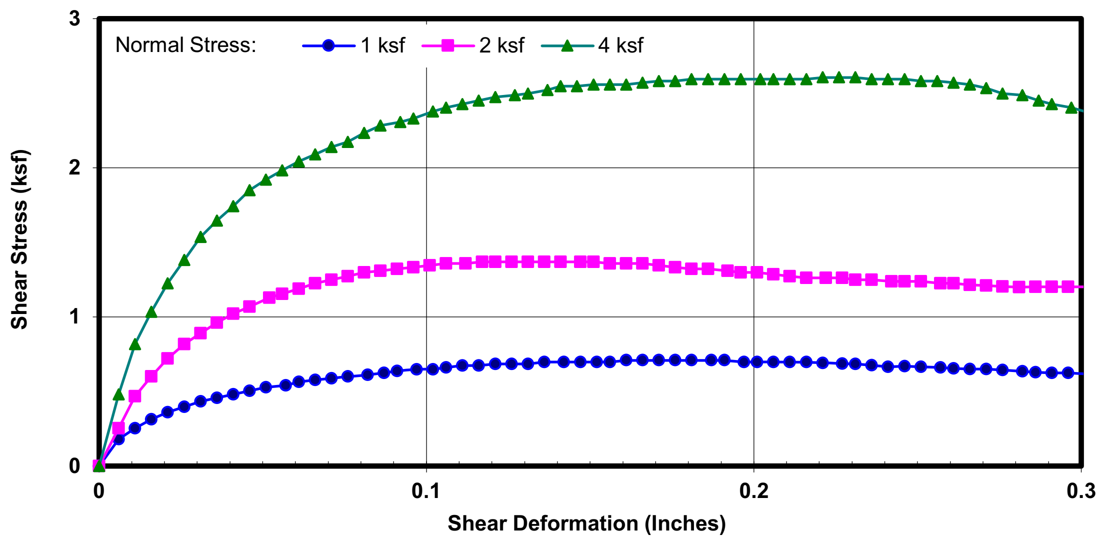
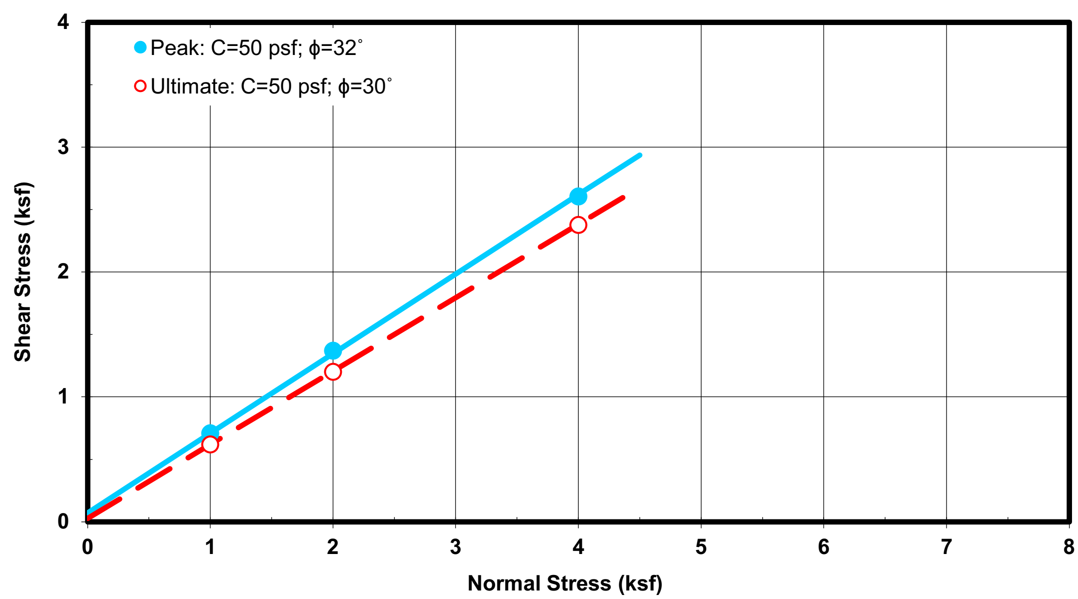
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name:	Paramount Bldg	**Tested By:**	AP	**Date:**	03/27/23
Project No.:	G23-009/1	**Computed By:**	NR	**Date:**	03/29/23
Boring No.:	B4	**Checked by:**	AP	**Date:**	03/29/23

Sample No.: - **Depth (ft):** 5
Sample Type: Mod. Cal.
Soil Description: Sand w/silt
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)
88.8	87.7	1.2	30.3	4	89	1	0.709	0.618
						2	1.369	1.201
						4	2.606	2.378





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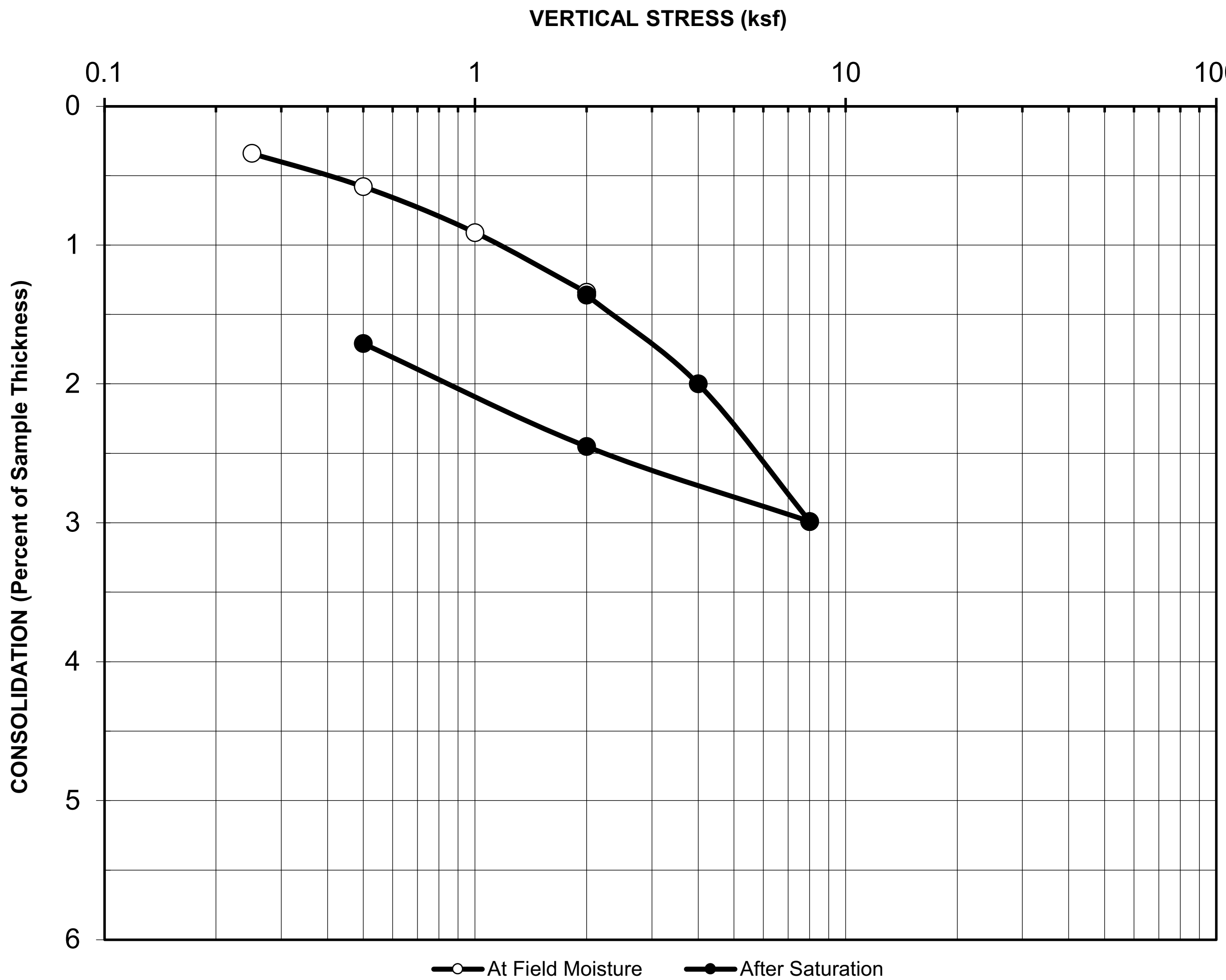

Boring No. :	B2	Initial Dry Unit Weight (pcf):	111.8
Sample Type:	Bulk	Initial Moisture Content (%):	14.7
Depth (feet):	0-5	Final Moisture Content (%):	19.3
Remold Cond.:	Remolded to 95% RC at OMC+2%	Assumed Specific Gravity:	2.7
Soil Description:	Sandy Silt	Initial Void Ratio:	0.51
Remarks:	Collapse= 0.02% upon inundation		

CONSOLIDATION CURVE
ASTM D 2435

Project Name:	Paramount Bldg
Project No.:	G23-009/1
Date:	3/24/2023
AP No:	23-0355 **Sheet No:** 1

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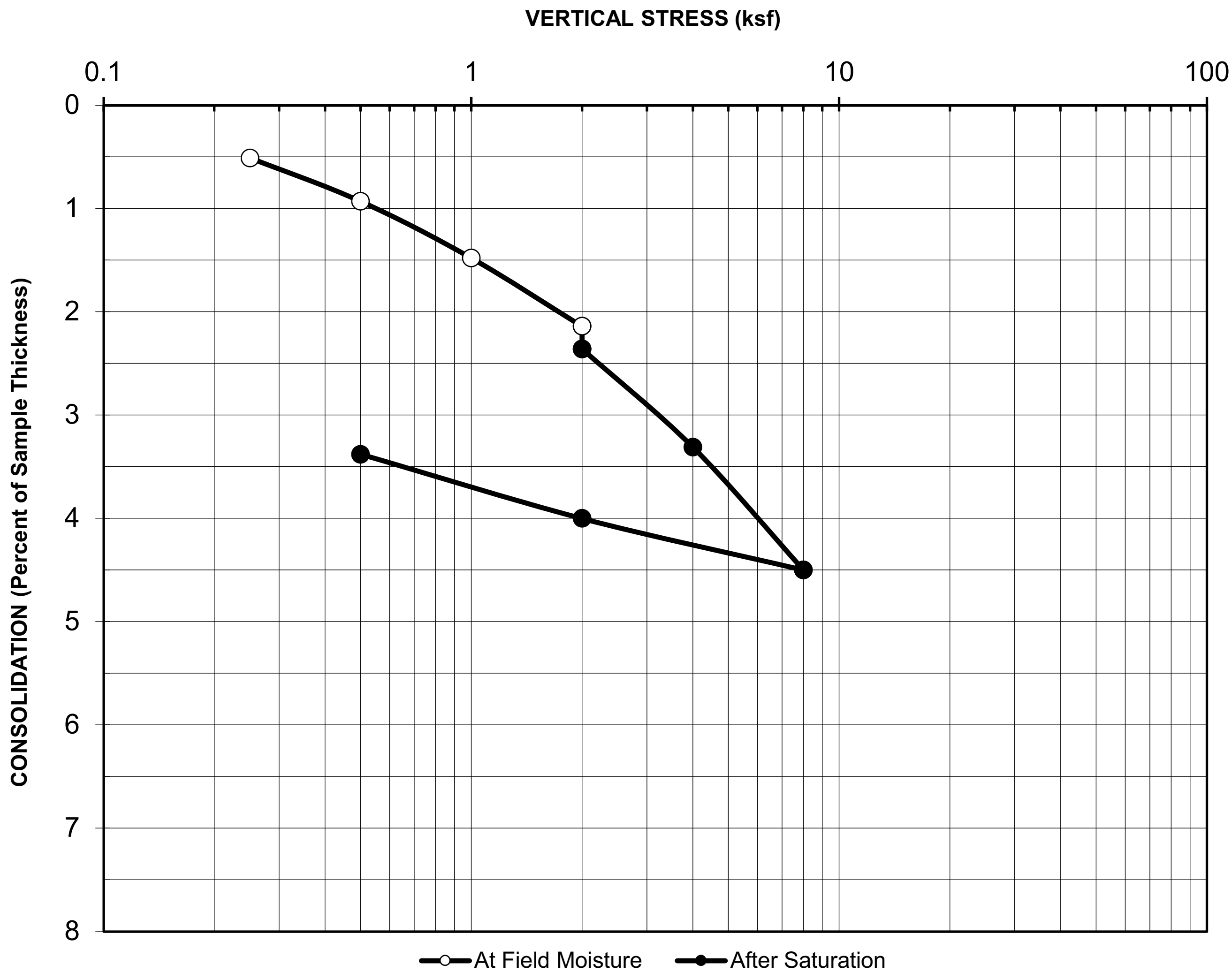

Boring No. :	B2	Initial Dry Unit Weight (pcf):	91.4
Sample No.:	-	Initial Moisture Content (%):	12.7
Depth (feet):	7	Final Moisture Content (%):	29.2
Sample Type:	Mod Cal	Assumed Specific Gravity:	2.7
Soil Description:	Sandy Silt	Initial Void Ratio:	0.84
Remarks:	Collapse= 0.22% upon inundation		

CONSOLIDATION CURVE
ASTM D 2435

Project Name: Paramount Bldg
Project No.: G23-009/1
Date: 3/23/2023
AP No: 23-0355 **Sheet No:** 1

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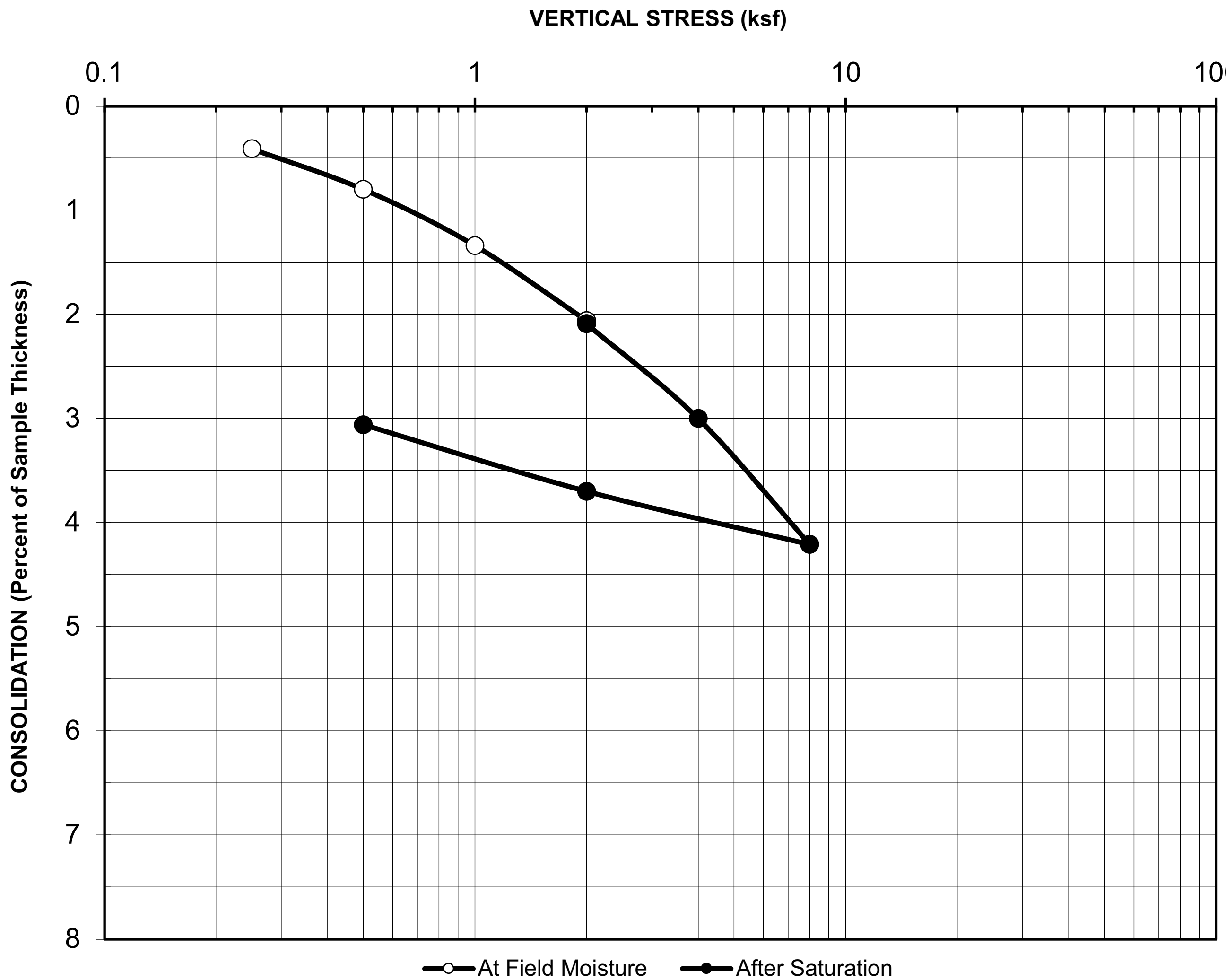

Boring No. :	B3	Initial Dry Unit Weight (pcf):	94.1
Sample No.:	-	Initial Moisture Content (%):	25.5
Depth (feet):	7	Final Moisture Content (%):	29.3
Sample Type:	Mod Cal	Assumed Specific Gravity:	2.7
Soil Description:	Silt w/sand	Initial Void Ratio:	0.79
Remarks:	Collapse= 0.03% upon inundation		

CONSOLIDATION CURVE
ASTM D 2435

Project Name:	Paramount Bldg
Project No.:	G23-009/1
Date:	3/23/2023
AP No:	23-0355
Sheet No:	1

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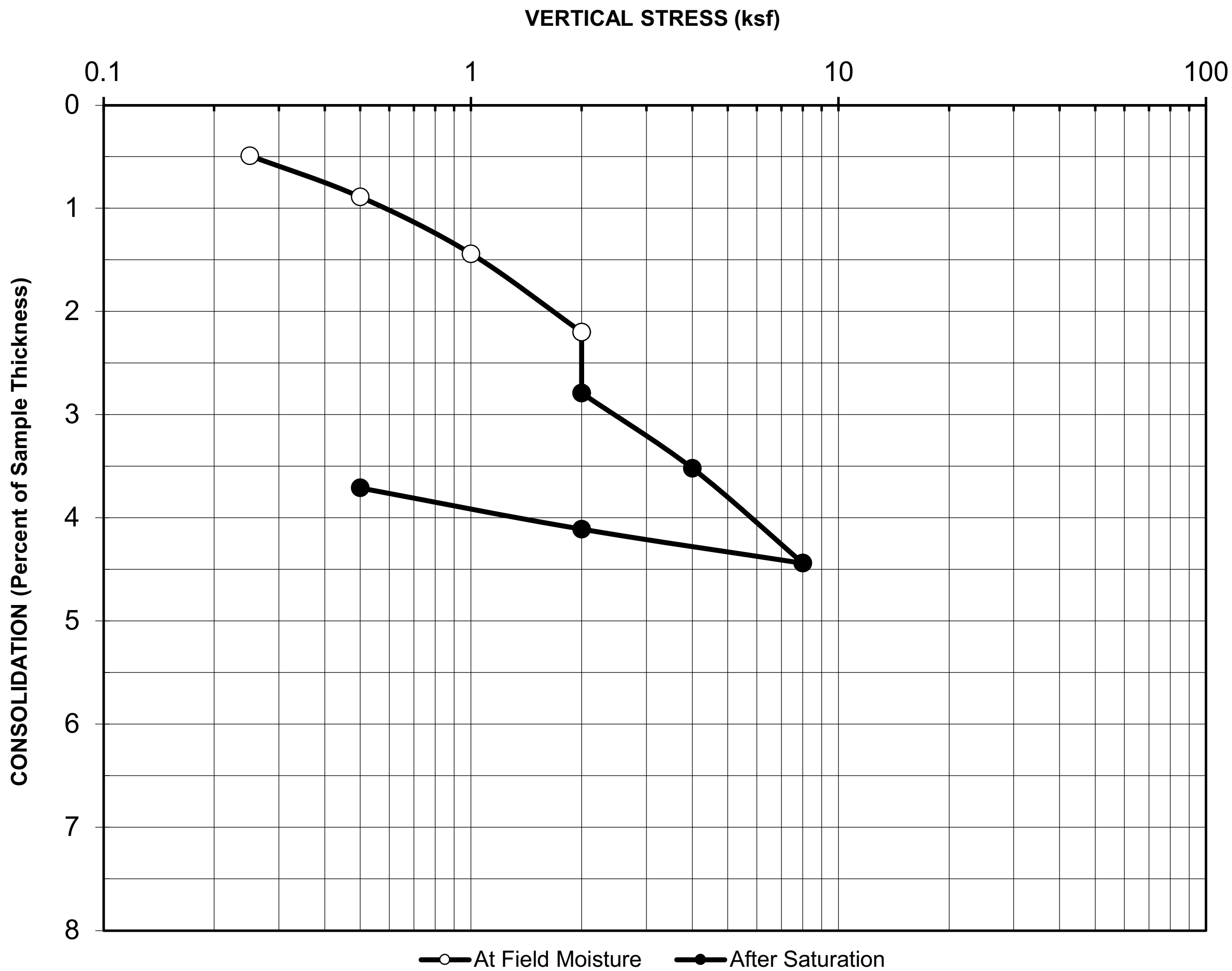

Boring No. :	B4	Initial Dry Unit Weight (pcf):	94.1
Sample No.:	-	Initial Moisture Content (%):	2.0
Depth (feet):	10	Final Moisture Content (%):	26.1
Sample Type:	Mod Cal	Assumed Specific Gravity:	2.7
Soil Description:	Poorly-Graded Sand w/silt	Initial Void Ratio:	0.79
Remarks:	Collapse= 0.59% upon inundation		

CONSOLIDATION CURVE
ASTM D 2435

Project Name: Paramount Bldg
Project No.: G23-009/1
Date: 3/23/2023
AP No: 23-0355 **Sheet No:** 1

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PERCENT PASSING NO. 200 SIEVE

ASTM D1140

Client: Garcrest Engineering
AP Lab No.: 23-0355
Project Name: Paramount Bldg
Test Date: 03/27/23
Project Number: G23-009/1

Boring No.	Sample No.	Depth (ft)	Percent Fines (%)
B1	-	20	21.9
B1	-	25	55.1
B1	-	35	12.2
B1	-	45	51.5
B2	-	10	58.4
B2	-	20	54.1
B2	-	30	54.2
B2	-	35	46.8
B2	-	40	55.5
B3	-	20	29.0
B4	-	25	44.4

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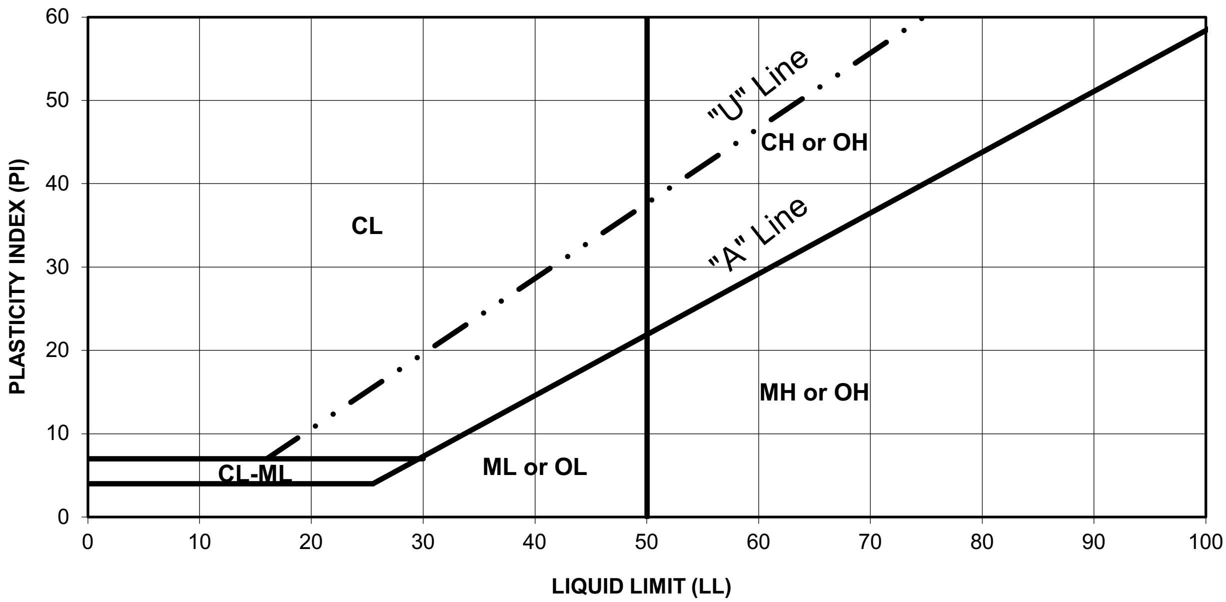
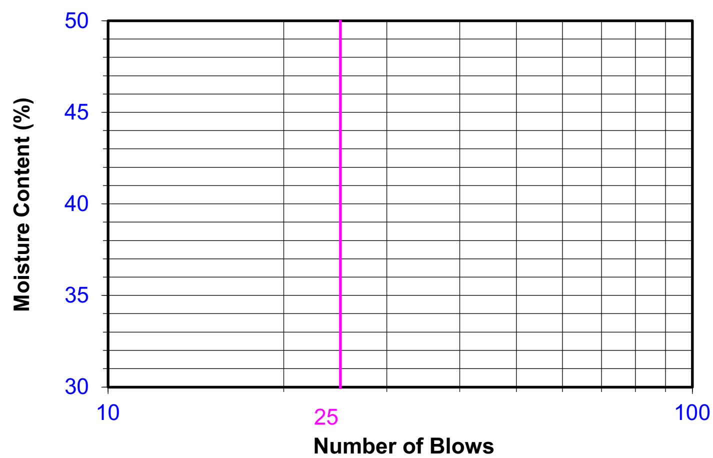
ATTERBERG LIMITS
ASTM D 4318

Client Name:	Garcrest Engineering	Tested By:	DK	Date:	03/28/23
Project Name:	Paramount Bldg	Computed By:	NR	Date:	03/29/23
Project No.:	G23-009/1	Checked By:	AP	Date:	03/29/23



PROCEDURE USED

- [ ] Wet Preparation
- [X] Dry Preparation
- [X] Procedure A
  Multipoint Test
- [ ] Procedure B
  One-point Test



Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
	B1	-	45	NP	NP	NP	
	B2	-	45	NP	NP	NP	

* NP denotes "non-plastic"

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CORROSION TEST RESULTS

Client Name: Garcrest Engineering
Project Name: Paramount Bldg
Project No.: G23-009/1

AP Job No.: 23-0355
Date: 03/27/23

Boring No.	Sample Type	Depth (feet)	Soil Description	Minimum Resistivity (ohm-cm)	pH	Sulfate Content (ppm)	Chloride Content (ppm)
B2	Bulk	0-5	Sandy Silt	870	9.6	423	161

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– LIQUEFACTION ANALYSIS

Garcrest Engineering and Construction, Inc.

Summary Sheet

Project Name	Paramount
Project No.	G23-009/1
Location	0
Boring	B-1
GW depth during test	44
Historic High GW depth	8
Design mag	6.6
Design Accel	0.55

Settlement due to dry seismic compaction	0.01
Settlement due to seismically induced liquefaction	1.78
Total seismically induced settlement	1.79

Garcrest Engineering and Construction, Inc.

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

Project Name	Paramount
Project No.	G23-009/1
Location	
Boring	B-1
Design mag	6.6
Design Accel	0.55
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)
0	8	120	25	35		8	4	480	1.54	0.75	51.95	0.993	74.63	320.0	170.326	1334935	0.0001276	0.0003	0.0001	7.952	0.00006	0.012
																						Total	0.012

Garcrest Engineering and Construction, Inc.

Seismically Induced Liquefaction Settlement (NCEER, 1997)

Project Name	Paramount
Project No.	G23-009/1
Location	0
Boring	B-1
GW depth during test	44
Historic High GW depth	8
Design mag	6.6
Design Accel	0.55
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 (%)		Effective Depth (feet)	Total Stress (psf)	Eff. Stress (test) (psf)	Eff Stress (Des) (psf)	Overburden Correction factor Cn	Rod Length Correction factor Cr	N160	alpha	Beta	N160 cs	Stress reduction Coefficient rd	CSR	CRR 7.5	MSF	FS	Remark	Volumetric strain (%)	Settlement (inches)
8	10	120	10	50		9	1080	1080	1017.6	1.29	0.75	17.33	5.000	1.200	25.80	0.981	0.372	0.309	1.39	1.149	not Liquefiable	0	0
10	14	120	11	35		12	1440	1440	1190.4	1.17	0.8	18.50	5.000	1.200	27.20	0.975	0.422	0.344	1.39	1.131	not Liquefiable	0	0
14	18	120	17	35		16	1920	1920	1420.8	1.04	0.85	27.11	5.000	1.200	37.53	0.966	0.467	HIGH	1.39	HIGH	not Liquefiable	0	0
18	23	120	17	22		20.5	2460	2460	1680	0.93	0.95	27.02	3.925	1.093	33.47	0.956	0.500	HIGH	1.39	HIGH	not Liquefiable	0	0
23	30	120	19	55		26.5	3180	3180	2025.6	0.81	0.95	26.40	5.000	1.200	36.68	0.936	0.525	HIGH	1.39	HIGH	not Liquefiable	0	0
30	35	120	18	35		32.5	3900	3900	2371.2	0.72	1	23.38	5.000	1.200	33.06	0.907	0.533	HIGH	1.39	HIGH	not Liquefiable	0	0
35	40	120	22	12		37.5	4500	4500	2659.2	0.66	1	26.14	1.554	1.032	28.52	0.872	0.527	0.389	1.39	1.023	Liquefiable	1	0.6
40	45	120	17	35		42.5	5100	5100	2947.2	0.61	1	18.61	5.000	1.200	27.34	0.828	0.512	0.348	1.39	0.943	Liquefiable	1	0.6
45	48	120	11	52		46.5	5580	5424	3177.6	0.58	1	11.55	5.000	1.200	18.87	0.788	0.495	0.202	1.39	0.565	Liquefiable	1.6	0.576
48	51.5	120	27	35		49.75	5970	5611.2	3364.8	0.57	1	27.71	5.000	1.200	38.25	0.755	0.479	HIGH	1.39	HIGH	not Liquefiable	0	0
																						Total	1.776

Garcrest Engineering and Construction, Inc.

Summary Sheet

Project Name	Paramount
Project No.	G23-009/1
Location	0
Boring	B-2
GW depth during test	48
Historic High GW depth	8
Design mag	6.6
Design Accel	0.55

Settlement due to dry seismic compaction	0.01
Settlement due to seismically induced liquefaction	0.90
Total seismically induced settlement	0.91

Garcrest Engineering and Construction, Inc.

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

Project Name	Paramount
Project No.	G23-009/1
Location	
Boring	B-2
Design mag	6.6
Design Accel	0.55
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)
0	8	120	25	35		8	4	480	1.54	0.75	51.95	0.993	74.63	320.0	170.326	1334935	0.0001276	0.0003	0.0001	7.952	0.00006	0.012
																						Total	0.012

Garcrest Engineering and Construction, Inc.

Seismically Induced Liquefaction Settlement (NCEER, 1997)

Project Name	Paramount
Project No.	G23-009/1
Location	0
Boring	B-2
GW depth during test	48
Historic High GW depth	8
Design mag	6.6
Design Accel	0.55
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 (%)		Effective Depth (feet)	Total Stress (psf)	Eff. Stress (test) (psf)	Eff Stress (Des) (psf)	Overburden Correction factor Cn	Rod Length Correction factor Cr	N160	alpha	Beta	N160 cs	Stress reduction Coefficient rd	CSR	CRR 7.5	MSF	FS	Remark	Volumetric strain (%)	Settlement (inches)
8	11.5	120	15	58		9.75	1170	1170	1060.8	1.26	0.75	25.37	5.000	1.200	35.44	0.980	0.386	HIGH	1.39	HIGH	not Liquefiable	0	0
11.5	19	120	22	35		15.25	1830	1830	1377.6	1.07	0.85	35.80	5.000	1.200	47.96	0.968	0.460	HIGH	1.39	HIGH	not Liquefiable	0	0
19	23	120	10	54		21	2520	2520	1708.8	0.92	0.95	15.71	5.000	1.200	23.85	0.954	0.503	0.271	1.39	0.746	Liquefiable	1.05	0.504
23	28	120	15	35		25.5	3060	3060	1968	0.83	0.95	21.29	5.000	1.200	30.55	0.940	0.523	HIGH	1.39	HIGH	not Liquefiable	0	0
28	33	120	20	54		30.5	3660	3660	2256	0.75	1	26.99	5.000	1.200	37.38	0.918	0.532	HIGH	1.39	HIGH	not Liquefiable	0	0
33	38	120	17	47		35.5	4260	4260	2544	0.68	1	20.91	5.000	1.200	30.10	0.887	0.531	HIGH	1.39	HIGH	not Liquefiable	0	0
38	44	120	28	56		41	4920	4920	2860.8	0.62	1	31.40	5.000	1.200	42.68	0.842	0.518	HIGH	1.39	HIGH	not Liquefiable	0	0
44	47	120	14	50		45.5	5460	5460	3120	0.58	1	14.64	5.000	1.200	22.57	0.798	0.499	0.250	1.39	0.695	Liquefiable	1.1	0.396
47	51.5	120	20	35		49.25	5910	5832	3336	0.56	1	19.98	5.000	1.200	28.98	0.760	0.481	0.409	1.39	1.179	not Liquefiable	0	0
																						Total	0.9

Garcrest Engineering and Construction, Inc.

Summary Sheet

Project Name	Paramount
Project No.	G23-009/1
Location	0

Boring	B-1
GW depth during test	44
Historic High GW depth	8
Design mag	6.8
Design Accel	0.83

Settlement due to dry seismic compaction	0.03
Settlement due to seismically induced liquefaction	2.51
Total seismically induced settlement	2.54

Garcrest Engineering and Construction, Inc.

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

Project Name	Paramount
Project No.	G23-009/1
Location	
Boring	B-1
Design mag	6.8
Design Accel	0.83
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)
0	8	120	25	35		8	4	480	1.54	0.75	51.95	0.993	74.63	320.0	257.038	1334935	0.0001925	0.0007	0.0002	9.340	0.00017	0.034
																					Total	0.034

Garcrest Engineering and Construction, Inc.

Seismically Induced Liquefaction Settlement (NCEER, 1997)

Project Name	Paramount
Project No.	G23-009/1
Location	0
Boring	B-1
GW depth during test	44
Historic High GW depth	8
Design mag	6.8
Design Accel	0.83
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 (%)		Effective Depth (feet)	Total Stress (psf)	Eff. Stress (test) (psf)	Eff Stress (Des) (psf)	Overburden Correction factor Cn	Rod Length Correction factor Cr	N160	alpha	Beta	N160 cs	Stress reduction Coefficient rd	CSR	CRR 7.5	MSF	FS	Remark	Volumetric strain (%)	Settlement (inches)
8	10	120	10	50		9	1080	1080	1017.6	1.29	0.75	17.33	5.000	1.200	25.80	0.981	0.562	0.309	1.28	0.706	Liquefiable	1.05	0.252
10	14	120	11	35		12	1440	1440	1190.4	1.17	0.8	18.50	5.000	1.200	27.20	0.975	0.636	0.344	1.28	0.695	Liquefiable	1	0.48
14	18	120	17	35		16	1920	1920	1420.8	1.04	0.85	27.11	5.000	1.200	37.53	0.966	0.705	HIGH	1.28	HIGH	not Liquefiable	0	0
18	23	120	17	22		20.5	2460	2460	1680	0.93	0.95	27.02	3.925	1.093	33.47	0.956	0.755	HIGH	1.28	HIGH	not Liquefiable	0	0
23	30	120	19	55		26.5	3180	3180	2025.6	0.81	0.95	26.40	5.000	1.200	36.68	0.936	0.793	HIGH	1.28	HIGH	not Liquefiable	0	0
30	35	120	18	35		32.5	3900	3900	2371.2	0.72	1	23.38	5.000	1.200	33.06	0.907	0.805	HIGH	1.28	HIGH	not Liquefiable	0	0
35	40	120	22	12		37.5	4500	4500	2659.2	0.66	1	26.14	1.554	1.032	28.52	0.872	0.796	0.389	1.28	0.628	Liquefiable	1	0.6
40	45	120	17	35		42.5	5100	5100	2947.2	0.61	1	18.61	5.000	1.200	27.34	0.828	0.773	0.348	1.28	0.579	Liquefiable	1	0.6
45	48	120	11	52		46.5	5580	5424	3177.6	0.58	1	11.55	5.000	1.200	18.87	0.788	0.747	0.202	1.28	0.347	Liquefiable	1.6	0.576
48	51.5	120	27	35		49.75	5970	5611.2	3364.8	0.57	1	27.71	5.000	1.200	38.25	0.755	0.723	HIGH	1.28	HIGH	not Liquefiable	0	0
																						Total	2.508

Garcrest Engineering and Construction, Inc.

<u>Summary Sheet</u>

Project Name	Paramount
Project No.	G23-009/1
Location	0
Boring	B-2
GW depth during test	48
Historic High GW depth	8
Design mag	6.8
Design Accel	0.83

Settlement due to dry seismic compaction	0.03
Settlement due to seismically induced liquefaction	1.44
Total seismically induced settlement	1.47

Garcrest Engineering and Construction, Inc.

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

Project Name	Paramount
Project No.	G23-009/1
Location	
Boring	B-2
Design mag	6.8
Design Accel	0.83
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)
0	8	120	25	35		8	4	480	1.54	0.75	51.95	0.993	74.63	320.0	257.038	1334935	0.0001925	0.0007	0.0002	9.340	0.00017	0.034
																					Total	0.034

Garcrest Engineering and Construction, Inc.

Seismically Induced Liquefaction Settlement (NCEER, 1997)

Project Name	Paramount
Project No.	G23-009/1
Location	0
Boring	B-2
GW depth during test	48
Historic High GW depth	8
Design mag	6.8
Design Accel	0.83
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 (%)		Effective Depth (feet)	Total Stress (psf)	Eff. Stress (test) (psf)	Eff Stress (Des) (psf)	Overburden Correction factor Cn	Rod Length Correction factor Cr	N160	alpha	Beta	N160 cs	Stress reduction Coefficient rd	CSR	CRR 7.5	MSF	FS	Remark	Volumetric strain (%)	Settlement (inches)
8	11.5	120	15	58		9.75	1170	1170	1060.8	1.26	0.75	25.37	5.000	1.200	35.44	0.980	0.583	HIGH	1.28	HIGH	not Liquefiable	0	0
11.5	19	120	22	35		15.25	1830	1830	1377.6	1.07	0.85	35.80	5.000	1.200	47.96	0.968	0.694	HIGH	1.28	HIGH	not Liquefiable	0	0
19	23	120	10	54		21	2520	2520	1708.8	0.92	0.95	15.71	5.000	1.200	23.85	0.954	0.759	0.271	1.28	0.458	Liquefiable	1.05	0.504
23	28	120	15	35		25.5	3060	3060	1968	0.83	0.95	21.29	5.000	1.200	30.55	0.940	0.789	HIGH	1.28	HIGH	not Liquefiable	0	0
28	33	120	20	54		30.5	3660	3660	2256	0.75	1	26.99	5.000	1.200	37.38	0.918	0.803	HIGH	1.28	HIGH	not Liquefiable	0	0
33	38	120	17	47		35.5	4260	4260	2544	0.68	1	20.91	5.000	1.200	30.10	0.887	0.801	HIGH	1.28	HIGH	not Liquefiable	0	0
38	44	120	28	56		41	4920	4920	2860.8	0.62	1	31.40	5.000	1.200	42.68	0.842	0.781	HIGH	1.28	HIGH	not Liquefiable	0	0
44	47	120	14	50		45.5	5460	5460	3120	0.58	1	14.64	5.000	1.200	22.57	0.798	0.754	0.250	1.28	0.427	Liquefiable	1.1	0.396
47	51.5	120	20	35		49.25	5910	5832	3336	0.56	1	19.98	5.000	1.200	28.98	0.760	0.726	0.409	1.28	0.724	Liquefiable	1	0.54
																							Total	1.44

APPENDIX D– PERCOLATION TESTING

Well B-5

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	16:55		1.00	-		9.0			
	17:00	15	6.00	5.00	60	4.0	6.5	0.25	4.00
2	17:02		2.00			8.0			
	17:07	15	6.00	4.00	48	4.0	6.0	0.31	3.20
3	17:08		3.50			6.5			
	17:13	15	7.00	3.50	42	3.0	4.8	0.36	2.80
4	17:15		3.20			6.8			
	17:20	15	6.00	2.80	33.6	4.0	5.4	0.45	2.24
5	17:21		3.20			6.8			
	17:26	15	6.00	2.80	33.6	4.0	5.4	0.45	2.24

Falling Head, Flow Rate, Q

$d=$ 8 in

$\Delta H_w=$ 33.6 in

$\Delta t=$ 15 min

$Q=$ 112.59 in^3/min

Wetted Surface Area

$H_w=$ 48 in

$A_{ws(1))}=$ 1206.37 in^2

$A_{ws(2)}=$ 50.27 in^2

$A_{ws(1)} + A_{ws(2)}=$ 1256.64 in^2

Infiltration Rate

IR= 0.090 in/min

IR= 5.38 in/hr

Well B-6

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	16:27		1.00	-		9.0			
	16:32	15	5.00	4.00	48	5.0	7.0	0.31	3.20
2	16:34		1.50			8.5			
	16:39	15	5.50	4.00	48	4.5	6.5	0.31	3.20
3	16:40		3.50			6.5			
	16:45	15	5.50	2.00	24	4.5	5.5	0.63	1.60
4	16:47		3.00			7.0			
	16:52	15	5.50	2.50	30	4.5	5.8	0.50	2.00
5	16:53		3.00			7.0			
	16:58	15	5.50	2.50	30	4.5	5.8	0.50	2.00

Falling Head, Flow Rate, Q

$d =$ 8 in
$\Delta H_w =$ 30 in
$\Delta t =$ 15 min
$Q =$ 100.53 in^3/min

Wetted Surface Area

$H_w =$ 54 in
$A_{ws(1))} =$ 1357.17 in^2
$A_{ws(2)} =$ 50.27 in^2
$A_{ws(1)} + A_{ws(2)} =$ 1407.43 in^2

Infiltration Rate

IR= 0.071 in/min
IR= 4.29 in/hr