GARCREST
ENGINEERING AND CONSTRUCTION, INC

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April 28, 2025

Mr. Henrik Nazarian
D&D Engineering, Inc
119 W Hyde Park Blvd
Inglewood, CA 90302

Subject: **Letter of Limited Geotechnical Investigation**
Proposed Buildings
700 Warren Lane
Inglewood, California
Project Number: G25-007/1

Dear Mr. Nazarian:

As requested, we are pleased to present the results of our limited geotechnical investigation to provide design recommendations for the proposed buildings located at the subject site.

SCOPE OF SERVICES

Our scope of services included the following items:

- Conducting a field investigation program to evaluate the subsurface soil conditions at the site,
- Laboratory testing to evaluate the strength properties of the onsite soils,
- Discussion of the subsurface soil conditions encountered,
- Discussion of site liquefaction potential,
- Recommendations for California Building Code (CBC) seismic factors;
- Recommendations for temporary excavations,
- Recommendations for foundation support and slab on grade,
- Preparation of this letter report.

A summary of our understanding of the project and our findings and conclusions are summarized in the following sections.

PROJECT AND SITE DESCRIPTION

We understand that following the demolition of the existing shed type structures located at the site, new modular type buildings are proposed for the project site. The proposed buildings will be single story, lightly loaded structures. One of the buildings (workshop) will be located along the north side of the site, along Warren Lane, while other office buildings will be located along the center east, and along the south side of the site.

The site is a City maintenance yard located at 700 Warren Lane in Inglewood California. The site is currently occupied by single story maintenance buildings along the north, center, and south, consisting of equipment shed structures and a small office building.

The site of the buildings and improvements are located near the northwest corner of Edward Vincent Park and is bounded by Warren Lane to the north and the remainder of the park to the south, west, and east. A gated access drive is located along the east of the subject site, providing access from Warren Lane, with additional gate entries to the site. The site is located on a southerly descending topography and consists of two tiers of level area, separated by stairs and landscaping. The grade difference between the two tiers is approximately 5 feet. Various utilities may cross the site.

The approximate location of the site is shown on Plate 1, Site Location Map. The approximate locations of the proposed buildings along with our field exploration location are shown on Plate 2, Plot Plan.

FIELD INVESTIGATION AND LABORATORY TESTING

We performed four borings at the site of the proposed improvements. The borings were performed using an 8-inch diameter, truck-mounted hollow stem auger type equipment. The borings were performed to a depth of approximately 50½ feet below existing grade. The approximate locations of the borings are presented on Plate 2, Plot Plan. The borings were logged by our field representative and relatively undisturbed samples were collected, sealed and transported to our office for further evaluation. The borings were backfilled using the excavated soil cuttings and tamped. The log of boring is presented on Plates 3 through 6, Log of Boring.

Laboratory testing was performed on representative undisturbed samples to obtain pertinent engineering characteristics of the subsurface soils. AP Engineering and Testing, Inc. was retained for performing the laboratory testing. All testing was performed in accordance with applicable ASTM methods. The testing included the following:

- Direct Shear Test
- Consolidation
- InSitu Moisture and Density Test
- Maximum Density/ Optimum Moisture
- Preliminary Corrosivity Test

The results of the laboratory testing performed are attached. We have reviewed, concur, and accept the results of the tests performed.

SUBSURFACE CONDITIONS

Fill soils were encountered to a depth of approximately 2½ feet below grade in our boring. Deeper fill may exist between and beyond our boring. The fill soils consist generally of sandy silt and clayey sand soils.

The fill soils are underlain by the onsite native soils which generally consist of dense to very dense clayey sands and stiff to very stiff sandy silts, and dense sand soils.

Groundwater was not encountered in our borings to the depth explored. According to the State, (CGS, 1998), historical high groundwater at the site is estimated to be on the order of 10 to 15 feet below grade, a clearer depth is not easily available.

LIQUEFACTION AND SEISMICALLY SETTLEMENT

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

As previously stated, groundwater was not encountered within our boring to the depth explored. The site is located within a State of California designated liquefaction hazard zone and historical high groundwater at the site is roughly estimated at a depth of between 10 to 15 feet below grade (CGS 1998).

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 the PGA_M for the site per the "life safety requirements" of the California Building Code. The result of the evaluation is summarized as a Magnitude 6.7 and a ground acceleration of 0.92g. 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 Maximum Considered earthquake (MCE). 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 herein.

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).

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 negligible.

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.

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RECOMMENDATIONS

General

Based on the results of our field investigation and laboratory testing, it is our professional opinion that the subject project is feasible from a geotechnical standpoint provided the recommendations presented herein are incorporated into the project design and construction.

Fill soils were encountered at the site to a depth of approximately 2½ feet below grade within the boring performed at the site. The fill soils at the site consist of sandy silt and clayey sand type soils. The fill soils are underlain by the onsite native soils to the depth explored. The native soils consist of stiff to very stiff sandy silts and dense to very dense clayey sands and sands.

Based on the results of our investigation, the onsite fill soils are not considered suitable for the support of the proposed modular building foundations. The proposed buildings may be supported on shallow spread foundations deepened and established in the firm and unyielding native soils at the site. An allowable bearing capacity of 2,500 pounds per square foot may be used for design.

The onsite fill soils are not considered suitable for the support of proposed slabs on grade in their current state. We recommend that new slabs on grade be supported on at least 12-inches of properly compacted engineered fill as recommended herein. The proposed slab on grade may be supported on the properly compacted engineered fill.

Minor Earthwork

<u>Site Preparation</u>

As discussed, the onsite, fill soils are not considered suitable for the support of the proposed modular buildings and slab on grade in their current condition. Building foundations should be deepened to the native soils and supported on the firm unyielding natural soils.

For the support of slabs on grade, we recommend that onsite fill soils be overexcavated to the firm and unyielding native soils or at least 12-inches below the bottom of the slab section, whichever is deeper, and recompacted as properly compacted engineered fill. The 12-inches are in addition to the capillary break thickness of the proposed slab, if required.

For support of pavement sections at the site, we recommend that the onsite fill soils be overexcavated to the firm and unyielding native soils or at least 2 feet below grade, whichever is deeper and recompacted as properly compacted engineered fill.

Following the removal of the upper fill soils, the exposed subgrade should be inspected by a Garcrest representative for unsuitable and deleterious materials, and deepened as necessary. The exposed subgrade should be scarified to a depth of 6-inches, brought to within 2 percent above the optimum moisture content and compacted to a minimum of 90 percent relative compaction as obtainable by ASTM Designation D-1557.

Following the subgrade preparation, fill soils as required may be placed and compacted as discussed below to the finish subgrade level.

<u>Material for Fill</u>

The onsite soils less any debris, organic matter, and oversize (greater than 4-inches in diameter) material, may be used as fill soils. Import fill should have a minimum sand equivalent of 30 and an expansion index of less than 20. The import soils should contain enough fines to provide a stable subgrade and maintain a low to medium permeability. All import materials should be approved by our personnel prior to import onto the site.

<u>Compaction</u>

Fill soils should be placed in loose lifts of 8-inches or less, brought to within 2 percent above the optimum moisture content and compacted to a minimum 90 percent relative compaction.

Excavations and Temporary Slopes

Excavations deeper than 4 feet should be sloped 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 footings.

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

All excavation, including temporary excavations and excavations in hillside areas, are inherently dangerous construction activities and should be performed by a licensed professional, experienced in that nature of work, with the presence of a competent person at the site at all times. Care should always be exercised to create a safe working environment at the construction site. 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.

Seismic Considerations

The site is located within the seismically active Southern California region. As a minimum, we recommend that the proposed structures 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 2022 edition of California Building Code (CBC), which is based on the 2021 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 1.932 and 0.68 respectively. Site coefficients Fa and Fv of 1.0 and 1.7 respectively, may be used for the calculation of the spectral response accelerations, however given that S1 is greater than 0.2, based on ASCE 7-16 (Section 11.4.8), a site response analysis may be required. With the above coefficients however, spectral response accelerations $\mathbf{S_{MS}}$ and $\mathbf{S_{M1}}$ of 1.932 g and 1.156g and $\mathbf{S_{DS}}$ and $\mathbf{S_{D1}}$ of 1.288 g and 0.770g may be used for a Site Class D.

Foundations

<u>Bearing Value</u>

The proposed modular building foundations may be supported on spread footings deepened and established in the firm and unyielding native soils at the site. Continuous walls should not be underlain by a fill/native transition.

Foundations should be at least 18-inches wide and established at least 36-inches below the lowest adjacent grade. Foundations sized as recommended above may be designed for a net dead-plus-live allowable pressure of 2,500 pounds per square feet. A one-third increase may be used for wind and seismic loading conditions.

The weight of concrete in the footings may be taken as 50 pounds per cubic foot and the weight of the soil backfill may be neglected when determining the downward loads.

Lateral Resistance

Resistance to lateral loads may be provided by friction between the soil and the foundation and passive resistance of the soil against the vertical face of the foundation. A coefficient of friction of 0.4 may be used between the footing and the underlying soil materials. The passive resistance of the onsite soil material may be taken as equivalent to the pressure developed by a fluid with a density of 350 pounds per cubic foot. A one-third increase may be used for wind and seismic loading conditions and the passive and friction values may be combined without reduction.

Settlement

Based on the type of materials at the site and anticipated foundation loads, we anticipate the static total and differential foundations settlements to be less than ½- inch and ¼-inch, respectively for spread foundations supported as recommended above.

The onsite materials are granular in nature and the majority of the settlement is anticipated to occur during the construction phase.

Slab-on-grade

Prior to placement of slabs on grade, the subgrade beneath slabs on grade should be prepared as recommended in the Earthwork section of this report.

Following the preparation of the subgrade as recommended above, the concrete floor slab may be supported on grade. The concrete slab should be at least 5-inches thick and be designed by a structural engineer for the minimum reinforcement requirements. We recommend minimum reinforcement of No.4 at 18-inches on center for the design of the slab.

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

The proposed floor slab on grade may be designed for a modulus of subgrade reaction of 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.

If hardwood flooring over slab-on-grade is considered, we recommend that a competent hardwood flooring installer be consulted for installation procedures or care requirement specific to the type of hardwood flooring used.

It should be noted that the abovementioned 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.

Walls Below Grade

Lateral Pressure

Although not anticipated, if minor retaining walls are required for the project, for the design of cantilevered walls below grade with a level backfill, an active equivalent fluid pressure of 35 pounds per cubic foot may be used for design. For use of basement conditions, if required by the structural engineer, an equivalent fluid pressure of 50 pounds per cubic foot may be used for at-rest conditions. Where the surface of the retained earth slopes up away from the wall, a greater pressure may be required.

The recommended lateral earth pressures assume that drainage is provided behind walls below grade to prevent accumulation of hydrostatic pressures. Alternately the walls should be designed for hydrostatic pressures.

The design of walls below grade should include any surcharge imposed by the footings of any adjacent structure within a distance of 10 feet behind the wall. This surcharge may be neglected if foundations are beyond 10 feet behind the wall.

For seismic purposes, an additional lateral earth pressure should be used during design for walls greater than 6 feet in height. The pressure distribution may be considered to be an inverted triangle with a maximum pressure at the top and zero at the bottom. The resultant of this distribution may be assumed to be at 2/3 the height of the wall from the bottom of the wall. A maximum pressure of 20H pounds per square foot may be used, where H is the retained height.

Backfill

All required soil backfill should be mechanically compacted to a minimum of 90 percent relative compaction. Soil backfill should be placed in loose lifts of 8-inches or less, brought to within 2 percent above the optimum moisture content, and compacted with mechanical equipment. Jetting or flooding is not permitted. The onsite soils, less any debris or organic and deleterious materials may be used for backfill.

Drainage

The recommended lateral earth pressures assume that drainage is provided behind walls below grade to prevent accumulation of hydrostatic pressures. Walls below grade should be provided with backdrains to reduce the potential for the accumulation of hydrostatic pressures. Backdrains may consist of a 2-foot wide zone of Caltrans Class 2 permeable material wrapped in a non-woven geofabric or Miradrain 6000 (or equivalent) located immediately behind the wall, extending to within 1 foot of the ground surface. A perforated pipe (Schedule 40 PVC) should be installed at the base of the backdrain and sloped to discharge to a suitable collection facility. A slope of at least 2-inches per 100 feet should be used for the drain lines. Prior to construction of the wall backdrain system, the back of the retaining wall should be adequately waterproofed to reduce the potential for migration of water through the retaining wall.

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 one boring at the site. Borings B-4 was drilled to a depth of 10 feet and percolation testing was performed directly in the boring. 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 overnight prior to performing the testing the following day. Depth readings were taken every 30 minutes for a period of approximately 2½-hour 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 herein and summarized in the following table.

The measured percolation rate as well as the rate reductions are 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.	Field Percolation Rate (inch/hr)	Adjusted Percolation Rate (inch/hr)
B-4	4.1	0.1

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.

It is our professional opinion that percolation rates as measured in our borings and adjusted to between 0.1 inch/hr in accordance with the GS200.1 method, may be considered relatively representative of the overall conditions at the site. These rates have not been factored for design purposes but include sidewall reductions for borehole testing.

Groundwater was not encountered in the borings within the depth explored. According to the State (CGS, 1998), historical high groundwater is anticipated to be approximately 10 to 15 feet below grade.

Given the very low rates of percolation, infiltration at the site may not be practically feasible.

If however, infiltration is considered, then infiltration devices may consist of excavated pits or trenches to depths and size as needed for design capacity. The devices may be backfilled with granular material conforming to the requirements of Class 2 Permeable Base Material as defined by the most current State Specifications or crushed rock material between ¾- to 1-inch open graded material. The use of recycled material is not permitted. The base or rock materials should be surrounded by non-woven filter fabric to reduce the potential of fines migration into the device. Prefabricated devices should also be surrounded by base or rock material wrapped in filter fabric. Adequate overflow capacities should be incorporated into the design of the proposed devices. Infiltration devices considered for the proposed project should be installed a distance of at least 15 feet from proposed or existing foundations

<u>Additional Discussions</u>

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 10 to 15 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 dry seismically induced settlements, with estimated settlements of less than ¼-inch. Seismically induced liquefaction settlements are estimated to be negligible. 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 15 feet. The design of the proposed devices should include consideration for flexible connections in the event of localized settlement.

Perched Water Conditions

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

The onsite soils are generally 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. However the relatively low rates of percolation, may render infiltration less effective.

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.

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 15 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	9
Truck Traffic	6	3½	11
Heavy Truck Traffic	7	4	13

Portland Cement Concrete Pavement

Traffic Usage	Traffic Index	Portland Cement Concrete (inches)	Base Course (inches)
Automobile Parking Areas	4	7	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 2 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.

Site Drainage

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

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

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 attached to this letter report. Preliminary corrosivity testing indicates that the soils have a moderate corrosivity potential to buried ferrous metals and a mild potential to buried concrete structures. Based on the preliminary corrosivity results, concrete structures should comply with cement type, minimum compressive strength, and minimum water/cement ratio requirements as specified in ACI guidelines 318, Section 4.3.

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

LIMITATIONS

The findings, conclusions, and recommendations presented herein have been performed in general accordance with generally accepted geotechnical engineering practice. No other warranty, express or implied is made.

This report may only be used by the client and only for the scope of work outlined herein and within a limited time frame. Changes of use for the subject site may change over time and may result in the need to perform additional services in order to update the findings and conclusions presented herein. The unauthorized use of this report and non-compliance with the above by any party including the client releases the Consultant from any liability resulting from this use.

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.

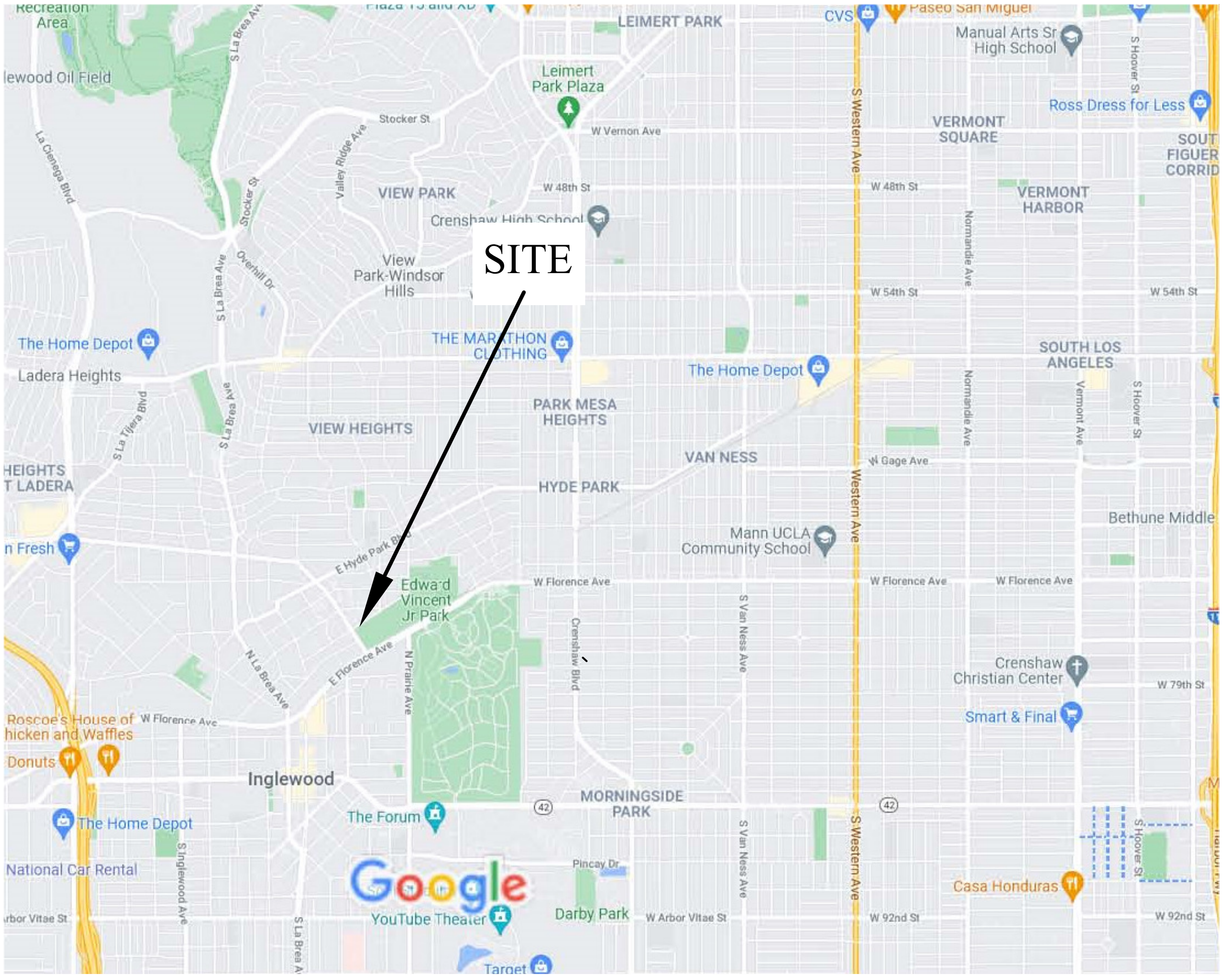
Respectfully submitted,
GARCREST Engineering and Construction, Inc.

REGISTERED PROFESSIONAL ENGINEER
ARMEN GAPRELIAN
No. 2668
Exp. 12/31/26
GEOTECHNICAL
STATE OF CALIFORNIA

Armen Gaprelian, PE, GE
Principal Engineer

Path: F:\GARCREST\Projects\2025 Projects\G25-007.1 - Inglewood Park 2\Inglewood 2 Letter.doc

Enclosures: Plate 1 – Site Location Map
Plate 2 – Plot Plan
Plate 3 through 6 – Log of Boring
Laboratory Test Results
Results of Liquefaction Analysis
Results of Percolation Testing

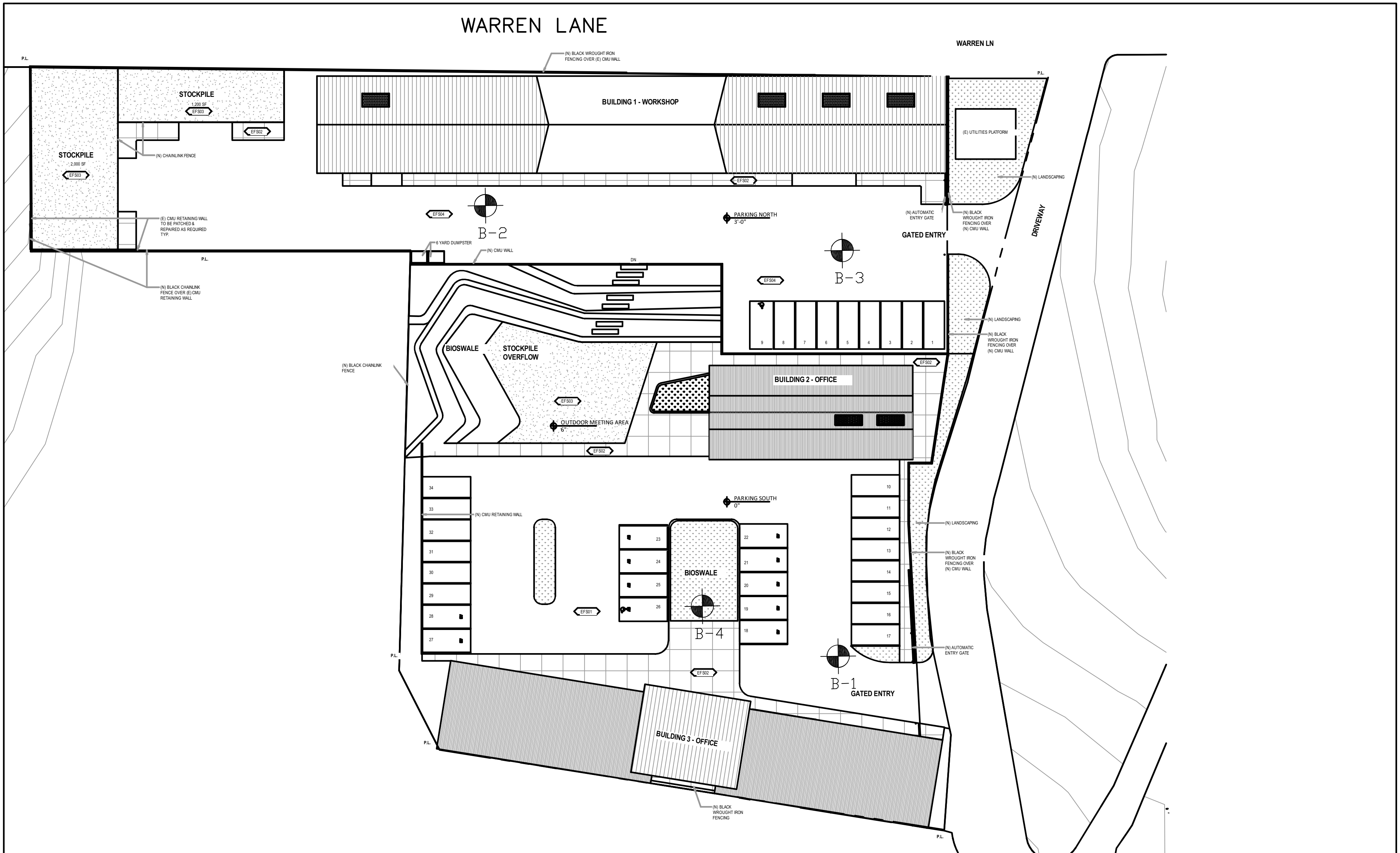


SCALE: Not to Scale

REFERENCE: GoogleMaps (2025)

GARCREST ENGINEERING AND CONSTRUCTION, INC.	**SITE LOCATION MAP** Proposed Buildingss 700 Warren Lane, Inglewood, California Project No. G25-007/1	PLATE 1

NORTH



KEY — Approximate Boring Location

REFERENCE: Base plan provided by D&D Engineering, Inc.

SCALE

1 50

NORTH

GARCREST
ENGINEERING AND CONSTRUCTION, INC.

PLOT PLAN

Proposed Building s
700 Warren Lane, Inglewood, California
Project No. G25-007/1

PLATE
2

Garcrest Engineering & Construction, Inc.
LOG OF BORING

PROJECT NO.: G25-007/1
PROJECT NAME: Inglewood Park
LOCATION: 700 Warren Lane, Inglewood, CA
ELEVATION:

DRILLER: Choice Drilling
DRILL METHOD: 8" Hollow Stem Auger
HAMMER: 140 pound Auto/30 inches

LOGGED BY: BC
OPERATOR: James/Toby
RIG TYPE: CME 75
DATE: 3/20/2025

BORING NO.: *B-1*

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
							6-inch asphalt, 4-inch base course			
	Bulk					SC	**FILL** **CLAYEY SAND -** fine, brown, moist			
						SC	**NATIVE** **CLAYEY SAND -** fine, some gravel, brown to dark brown, fine, moist dense			CORR MAX
5	Ring	10 13 16	29	1				12.5	125	DS
10	Ring	7 13 15	28	2			-- Some Silt, very fine to fine	15.9	117	CONS
						SP	**SAND** - brown, fine to coarse, slightly moist, dense			
15	SPT	7 16 21	37	3						
20	Ring	25 50/5"		4						
25	SPT	12 14 22	36	5						

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

chk: AG 04/16/25

PLATE 3

Garcrest Engineering & Construction, Inc.
LOG OF BORING

PROJECT NO.:	G25-007/1	**DRILLER:**	Choice Drilling	**LOGGED BY:**	BC
PROJECT NAME:	Inglewood Park	**DRILL METHOD:**	8" Hollow Stem Auger	**OPERATOR:**	James/Toby
LOCATION:	700 Warren Lane, Inglewood, CA	**HAMMER:**	140 pound Auto/30 inches	**RIG TYPE:**	CME 75
ELEVATION:				**DATE:**	3/20/2025

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	SPT	7 19 21	40	9		**SP**	**SAND** - brown, fine to coarse, moist, dense			
35	SPT	6 18 17	35	11						
40	SPT	5 15 18	33	12						
						SM	**SILTY SAND -** Brown, fine, moist, dense			
45	SPT	8 20 21	41	13						
50	SPT	10 12 18	30	14						
55							**NOTES:** BORING TERMINATED AT 51.5 feet. No Groundwater Encountered Boring backfilled with cuttings and asphalt patched			

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

chk: AG 04/16/25

PLATE A-2b

Garcrest Engineering & Construction, Inc.
LOG OF BORING

PROJECT NO.:	G25-007/1	**DRILLER:**	Choice Drilling	**LOGGED BY:**	BC
PROJECT NAME:	Inglewood Park	**DRILL METHOD:**	8" Hollow Stem Auger	**OPERATOR:**	James/Toby
LOCATION:	700 Warren Lane, Inglewood, CA	**HAMMER:**	140 pound Auto/30 inches	**RIG TYPE:**	CME 75
ELEVATION:				**DATE:**	3/20/2025

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
0							**4-inch asphalt, 3-inch base course**			
						ML	**FILL** **SANDY SILT -** bown, moist			
						ML	**NATIVE** **SANDY SILT -** same Clay, some gravel, brown, moist, very stiff to hard			
5	Ring	9 16 23	39	1				12.9	120.0	DS
10	SPT	7 11 16	27	2		**SM**	**SILTY SAND** - fine, some gravel, moist, brown, medium dense			
						SP	**SAND** - some gravel, light brown to brown, moist, very dense			
15	Ring	25 50-5"		3				6.9	128	
20	SPT	9 17 24	41	4			-- dense			
25							**NOTES:** BORING TERMINATED AT 21.5 feet. No Groundwater Encountered Boring backfilled with cuttings and asphalt patched			

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

chk: AG 04/16/25

PLATE 4

Garcrest Engineering & Construction, Inc.
LOG OF BORING

PROJECT NO.:	G25-007/1	**DRILLER:**	Choice Drilling	**LOGGED BY:**	BC
PROJECT NAME:	Inglewood Park	**DRILL METHOD:**	8" Hollow Stem Auger	**OPERATOR:**	James/Toby
LOCATION:	700 Warren Lane, Inglewood, CA	**HAMMER:**	140 pound Auto/30 inches	**RIG TYPE:**	CME 75
ELEVATION:				**DATE:**	3/20/2025

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
							5-inch asphalt, 7-inch base			
						ML	**FILL** **SANDY SILT -** bown, moist			
						ML	**NATIVE** **SANDY SILT -** same Clay, some gravel, brown, moist, stiff to very stiff			
5		4 6 9	15	1						
						SM	**SILTY SAND** - fine, some gravel, moist, brown, medium dense			
10		9 7 10	17	2						
15		9 16 18	34	3		SP	**SAND** - fine to coarse, some gravel, slightly moist, light brown to brown dense			
20		33 50-4"		4			-- dry to slightly moist, dense, fine to very fine gravel, slightly silty	1.8	121.0	
25		7 17 50	67	5						

NOTES:
BORING TERMINATED AT 26.5 feet.
No Groundwater Encountered
Boring backfilled with cuttings and patched

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

chk: AG 04/16/25

PLATE 5

Garcrest Engineering & Construction, Inc.
LOG OF BORING

PROJECT NO.:	G25-007/1	**DRILLER:**	Choice Drilling	**LOGGED BY:**	BC
PROJECT NAME:	Inglewood Park	**DRILL METHOD:**	8" Hollow Stem Auger	**OPERATOR:**	James/Toby
LOCATION:	700 Warren Lane, Inglewood, CA	**HAMMER:**	140 pound Auto/30 inches	**RIG TYPE:**	CME 75
ELEVATION:				**DATE:**	3/20/2025

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
							3-inch asphalt, 4-inch base			
						SC	**FILL** **CLAYEY SAND -** fine, brown, moist			
						SC	**NATIVE** **CLAYEY SAND -** fine, some gravel, brown to dark brown, fine, moist very dense			
5	Ring	15 25 50	75	1				7.3	111	
10	Ring	15 17 23	40	2			-- dense			
15										
20										
25										

NOTES:
BORING TERMINATED AT 11.5 feet.
No Groundwater Encountered
Boring converted to percolation hole with percolation testing performed between 5 to 10 feet. Following test, pipe was removed and boring backfilled with cuttings and asphalt patched

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

chk: AG 04/16/25

PLATE 6

April 1, 2025

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: Inglewood Park
Project No.: G25-007/1

Dear Armen,

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

- 3 Moisture Content and Density (ASTM D 2216 & D 7263)
- 2 Direct Shear (ASTM D 3080)
- 1 Corrosion Suite (CTM 417, 422 & 643)
- 1 Modified Proctor Compaction (ASTM D 1557)
- 1 Consolidation (ASTM D 2435)

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

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

Respectfully submitted,

AP Engineering and Testing, Inc.
Certificate No. 10130

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

REGISTERED PROFESSIONAL ENGINEER
APICHART PHUKUNHAPHAN
No. 002156
Exp. 12/31/25
GEOTECHNICAL
STATE OF CALIFORNIA

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: Inglewood Park
Project No.: G25-007/1

AP Lab No.: 25-0353
Test Date: 03/26/25

Boring No.	Sample No.	Sample Depth (ft.)	Moisture Content (%)	Dry Density (pcf)
B2	-	15	6.9	127.6
B3	-	20	1.8	120.9
B4	-	5	7.3	110.8

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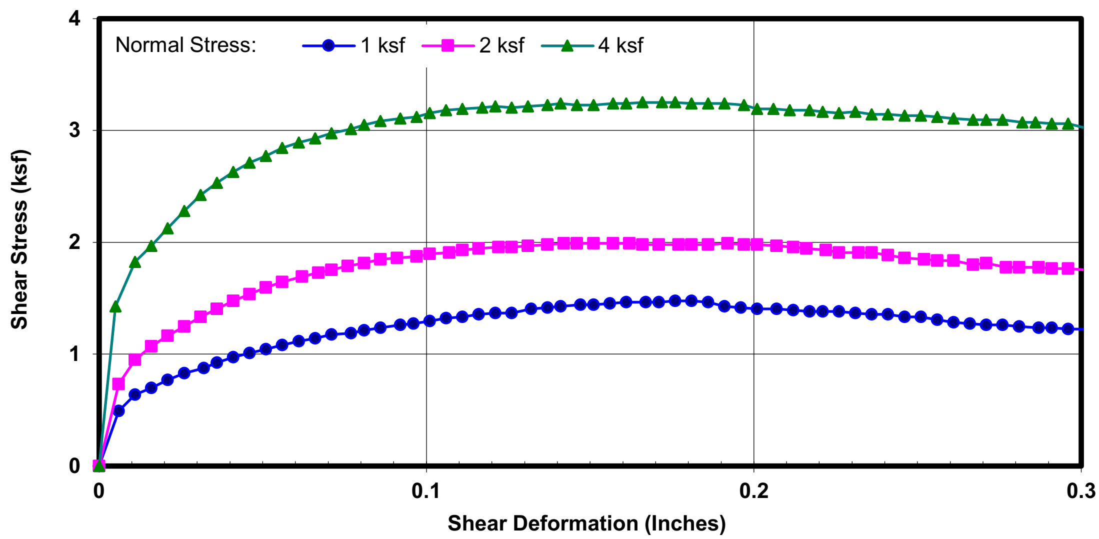
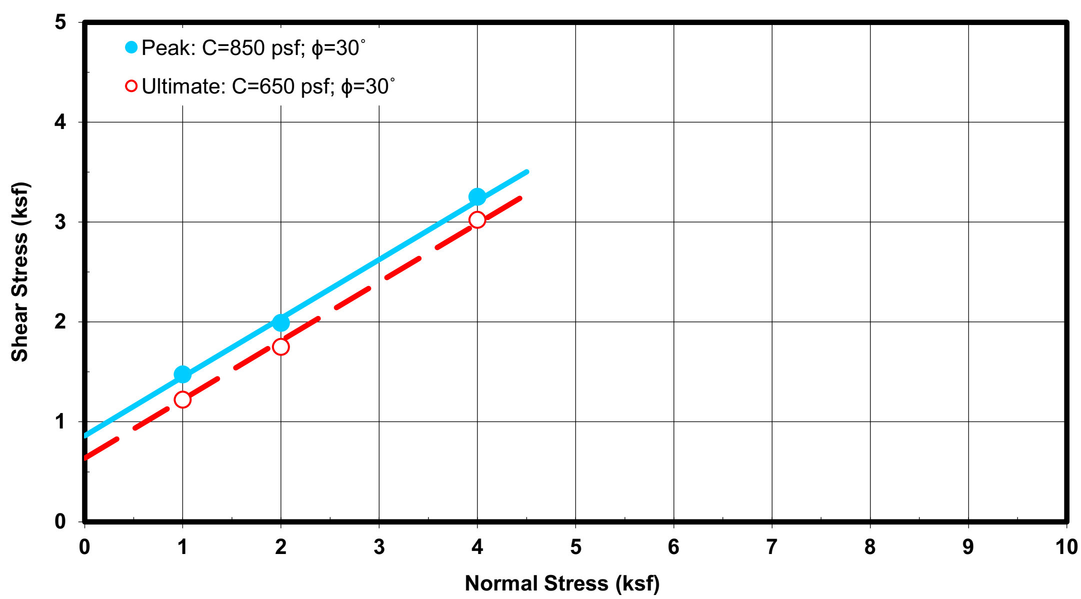
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name: Inglewood Park — **Tested By:** ST — **Date:** 03/28/25
Project No.: G25-007/1 — **Computed By:** JG — **Date:** 03/31/25
Boring No.: B1 — **Checked by:** AP — **Date:** 04/01/25
Sample No.: - — **Depth (ft):** 5
Sample Type: Mod. Cal.
Soil Description: Sandy Clay
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)
140.1	124.5	12.5	13.1	96	100	1	1.476	1.224
						2	1.992	1.752
						4	3.252	3.024





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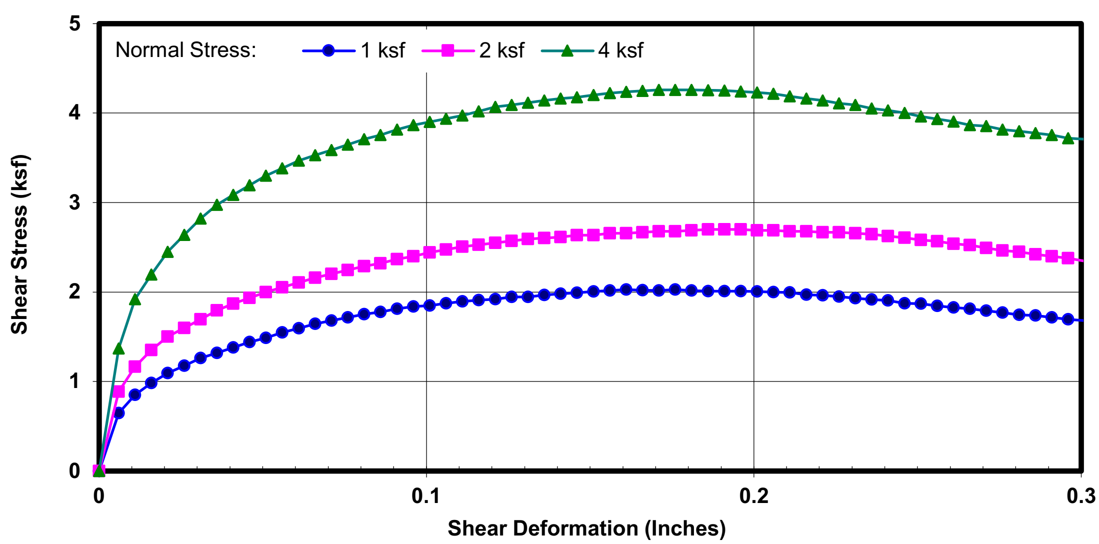
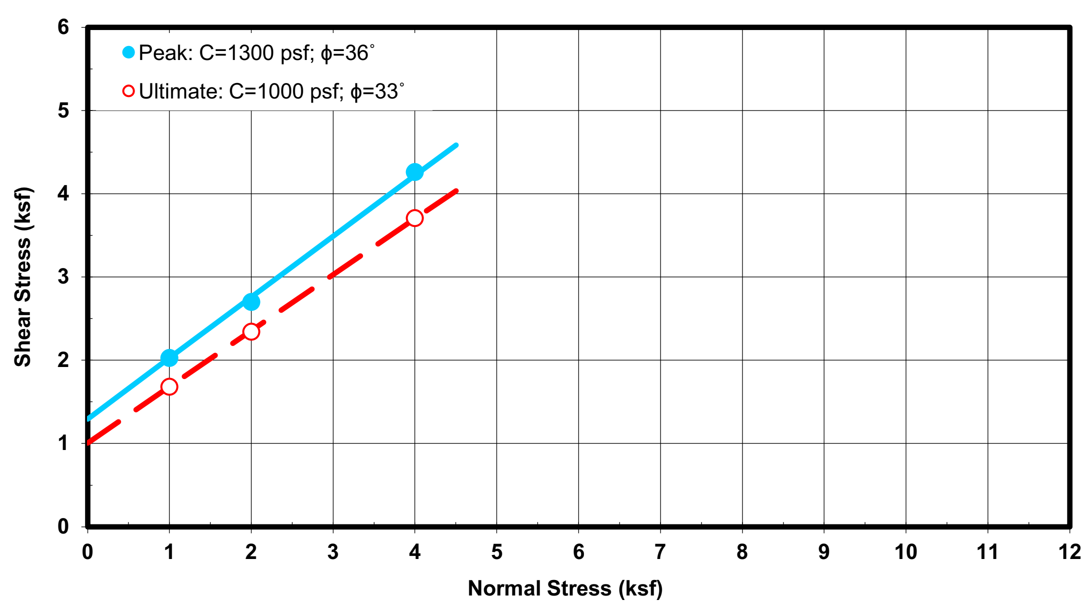
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name: Inglewood Park
Project No.: G25-007/1
Boring No.: B2
Sample No.: - **Depth (ft):** 5
Sample Type: Mod. Cal.
Soil Description: Sandy Clay
Test Condition: Inundated **Shear Type:** Regular

Tested By: ST **Date:** 03/29/25
Computed By: JG **Date:** 04/01/25
Checked by: AP **Date:** 04/01/25

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)
135.7	120.3	12.9	14.7	86	99	1	2.028	1.681
						2	2.700	2.344
						4	4.260	3.708





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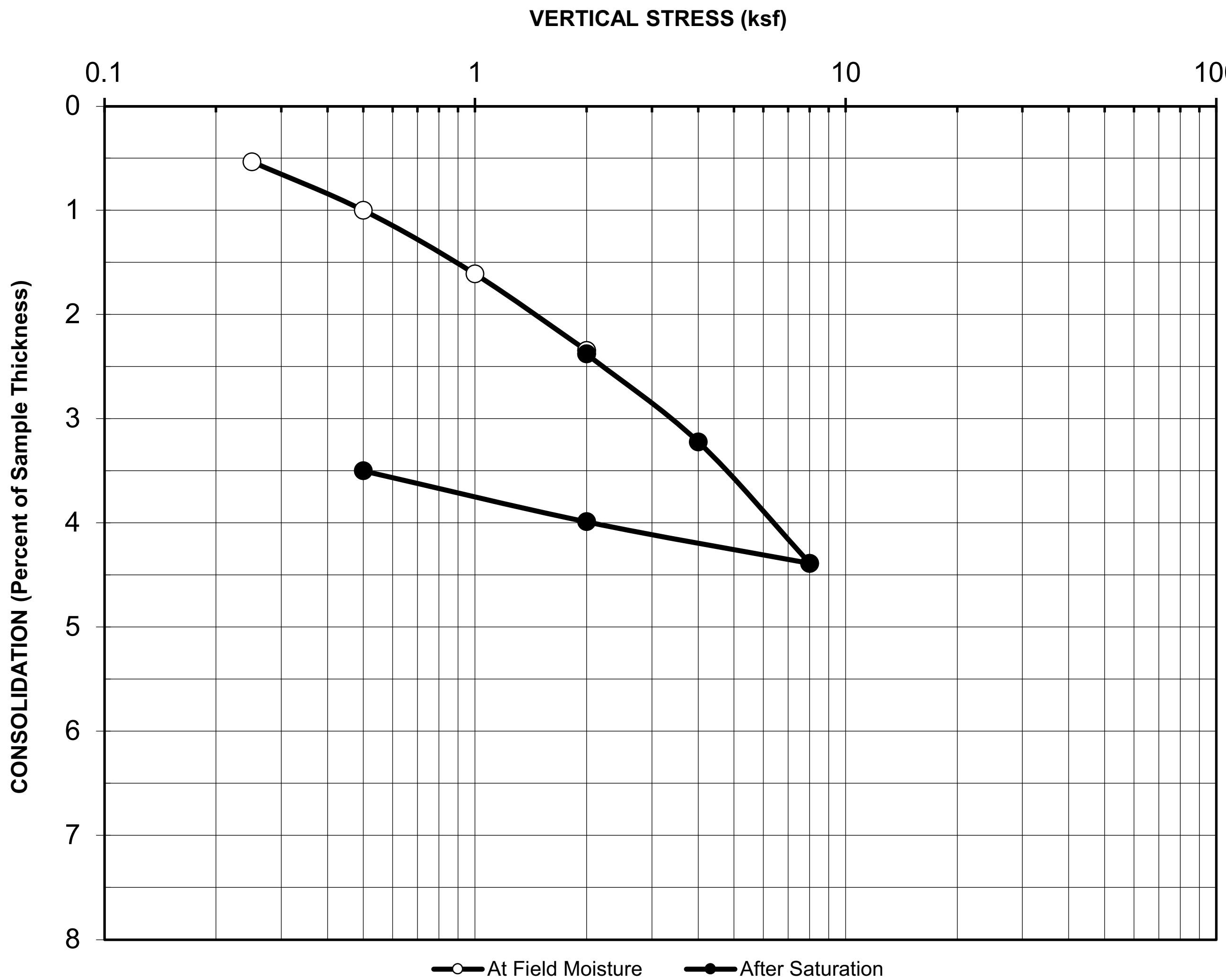

Boring No. :	B1	Initial Dry Unit Weight (pcf):	117.3
Sample No.:	-	Initial Moisture Content (%):	15.9
Depth (feet):	10	Final Moisture Content (%):	16.0
Sample Type:	Mod Cal	Assumed Specific Gravity:	2.7
Soil Description:	Sandy Clay	Initial Void Ratio:	0.44
Remarks:	Collapse= 0.03% upon inundation		

CONSOLIDATION CURVE
ASTM D 2435

Project Name: Inglewood Park
Project No.: G25-007/1
Date: 3/26/2025
AP No: 25-0353 **Sheet No:** 1

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COMPACTION TEST

Client:	Garcrest Engineering			AP Number:	25-0353
Project Name:	Inglewood Park	Tested By:	TV	Date:	03/31/25
Project No. :	G25-007/1	Calculated By:	JG	Date:	04/01/25
Boring No.:	B1	Checked By:	AP	Date:	04/01/25
Sample No.:	-	Depth (ft.):	0-5		

Visual Sample Description: Clayey Sand

Compaction Method: [X] ASTM D1557 [ ] ASTM D698

Preparation Method: [ ] Moist [X] Dry

METHOD: A

MOLD VOLUME (CU.FT): 0.0333

Wt. Comp. Soil + Mold (gm.)	3884	3969	3918	3764		
Wt. of Mold (gm.)	1835	1835	1835	1835		
Net Wt. of Soil (gm.)	2049	2134	2083	1929		
Container No.						
Wt. of Container (gm.)	151.78	135.61	149.15	142.88		
Wet Wt. of Soil + Cont. (gm.)	732.83	725.05	721.03	714.50		
Dry Wt. of Soil + Cont. (gm.)	695.30	674.01	660.46	688.75		
Moisture Content (%)	6.90	9.48	11.85	4.72		
Wet Density (pcf)	135.52	141.10	137.73	127.55		
Dry Density (pcf)	126.76	128.89	123.14	121.80		

Maximum Dry Density (pcf): **129.1**
Maximum Dry Density w/ Rock Correction (pcf): **N/A**

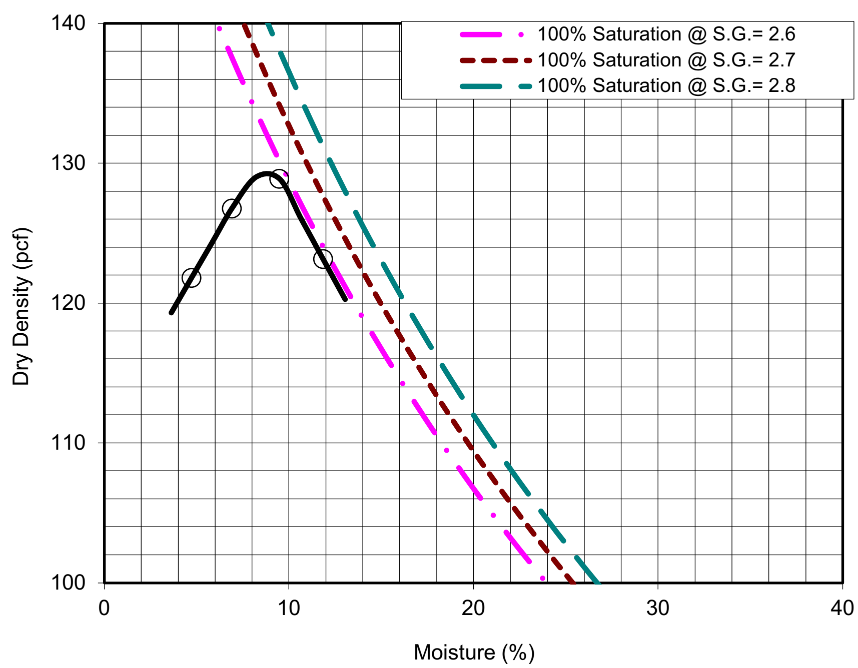
Optimum Moisture Content (%): **8.8**
Optimum Moisture Content w/ Rock Correction (%): **N/A**

PROCEDURE USED

[X] *METHOD A: Percent of Oversize:* *1.4%*
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)

[ ] *METHOD B: Percent of Oversize:* *N/A*
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)

[ ] *METHOD C: Percent of Oversize:* *N/A*
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)



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CORROSION TEST RESULTS

Client Name: Garcrest Engineering
Project Name: Inglewood Park
Project No.: G25-007/1

AP Job No.: 25-0353
Date: 03/31/25

Boring No.	Sample No.	Depth (feet)	Soil Description	Minimum Resistivity (ohm-cm)	pH	Sulfate Content (ppm)	Chloride Content (ppm)
B1	-	0-5	Clayey Sand	3,799	7.6	9	5

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

Garcrest Engineering and Construction, Inc.

Summary Sheet

Project Name	Inglewood Warren
Project No.	G25-007/1
Location	Inglewood CA
Boring	B-1
GW depth during test	100
Historic High GW depth	10
Design mag	6.7
Design Accel	0.92

Settlement due to dry seismic compaction	0.15
Settlement due to seismically induced liquefaction	0.00
Total seismically induced settlement	0.15

Garcrest Engineering and Construction, Inc.

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

Project Name	Inglewood Warren
Project No.	G25-007/1
Location	Inglewood CA
Boring	B-1
Design mag	6.7
Design Accel	0.92
Hammer Energy Ce	1.25
Borehole diameter Cb	1
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	7	120	19	25		7	3.5	420	1.57	0.75	35.03	0.994	65.44	280.0	249.594	1094986	0.0002279	0.0010	0.0005	8.631	0.00040	0.067
7	10	120	19	25		3	8.5	1020	1.31	0.75	29.12	0.982	61.53	680.0	599.197	1604568	0.0003734	0.0023	0.0015	8.631	0.00114	0.082
																					Total	0.149

Garcrest Engineering and Construction, Inc.

Seismically Induced Liquefaction Settlement (NCEER, 1997)

Project Name	Inglewood Warren
Project No.	G25-007/1
Location	Inglewood CA
Boring	B-1
GW depth during test	100
Historic High GW depth	10
Design mag	6.7
Design Accel	0.92
Hammer Energy Ce	1.25
Borehole diameter Cb	1
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)
10	12	120	19	25		11	1320	1320	1257.6	1.21	0.8	28.65	4.289	1.115	36.23	0.977	0.613	HIGH	1.33	HIGH	not Liquefiable	0	0
12	17	120	37	5		14.5	1740	1740	1459.2	1.09	0.85	53.46	0.000	1.000	53.46	0.970	0.691	HIGH	1.33	HIGH	not Liquefiable	0	0
17	22	120	37	5		19.5	2340	2340	1747.2	0.95	0.85	46.88	0.000	1.000	46.88	0.958	0.767	HIGH	1.33	HIGH	not Liquefiable	0	0
22	28	120	36	5		25	3000	3000	2064	0.84	0.95	44.91	0.000	1.000	44.91	0.942	0.819	HIGH	1.33	HIGH	not Liquefiable	0	0
28	33	120	40	5		30.5	3660	3660	2380.8	0.75	1	46.93	0.000	1.000	46.93	0.918	0.844	HIGH	1.33	HIGH	not Liquefiable	0	0
33	38	120	35	5		35.5	4260	4260	2668.8	0.68	1	37.44	0.000	1.000	37.44	0.887	0.847	HIGH	1.33	HIGH	not Liquefiable	0	0
38	44	120	33	5		41	4920	4920	2985.6	0.62	1	32.18	0.000	1.000	32.18	0.842	0.830	HIGH	1.33	HIGH	not Liquefiable	0	0
44	50	120	41	25		47	5640	5640	3331.2	0.57	1	36.46	4.289	1.115	44.94	0.783	0.793	HIGH	1.33	HIGH	not Liquefiable	0	0
																						Total	0

Well B-4

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	1:00		3.00	-		7.0			
	1:30	30	3.17	0.17	2.0	6.8	6.9	14.71	0.07
2	1:30		3.00			7.0			
	2:00	30	3.17	0.17	2.0	6.8	6.9	14.71	0.07
3	2:00		3.00			7.0			
	2:30	30	3.17	0.17	2.0	6.8	6.9	14.71	0.07
4	2:30		3.00			7.0			
	3:00	30	3.17	0.17	2.0	6.8	6.9	14.71	0.07
5	3:00		3.00			7.0			
	3:30	30	3.17	0.17	2.04	6.8	6.9	14.71	0.07

GS 200.1

Field Perc Rate (inch/hr) 4.1

Falling Head, Flow Rate, Q

d= 8 in

ΔH_w= 2.04 in

Δt= 30 min

Q= 3.42 in^3/min

Wetted Surface Area

H_w= 81.96 in

$A_{ws(1))}$= 2059.88 in^2

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

$A_{ws(1)} + A_{ws(2)}$= 2110.14 in^2

Adjusted Infiltration Rate

IR= 0.002 in/min

IR= 0.10 in/hr