



June 12, 2024

RMA Project Number 07-240341-0

Hillcrest  
2705 Mountain View Dr.  
La Verne, CA 91750

Attention: Mike Townsend

Subject: Geotechnical Investigation  
Gateway Project  
Park Avenue and A Street  
Le Verne, CA

Dear Mr. Townsend:

In accordance with your request, a geotechnical investigation has been completed for the proposed development at the above referenced property. The results of the investigation and testing are presented in the accompanying report, which includes a description of site conditions, results of our field exploration and testing, laboratory testing, conclusions, and recommendations. This report has been prepared for specific application to this project, in accordance with generally accepted geotechnical engineering practice.

We appreciate this opportunity to be of service to you. If you have any questions regarding this report, please do not hesitate to contact us at your convenience.

Respectfully submitted,

RMA GeoScience

Haiyan Liu, PE, C81463  
Project Engineer



Mark Swiatek, CEG 1781  
Principal Geologist

Distribution: (1) Addressee



**GEOTECHNICAL INVESTIGATION  
GATEWAY PROJECT  
PARK AVENUE & A STREET  
LA VERNE, CALIFORNIA**

For

Hillcrest  
2705 Mountain View Dr  
La Verne, CA 91750

June 12, 2024

Project No. 07-240341-0

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**APPENDICES**

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Appendix B	Laboratory Tests
Appendix C	General Earthwork and Grading Specifications
Appendix D	References

## 1.00 INTRODUCTION

### 1.01 Purpose

The purpose of the geotechnical investigation is to summarize the geotechnical and geologic conditions at the site and to assess their potential impact on the proposed development.

### 1.02 Scope of the Investigation

The general scope of this geotechnical investigation included the following:

- Review of published and unpublished geologic, seismic, groundwater, and geotechnical literature
- Examination of aerial photographs and topographic maps
- Review of State of California Alquist-Priolo Earthquake Fault Zone and Seismic Hazard maps
- Contacting of Underground Service Alert (USA) to locate onsite utility lines
- Logging, sampling, and backfilling of nine (9) hand auger borings.
- Laboratory testing of representative soil samples
- Geotechnical evaluation of the compiled data
- Preparation of this report presenting our findings, conclusions, and recommendations

Our scope of work did not include a preliminary site assessment for the potential of hazardous materials onsite.

### 1.03 Site Location and Description

The Gateway project has two parts: Gateway North and Gateway South and would address the geographic division of Hillcrest East and West campuses. The location of newly purchased homes in the Gateway project includes 2692, 2712 and 2730 Park Avenue and 2675, 2677, 2781, and 2783 A Street. The existing house will be demolished and 9 units of one-story single-family dwellings will be constructed.

### 1.04 Site History

Based on review of aerial photographs and other information readily available online the following site history has been determined. The existing home areas were occupied by single-family residences with an orchard and vacant lands from as far back as the earliest aerial photographed reviewed from 1948. Residential structures at 2692 and 2704 Park Avenue were shown in aerial photography in 1948. Residential structures at 2712 and 2730 Park Avenue were shown in aerial photography in 1953. Residential structures at 2675, 2677, 2781, and 2783 A Street were shown in aerial photography in 1964.

### **1.05 Record Review Findings**

Review of published geologic, seismic, groundwater, and geotechnical literature was performed for the site, and the information found was used to facilitate the writing of this investigation.

### **1.06 Planned Development**

It is our understanding that the current Gateway project includes 9 new homes. The new residential structures will be one-story wood-frame construction with slab-on-grade foundations.

### **1.07 Investigation Methods**

Our investigation consisted of office research, field exploration, laboratory testing, review and analysis of the compiled data, and preparation of this report. It has been performed in a manner consistent with generally accepted engineering and geologic principles and practices, and has incorporated applicable requirements of California Buildings Code. Definitions of technical terms and symbols used in this report include those of the ASTM International, the California Building Code, and commonly used geologic nomenclature.

Technical supporting data are presented in the attached appendices. Appendix A presents a description of the methods and equipment used in performing the field exploration and logs of our subsurface exploration. Appendix B presents a description of our laboratory testing and the test results. General Earthwork and Grading Specifications are presented in Appendix C. References are presented in Appendix D.

## **2.00 FINDINGS**

### **2.01 Geologic Setting**

The site is situated at the east end of the San Gabriel Valley between the San Jose Hills to the south and the San Gabriel Mountains to the north. The site lies on a broad alluvial fan that was formed from deposition of sediments emanating from San Dimas Canyon to the north. According to Dibblee (2002) the alluvial sediments are Pleistocene in age. A regional geologic map was presented in Figure 3.

The earth materials encountered during our investigation are described below. A full description of the earth materials encountered within each borehole is included on the boring logs, Appendix A.

### **2.02 Earth Materials**

#### Landscaping Lawn with Top Soils

Landscaping grass with top soils were encountered in all nine (9) hand auger borings with thickness ranging from 12 inches to 18 inches thick. The top soils consisted of silty fine sand with trace gravel and clay. The top soils were moist to very moist.

### Older Alluvium Deposits (Q<sub>oa</sub>)

The site is underlain by older alluvium which consists of light brown to dark brown with tan Silty Sand with gravel and clay and medium to coarse sand with gravel and silt. The soils were in a moist to very moist condition.

### **2.03 Expansive Soils**

Based on our preliminary observations and laboratory data, the soils at shallow depths are expected to have an expansion index in the low range. Additional expansion index and plasticity index testing will be required at the completion of rough grading to verify the properties of the near surface soils.

### **2.04 Surface and Groundwater Conditions**

Groundwater was not encountered in the boring excavated to a maximum depth of 15 feet. Surface water on the site is limited to precipitation falling directly on the ground surface and around buildings. There was no surface water encountered at the time of our investigation. Depth to historic high groundwater is greater than 100 feet below existing grade according to the Seismic Hazard Zone Report for the area (CDMG, 1998).

### **2.05 Faults**

The proposed site is not located within an Alquist-Priolo Earthquake Fault Zone, and there are no known active faults on or immediately adjacent to the property. However, there are faults in close enough proximity to the site to cause moderate to intense ground shaking during the lifetime of the proposed development. Additionally, the site has experienced earthquake-induced ground shaking in the past and can be expected to experience further shaking in the future. The closest zoned fault is the Cucamonga section of the Sierra Madre fault zone, located approximately 6 miles to the northeast of the subject site.

### **2.06 Landslides**

According to the California Geological Survey Seismic Hazard Zones Map for the area (1999) the site does not lie in a landslide hazard zone. Since the site is relatively flat and there are no nearby slopes earthquake-induced landsliding does not appear to be a hazard to proposed development.

### **2.07 Liquefaction**

According to the California Geological Survey Seismic Hazard Zones Map (1999) the site does not lie in a liquefaction hazard zone. Liquefaction is a phenomenon where earthquake-induced ground vibrations increase the pore pressure in saturated, granular soils until it is equal to the confining, overburden pressure. When this occurs, the soil can completely lose its shear strength and enter a liquefied state. The possibility of liquefaction is dependent upon grain size, relative density, confining pressure, saturation of the soils, and intensity and duration of ground shaking. In order for liquefaction to occur, three criteria must be met: underlying loose, coarse-grained (sandy) soils, a groundwater depth of less than about 50 feet, and a potential for seismic shaking from nearby large-magnitude earthquake.

Depth to groundwater in the region of the site is greater than 100 feet below ground surface as discussed in Section 2.04 and the site soils are relatively stiff or dense, therefore the liquefaction hazard potential at the site is negligible. A map showing the potential liquefaction hazard zone is presented in Figure 4.

## 2.08 Historic Seismicity

The region of the subject site has experienced shaking from several earthquakes recorded back to 1812. The nearest large historic earthquake is the Northridge Earthquake that occurred in 1994, the epicenter of which is 49.7 miles from the site. Historic earthquakes with magnitudes of greater than or equal to 6.0 and have been epicentered within approximately 50 miles of the site, are summarized in the table below.

**Large Historic Earthquakes**

<u>Earthquake Event</u>	<u>LAT. NORTH</u>	<u>LONG. WEST</u>	<u>DATE</u>	<u>DEPTH (km)</u>	<u>Quake Mag.</u>	<u>APPROX. DISTANCE mi [km]</u>
Raymond	34.830	118.750	11/27/1852	0	7.0	26.4(42.5)
Tejon Pass	34.900	118.900	10/23/1916	0	6.0	34.4( 55.3)
Sylmar	34.411	118.4	02/09/1971	8.4	6.4	34.6( 55.6)
Long Beach	33.6170	117.967	3/11/1933	0	6.3	35.5(57.2)
Unnamed	35.637	118.583	07/23/1952	0	6.1	35.9( 57.8)
Kern County	35.000	119.017	07/21/1952	0	7.7	41.5( 66.8)
Unnamed	34.300	118.600	04/04/1893	0	6.0	45.2( 72.7)
Unnamed	35.383	118.850	07/29/1952	0	6.1	45.6( 73.5)
Northridge	34.2130	118.5370	01/17/1994	18	6.7	49.7( 79.9)

## 2.09 Flooding Potential

According to Federal Emergency Management Agency (Flood Insurance Rate Map #06037C1725F, Effective date 9/26/2008), the site is located in an area of Flood Zone X, which is an area where the likelihood of flood hazards is considered minimal.

## 3.00 CONCLUSIONS AND RECOMMENDATIONS

### 3.01 General Conclusion

Based on the data collected thus far the project appears feasible from a geotechnical standpoint. Our preliminary recommendations provided below are based on the data collected thus far and our understanding of the project and our general experience in engineering geology and geotechnical engineering.

### 3.02 Seismic Design Parameters

Mapped Spectral Accelerations were obtained by using the online ATC Calculator (ASCE 7-16 Standard) and a site class D was used for the project site based on the soil profiles of the subject site and our experiences at

the project areas. Since the mapped risk-targeted maximum considered earthquake ( $MCE_R$ ) spectral response acceleration parameter at a period of 1 second ( $S_1$ ) is greater than 0.2, a ground motion hazard analysis is required per ACSE/SEI 7-16 to be performed in accordance with Section 21.2 for structures on Site Class D. However, instead of performing the ground motion hazard analysis, a long period coefficient ( $F_V$ ) of 1.7 was used for calculation of  $S_{M1}$  and  $S_{D1}$ . The parameters generated for the subject site are presented in the following table:

**2022 California Building Code (CBC) Seismic Parameters**

Parameter	Value
Site Location	Latitude = 34.1067 degrees Longitude = -117.7761 degrees
Site Class	Site Class = D
Mapped Spectral Accelerations	$S_s$ (0.2- second period) = 1.713g $S_1$ (1-second period) = 0.642g
Site Coefficients (Site Class D)	$F_a$ = 1.0 $F_v$ = 1.7
Maximum Considered Earthquake Spectral Accelerations (Site Class D)	$S_{MS}$ (0.2- second period) = 1.713g $S_{M1}$ (1-second period) = 1.091g
Design Earthquake Spectral Accelerations (Site Class D)	$S_{DS}$ (0.2- second period) = 1.142g $S_{D1}$ (1-second period) = 0.728g

For Risk Category II structures with mapped spectral response acceleration parameter at 1-s period ( $S_1$ ) is less than 0.75, the Seismic Design Category is D (ASCE 7-16 Section 11.6).

Peak earthquake ground acceleration adjusted for site class effects ( $PGA_M$ ) has been determined in accordance with ASCE 7-16 Section 11.8.3 as follows:  $PGA_M = F_{PGA} \times PGA = 1.1 \times 0.731g = 0.804g$ .

### 3.03 Liquefaction and Secondary Earthquake Hazards

Potential secondary seismic hazards that can affect land development projects include liquefaction, tsunamis, seiches, seismically induced settlement, seismically induced flooding and seismically induced landsliding.

#### Liquefaction

Liquefaction hazard potential for the site is discussed in Section 2.07 of this report.

#### Tsunamis and Seiches

Tsunamis are sea waves that are generated in response to large-magnitude earthquakes. When these waves reach shorelines, they sometimes produce coastal flooding. Seiches are the oscillation of large bodies of standing water, such as lakes, that can occur in response to ground shaking. Tsunamis and seiches do not pose hazards due to the inland location of the site.

### Seismically Induced Flooding

The Puddingstone Reservoir located approximately 1.5 miles southwest of the site sits at an elevation that is roughly 100 feet lower than the site. The potential for seismic induced flooding is unlikely.

### Seismically Induced Landsliding

According to the California Geological Survey Seismic Hazard Zones Map (1999) the site does not lie in a landslide hazard zone. Since the site is relatively flat earthquake-induced landsliding does not appear to be a hazard to proposed development.

### **3.04 General Earthwork and Grading**

All grading should be performed in accordance with the General Earthwork and Grading Specifications outlined in Appendix C, unless specifically revised or amended below. Recommendations contained in Appendix C are general specifications for typical grading projects and may not be entirely applicable to this project.

It is also recommended that all earthwork and grading be performed in accordance with the requirements of the lead agency.

### **3.05 Earthwork Shrinkage and Subsidence**

The site is not located within a zone of land subsidence according to the United States Geological Survey California Water Science Center website. It is our opinion that the potential for land subsidence due to over pumping of groundwater or oil extraction is low.

Shrinkage is the decrease in volume of soil upon removal and recompaction expressed as a percentage of the original in-place volume. Subsidence occurs as natural ground is densified to receive fill. These factors account for changes in earth volumes that will occur during grading. Our estimates are as follows:

- Shrinkage factor = 5% to 15% for soil removed and replaced as compacted fill.
- Subsidence factor = 0.1 foot.

The degree to which fill soils are compacted and variations in the in-situ density of existing soils will influence earth volume changes. Consequently, some adjustments in grades near the completion of grading could be required to balance the earthwork.

### **3.06 Removals and Overexcavation**

Upon demolition and removal of all existing structures, site improvements, all vegetation, organic rich soil (soils containing more than 2 percent organics by weight), trash and debris should be cleared from the grading area and removed from the site. After the removal of deleterious materials, stripping of organic-rich soils, and removal of tree roots, the following removals and over-excavation must be done within the area of the limits of grading:

- All fill materials should be removed down to competent native soils. Within the building areas, removals

are recommended to a minimum of five (5) feet below existing grade or three feet (3) below the bottom of planned footings, whichever is greater. Overexcavation should extend a minimum of five (5) feet outside the limits of the structure footprints.

- Within the area of planned walkway and all other areas where grading is planned, the subgrade must be over-excavated at least three (3) feet below the stripped surface or the finished subgrade surface, whichever is lower.

Following the over-excavation indicated above, a designated representative for the Project Geotechnical Engineer must review the exposed ground surface and determine if any additional over-excavation is required. The over-excavated ground surface in all areas determined to be satisfactory for the support of fills must be scarified to a minimum depth of 6 inches. Scarification should continue until the soils are broken down and free from lumps or clods and until the scarified zone is uniform. The moisture content of the scarified zone shall be adjusted to within 2% of the optimum moisture content. The scarified zone must then be uniformly compacted to at least 90% relative compaction

The above recommendations are based on the assumption that soils encountered during field exploration are representative of soils throughout the site. However, there can be unforeseen and unanticipated variations in soils between points of subsurface exploration. Hence, overexcavation depths must be verified, and adjusted if necessary, at the time of grading.

### **3.07 Swimming Pool Backfill**

The existing swimming pool should be demolished and backfilled. The pool location may be plotted on project fine grading plans or a separate plot plan. A permit may be required by City of La Verne. The following minimum standards are recommended.

- Break out or puncture holes in the bottom of the pool/spa structures to allow water to drain.
- Remove all decking and pool structures bond beam to within 12" below grade.
- Remove all electrical conduit and gas lines.
- Back fill with minimum 3/4" gravel to a depth of 6" along entire pool/spa bottom.
- Backfill with dirt and compact every 12" lift of dirt up to the finished grade.
- Submit a soil compaction report, signed by a licensed Soils Engineer, demonstrating that a minimum compaction of 90% has been achieved.

### **3.08 Foundation**

Isolated spread footings and/or continuous wall footings are recommended to support the proposed structures. If the recommendations in the section on grading are followed and footings are established in compacted fill materials, footings may be designed using the following allowable soil bearing values:

- Continuous Footings:

For one- and two-stories residential structures, footings having a minimum width of 12 inches and a minimum depth of 15 inches below the lowest adjacent grade have allowable bearing capacity of

2,000 pounds per square foot (psf). This value may be increased by 20% for each additional foot of width and/or depth to a maximum value of 3,000 psf.

- Isolated Spread Footings:

Footings having a minimum width of 24 inches and a minimum depth of 15 inches below the lowest adjacent grade have allowable bearing capacity of 2,000 psf. This value may be increased by 20% for each additional foot of width or depth to a maximum value of 3,000 psf.

- Retaining Wall Footings:

Footings for retaining walls should be founded a minimum depth of 12 inches and have a minimum width of 12 inches. Footings may be designed using the allowable bearing capacity and lateral resistance values recommended for building footings. However, when calculating passive resistance, the upper 6 inches of the footings should be ignored in areas where the footings will not be covered with concrete flatwork. Reinforcement should be provided for structural considerations as determined by the design engineer.

The above bearing capacities represent an allowable net increase in soil pressure over existing soil pressure and may be increased by one-third for short-term wind or seismic loads. The anticipated maximum total settlement is expected to be 1 inch with a differential settlement of approximately ½ inch in a 30 feet span.

A nominal reinforcement consisting of at least one #4 bar placed within 3 inches of the top of footings and another placed within 3 inches of the bottom of footings is recommended. Reinforcement of wide footings should be determined by the structural engineer who may also require heavier reinforcement.

Due to the preliminary nature of the expansion tests performed for this study, we recommend additional testing be performed near the completion of rough grading to verify the test results and recommended foundation design criteria.

### **3.09 Slab-On-Grade**

Concrete floor slabs on grade with a minimum thickness of 4 inches are recommended for slabs on grade for the proposed structures for normal floor loading conditions. However, if heavy concentrated or moving loads are anticipated, slabs should be designed using a modulus of subgrade reaction (k) of 150 psi/in when soils are prepared in conformance with the grading recommendations contained within the report. Reinforcement of slabs on grading is not required to mitigate the expansive soils. Reinforcement may be specified by the structural engineer.

Concrete floor slabs on grade should be divided into squares or rectangles using weakened plane joints (contraction joints), each with maximum dimensions not exceeding 15 feet. Contraction joints should be made in accordance with American Concrete Institute (ACI) guidelines. If weakened plane joints are not used, then the slabs shall be reinforced with 6x6-10/10 welded wire fabric placed at mid-height of the slab.

Special care should be taken on floors slabs to be covered with thin-set tile or other inflexible coverings. These areas may be reinforced with 6x6-10/10 welded wire fabric placed at mid-height of the slab, to mitigate drying shrinkage cracks. Alternatively, inflexible flooring may be installed with unbonded fabric or liners to prevent

reflection of slab cracks through the flooring.

A moisture vapor retarder/barrier is recommended beneath all slabs-on-grade that will be covered by moisture-sensitive flooring materials such as vinyl, linoleum, wood, carpet, rubber, rubber-backed carpet, tile, impermeable floor coatings, adhesives, or where moisture-sensitive equipment, products, or environments will exist. We recommend that design and construction of the vapor retarder or barrier conform to Section 1805 of the 2022 California Building Code (CBC) and pertinent sections of American Concrete Institute (ACI) guidance documents 302.1R-04, 302.2R-06 and 360R-10.

The moisture vapor retarder/barrier should consist of a minimum 10 mils thick polyethylene with a maximum perm rating of 0.3 in accordance with ASTM E 1745. Seams in the moisture vapor retarder/barrier should be overlapped no less than 6 inches or in accordance with the manufacturer's recommendations. Joints and penetrations should be sealed with the manufacturer's recommended adhesives, pressure-sensitive tape, or both. The contractor must avoid damaging or puncturing the vapor retarder/barrier and repair any punctures with additional polyethylene properly lapped and sealed.

ACI guidelines allow for the placement of moisture vapor retarder/barriers either directly beneath floor slabs or below an intermediate granular soil layer.

Placing the moisture retarder/barrier directly beneath the floor slab will provide improved curing of the slab bottom and will eliminate potential problems caused by water being trapped in a granular fill layer. Concrete slabs poured directly on a vapor retarder/barrier can experience shrinkage cracking and curling due to differential rates of curing through the thickness of the slab. Therefore, for concrete placed directly on the vapor retarded, we recommend a maximum water cement ratio of 0.45 and the use of water-reducing admixtures to increase workability and decrease bleeding.

If granular soil is placed over the vapor retarder/barrier, we recommend that the layer be at least 2 inches thick in accordance with traditional practice in southern California. Granular fill should consist of clean fine graded materials with 10 to 30% passing the No. 100 sieve and free from clay or silt. The granular layer should be uniformly compacted and trimmed to provide the full design thickness of the proposed slab. The granular fill layer should not be left exposed to rain or other sources of water such as wet-grinding, power washing, pipe leaks or other processes, and should be dry at the time of concrete placement. Granular fill layers that become saturated should be removed and replaced prior to concrete placement.

An additional layer of sand may be placed beneath the vapor retarder/barrier at the developer's discretion to minimize the potential of the retarder/barrier being punctured by underlying soils.

### **3.10 Miscellaneous Concrete Flatwork**

Miscellaneous concrete flatwork and walkways may be designed with a minimum thickness of 4 inches. Large slabs should be reinforced with a minimum of 6x6-10/10 welded wire mesh placed at mid-height in the slab. Control joints should be constructed to create squares or rectangles with a maximum spacing of 15 feet.

Walkways may be constructed without reinforcement. Walkways should be separated from foundations with a thick expansion joint filler. Control joints should be constructed into non-reinforced walkways at a maximum of 5 feet spacing.

The subgrade soils beneath all miscellaneous concrete flatwork should be compacted to a minimum of 95 percent relative compaction for a minimum depth of 12 inches. The geotechnical engineer should monitor the compaction of the subgrade soils and perform testing to verify that proper compaction has been obtained.

### 3.11 Footing Excavation and Slab Preparation

All footing excavations should be observed by the geotechnical consultant to verify that they have been excavated into competent soils. The foundation excavations should be observed prior to the placement of forms, reinforcement steel, or concrete. These excavations should be evenly trimmed and level. Prior to concrete placement, any loose or soft soils should be removed. Excavated soils should not be placed on slab or footing areas unless properly compacted.

Prior to the placement of the moisture barrier and sand, the subgrade soils underlying the slab should be observed by the geotechnical consultant to verify that all under-slab utility trenches have been properly backfilled and compacted, that no loose or soft soils are present, and that the slab subgrade has been properly compacted to a minimum of 95 percent relative compaction within the upper 12 inches. Footings may experience and overall loss in bearing capacity or an increased potential to settle where located in close proximity to existing or future utility trenches. Furthermore, stresses imposed by the footings on the utility lines may cause cracking, collapse and/or a loss of serviceability. To reduce this risk, footings should extend below a 1:1 plane projected upward from the closest bottom of the trench.

Subgrade soils beneath slabs on grade and walkways moist prior to the placement of concrete. The geotechnical consultant should verify that the appropriate moisture content has been achieved a maximum of 24 hours prior to the placement of concrete or moisture barriers.

### 3.12 Lateral Load Resistance

Lateral loads may be resisted by soil friction and the passive resistance of the soil. The following parameters are recommended.

- Passive Earth Pressure = 300 pcf (equivalent fluid weight).  
We recommend neglecting passive soil resistance from the upper foot of soil unless protected by a concrete slab or pavement.
- Coefficient of Friction (soil to footing) = 0.30
- Retaining structures should be designed to resist the following lateral active earth pressures:

Surface Slope of Retained Materials (Horizontal:Vertical)	Equivalent Fluid Weight (pcf)
Level	40
5:1	42
4:1	43
3:1	46
2:1	58

These active earth pressures are only applicable if the retained earth is allowed to strain sufficiently to achieve the active state. The required minimum horizontal strain to achieve the active state is approximately 0.0025H. Retaining structures should be designed to resist an at-rest lateral earth pressure if this horizontal strain cannot be achieved.

- At-rest Lateral Earth Pressure = 61 pcf (equivalent fluid weight)

The lateral earth pressure due to earthquake motions for the retaining walls are calculated by using  $PGA = S_{DS}/2.5 = 0.457g$  and  $\gamma = 130$  pcf for retaining wall dynamic load increment calculations. The point of application may vary between 0.37H to 0.40H.

- Basement (restrained) walls with level backfill:  $P_{ae} = \frac{1}{2}\gamma H^2(0.68 PGA/g) = 20.2H^2$
- Cantilever (unrestrained) walls with level backfill:  $P_{ae} = \frac{1}{2}\gamma H^2(0.42 PGA/g) = 12.3H^2$
- Cantilever (unrestrained) walls with no steeper than 2:1 slope:  $P_{ae} = \frac{1}{2}\gamma H^2(0.70 PGA/g) = 20.8H^2$

### 3.13 Cement Type and Corrosion Potential

Soluble sulfate tests performed on shallow samples of soil indicates that concrete at the subject project will have a negligible exposure to water-soluble sulfate in the soil. Our recommendations for concrete exposed to sulfate-containing soils are presented in the following table.

**Recommendations for Concrete exposed to Sulfate-containing Soils**

Sulfate Exposure	Water Soluble Sulfate (SO <sub>4</sub> ) in Soil (% by Weight)	Sulfate (SO <sub>4</sub> ) in Water (ppm)	Cement Type (ASTM C150)	Maximum Water-Cement Ratio (by Weight)	Minimum Compressive Strength (psi)
Negligible	0.00 - 0.10	0-150	--	--	2,500
Moderate	0.10 - 0.20	150-1,500	II	0.50	4,000
Severe	0.20 - 2.00	1,500-10,000	V	0.45	4,500
Very Severe	Over 2.00	Over 10,000	V plus pozzolan or slag	0.45	4,500

Use of alternate combinations of cementitious materials may be permitted if the combinations meet design recommendations contained in American Concrete Institute guideline ACI 318-11.

The soils were also tested for soil reactivity (pH) and electrical resistivity (ohm-cm). The test results indicate that the on-site soils have soil reactivity of 7.8 and 8.1 and electrical resistivity of 5,843 and 4,371 ohm-cm. A neutral or non-corrosive soil has a reactivity value ranging from 5.5 to 8.4. Generally, soils that could be considered moderately corrosive to ferrous metals have resistivity values of about 3,000 ohm-cm to 10,000 ohm-cm. Soils with resistivity values less than 3,000 ohm-cm can be considered corrosive and soils with resistivity values less than 1,000 ohm-cm can be considered extremely corrosive.

Based on our analysis, underlying onsite soils are moderately corrosive to ferrous metals. Protection of buried pipes utilizing coatings on all underground pipes; clean backfills and a cathodic protection system can be effective in controlling corrosion. A qualified corrosion engineer may be consulted to further assess the corrosive properties of the soil.

### **3.14 Utility Trench Backfill**

The onsite fill soils will not be suitable for use as pipe bedding for buried utilities. All pipes should be bedded in a sand, gravel or crushed aggregate imported material complying with the requirements of the Standard Specifications for Public Works Construction (Greenbook) Section 306-1.2.1. Crushed rock products that do not contain appreciable fines should not be utilized as pipe bedding and/or backfill. Bedding materials should be densified to at least 90% relative compaction (ASTM D1557). The geotechnical consultant should review and approve of proposed bedding materials prior to use.

The on-site soils are expected to be suitable as trench backfill provided they are screened of organic matter, boulders and cobbles over 6 inches in diameter. Trench backfill should be densified to at least 90% relative compaction (ASTM D1557). On-site granular soils with a sand equivalent value of 15 or greater may be water densified initially per Greenbook Specifications. Supplemental mechanical compaction methods will be required to attain the required 90% relative compaction.

All utility trench backfill within street right of way, utility easements, under or adjacent to sidewalks, driveways, or building pads should be observed and tested by the geotechnical consultant to verify proper compaction. Trenches excavated adjacent to foundations should not extend within the footing influence zone defined as the area within a line projected at a 1:1 drawn from the bottom edge of the footing. Trenches crossing perpendicular to foundations should be excavated and backfilled prior to the construction of the foundations. The excavations should be backfilled in the presence of the geotechnical engineer and tested to verify adequate compaction beneath the proposed footing.

Cal/OSHA construction safety orders should be observed during all underground work.

### **3.15 Temporary Excavations**

Based on the recommended removal depths as described in Section 3.08, temporary excavations within the limits of grading are expected to be 5 feet. Excavations may be cut vertically to a maximum height of 5 feet. Cuts above 5 feet may be laid back at a gradient of 1:1.

### **3.16 Drainage**

Surface drainage should be directed away from the proposed structures into suitable drainage devices. Neither excess irrigation nor rainwater should be allowed to collect or pond against building foundations or within low-lying or level areas of the lot. Surface waters should be diverted away from the tops of slopes and prevented from draining over the top of slopes and down the slope face.

### 3.17 Plan Review

Once formal plans are prepared for the subject property, this office should review the plans from a geotechnical viewpoint, comment on changes from the plan used during preparation of this report and revise the recommendations of this report where necessary.

## 4.00 CLOSURE

This investigation was completed in accordance with generally accepted industry practice to provide recommendations for developing the property from a geotechnical perspective. Information presented in this report is based on research, field investigation, laboratory testing, and engineer judgment obtained from similar projects completed on nearby properties. This assessment is not, and should not be construed as, a warranty or guarantee concerning the geotechnical conditions which may affect the future development of the property. All discovered information has been disclosed and a good faith effort has been made to consult pertinent sources.

This study and report have been prepared on behalf and for the exclusive use of Hillcrest and solely for use as a preliminary evaluation of the subject site. This report and its findings shall not, in whole or in part, be disseminated or conveyed to any other party, nor used by any other party in whole or in part, without prior written consent of RMA Geoscience, Inc. and Hillcrest Homes. However, RMA Geoscience, Inc. acknowledges and agrees that the report may be conveyed to the design professionals for consideration in developing the property.



## FIGURES



From Google Earth Pro June 2024

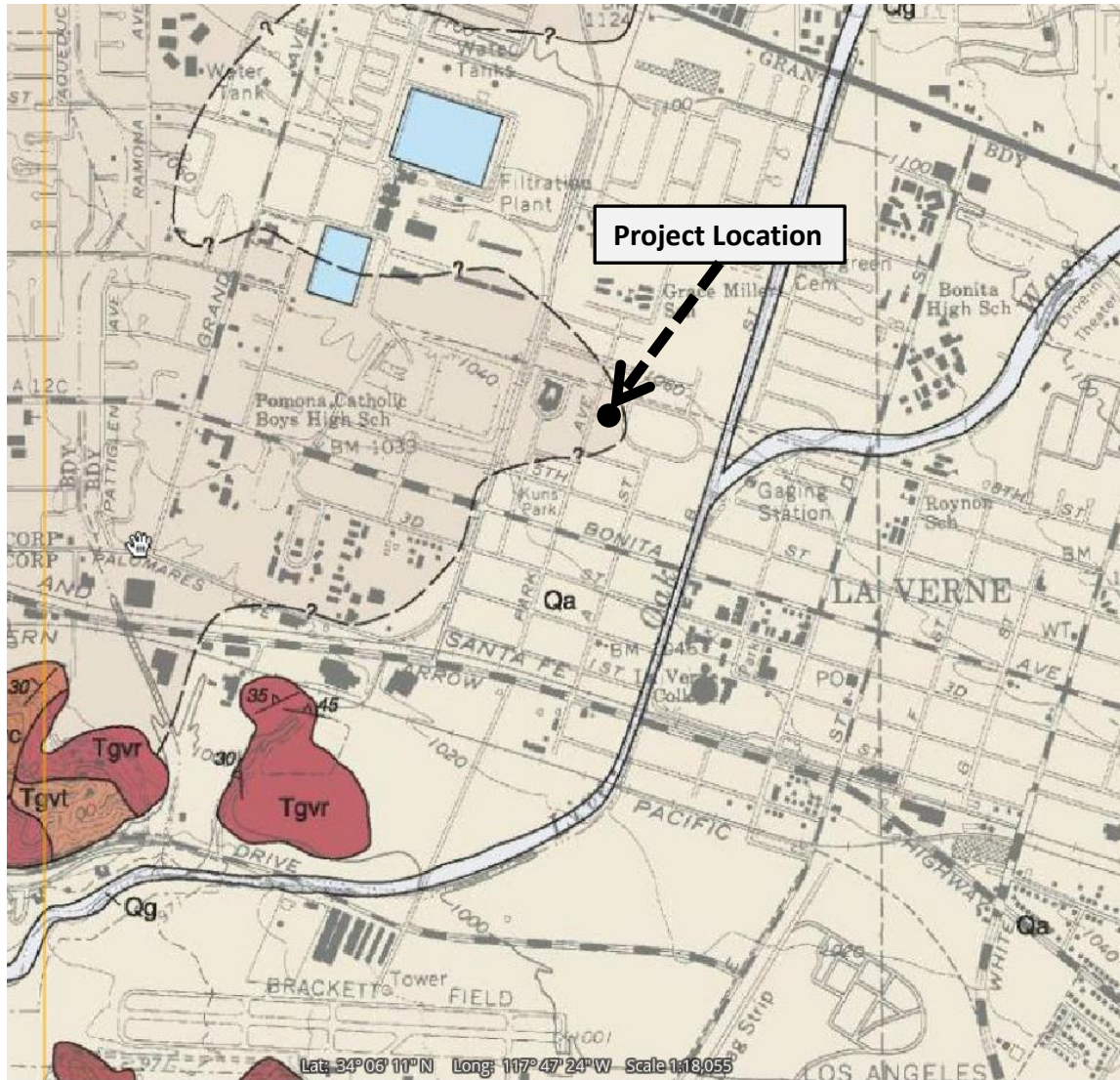
North  
↑  
Not to Scale

**Figure 1: Site Vicinity Map**



HA-1  Approximate Hand Auger location Base Map: site plan - revised 4-24-24 from Hillcrest

**Figure 2: Hand Auger Location Map**



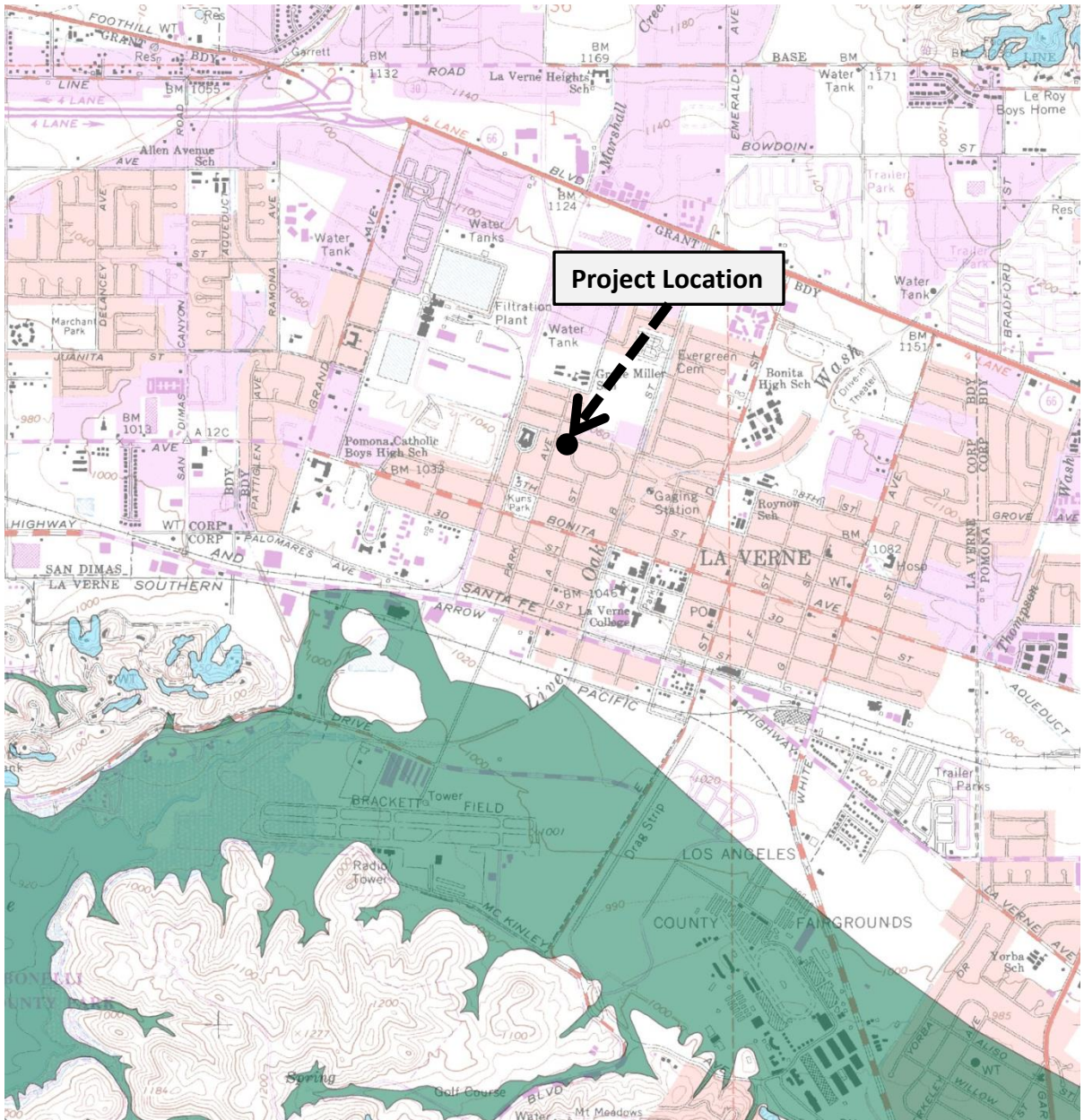
Source: <https://ngmdb.usgs.gov/mapview/>

North



Not to Scale

**Figure 3: Regional Geologic Map**



North



Not to Scale

Base Map: California Division of Mines and Geology, Seismic Hazard Zone Map, San Dimas Quadrangle, 1999

**Figure 4: Seismic Hazard Map**



**APPENDIX A**  
**FIELD INVESTIGATION**



## APPENDIX A

### FIELD INVESTIGATION

#### A-1.00 FIELD EXPLORATION

##### A-1.01 Number of Borings

Our subsurface investigation consisted of excavation of 9 exploratory borings to depths of 15 feet with a hand labor crew within subject sites.

##### A-1.02 Location of Boring

A Boring Location Map showing the approximate locations of the borings is presented as Figure 2.

##### A-1.03 Boring Logging

Logs of the boring were prepared by one of our staff and are attached in this appendix. The logs contain factual information and interpretation of subsurface conditions between samples. The strata indicated on these logs represent the approximate boundary between earth units and the transition may be gradual. The logs show subsurface conditions at the dates and locations indicated, and may not be representative of subsurface conditions at other locations and times.

Identification of the soils encountered during the subsurface exploration was made using the field identification procedure of the Unified Soils Classification System (ASTM D2488). A legend indicating the symbols and definitions used in this classification system and a legend defining the terms used in describing the relative compaction, consistency or firmness of the soil are attached in this appendix. Bag samples of the major earth units were obtained for laboratory inspection and testing.



**BASED ON STANDARD PENETRATION TESTS**

Compactness of sand		Consistency of clay	
Penetration Resistance N (blows/Ft)	Compactness	Penetration Resistance N (blows/ft)	Consistency
0-4	Very Loose	<2	Very Soft
4-10	Loose	2-4	Soft
10-30	Medium Dense	4-8	Medium Stiff
30-50	Dense	8-15	Stiff
>50	Very Dense	15-30	Very Stiff
		>30	Hard

N = Number of blows of 140 lb. weight falling 30 in. to drive 2-in OD sampler 1 ft.

**BASED ON RELATIVE COMPACTION**

Compactness of sand		Consistency of clay	
% Compaction	Compactness	% Compaction	Consistency
<75	Loose	<80	Soft
75-83	Medium Dense	80-85	Medium Stiff
83-90	Dense	85-90	Stiff
>90	Very Dense	>90	Very Stiff

**II. SOIL MOISTURE**

Moisture of sands		Moisture of clays	
% Moisture	Description	% Moisture	Description
<5%	Dry	<12%	Dry
5-12%	Moist	12-20%	Moist
>12%	Very Moist	>20%	Very Moist, wet





9854 Glenoaks Blvd., Sun Valley, CA 91352

# HAND AUGER NUMBER 2

Page 1 of 1

Client: Hillcrest

Project Name: Gateway Project

Project Number 07-240341

Project Location: Park Avenue, Le Verne, CA

Date Started: 5/30/2024 Completed: 5/30/2024

Ground Elevation: 1058' Boring Diameter: 4"

Excavation Method: hand digging

Ground Water Levels: Not Encountered

Contractor: Mike's excavating

Notes: \_\_\_\_\_

Logged By: HHL

Depth (ft)	Drive Sample	Blow Count (N Value)	Bulk Sample	Moisture Content (%)	Dry Density (pcf)	Wet Density (pcf)	Liquid Limit	Plastic Limit	Plasticity Index	Material Description	<#200	D <sub>50</sub>	USCS Classification
2				17.5	107.6	126.5				Landscaping Grass with top soil = 1.0 feet			
5				16.3	113.9	132.4				At 2 feet: Silty Sand with clay, dark brown, moist			SM
10				14.5	113.4	129.9				At 5 feet: Silty Sand with gravel and trace clay, light brown, moist			SM
15				6.9	121.5	129.8				At 10 feet: Silty Sand, moist, light brown			SM
										At 13 feet: became gravelly			
										At 15 feet: Sand and Gravel with silt , moist, light brown with tan and gray			SP-SM
20										<b>Total Depth = 15 feet</b>			
										<b>No groundwater encountered</b>			
										<b>Backfilled with cuttings</b>			

















**APPENDIX B**  
**LABORATORY TESTS**

## APPENDIX B

### B-1.00 LABORATORY TESTS

#### B-1.01 Maximum Density

Maximum density - optimum moisture relationships for the major soil type encountered during the field exploration were performed in the laboratory using the standard procedures of ASTM D1557.

#### B-1.02 Expansion Tests

Expansion index tests were performed on representative samples of the soil types present during grading by the test methods outlined in ASTM D4829.

#### B-1.03 Moisture Determination

Moisture content of the soil samples was performed in accordance to standard method for determination of water content of soil by drying oven, ASTM D2216. The mass of material remaining after oven drying is used as the mass of the solid particles.

#### B-1.04 Density of Split-Barrel Samples

The density of ring and tube samples, which were obtained using a split-barrel sampler, were determined in accordance with ASTM D2937. The results of these tests are provided on the boring logs in Appendix A.

#### B-1.05 Consolidation (One-Dimensional)

One-dimensional consolidation tests were performed on ring samples using the standard test method of ASTM D2435. The rate of consolidation of the tested samples was not determined and the applied loading increments are indicated on the summary of the test results. The test results are included in this Appendix B.

#### B-1.06 Soluble Sulfates

A test was performed on representative sample encountered during the investigation using the California Test Method 417.

#### B-1.07 Soil Reactivity (pH) and Minimum Resistivity

A near-surface soil samples were tested for soil reactivity (pH) and minimum electrical resistivity using California Test Method 643. The pH measurement determines the degree of acidity or alkalinity in the soils. The minimum resistivity is used as an indicator of how corrosive the soil is relative to buried metallic items.

#### B-1.08 Test Results

Test results for all laboratory tests performed on the subject project are presented in this appendix. For a sample-by-sample description, see the logs presented in Appendix A.

MAXIMUM DENSITY - OPTIMUM MOISTURE

(Test Method: ASTM D1557)

Sample Number	Optimum Moisture (Percent)	Maximum Density (lbs/ft <sup>3</sup> )
B3 @ 0-5 ft	11.5	125.2
B5 @ 0-5 ft	12.3	126.1

EXPANSION TEST

Test Method: ASTM D4829

Sample Number	Expansion Index	Expansion Classification
B-3 at 0-5 ft	27	Low
B-4 at 0-5 ft	0	Very Low

SOLUBLE SULFATES

(California Test Methods: 417 & 422)

Sample Number	Soluble Sulfate (ppm)	Chloride Content (ppm)
B1 @ 0-5 ft	23	20
B8 @ 0-5 ft	16	31

SOIL REACTIVITY (pH) AND MINIMUM RESISTIVITY

(California Test Method: 643)

Sample Number	pH	Minimum Resistivity (Ohm-cm)
B1 @ 0-5 ft	8.1	5843
B8 @ 0-5 ft	7.8	4371

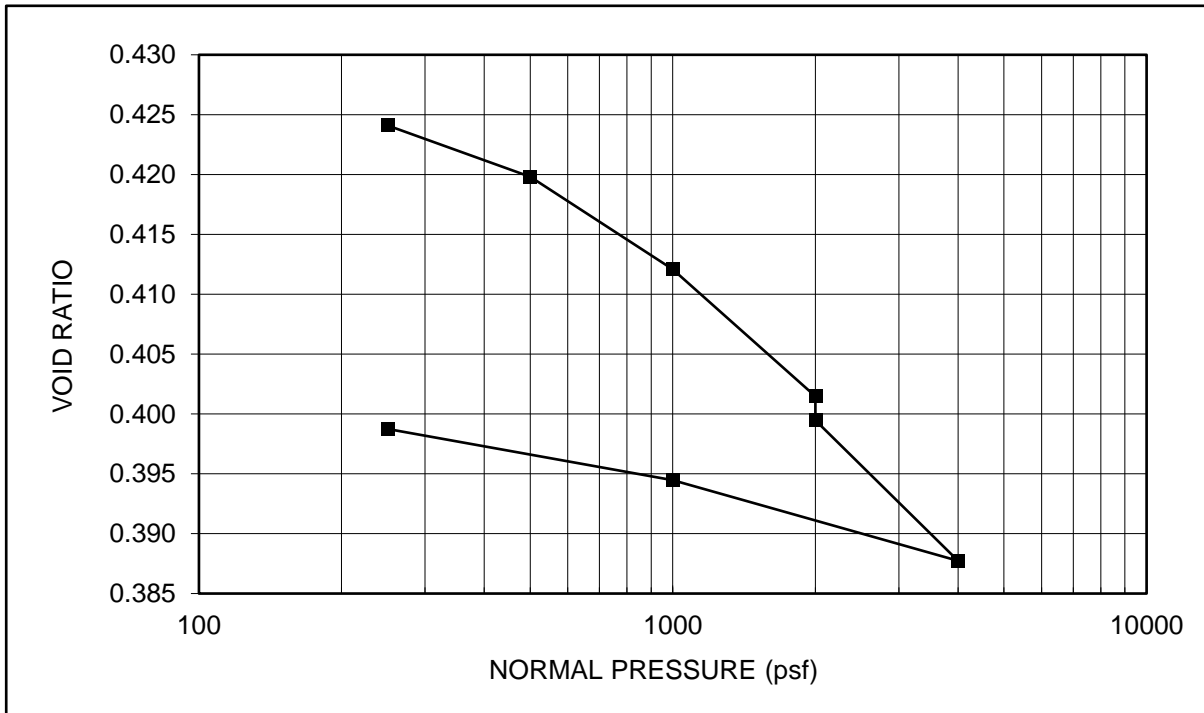


**ONE-DIMENSIONAL CONSOLIDATION  
ASTM D2435**

Project ID: 07-240341-0  
 Location: HA-1  
 Depth: 5 feet  
 Soil Description: Silty Sand

Dry Unit Weight (pcf): 115.5	Initial Dial Reading: 0.0232	inch
Initial Moisture: 14.0%	Initial Specimen Height: 1.0000	inch
Final Moisture: 18.2%	Initial Void Ratio: 0.432	
Initial Saturation: 85.9%	Final Void Ratio: 0.399	
Final Saturation: 99.6%	Specific Gravity : 2.65	

Moisture Condition	Load (psf)	Final Dial Reading (inches)	Sample Height (inches)	Void Ratio	Strain (%)	ΔStrain (%)
In Situ	250	0.0285	0.9947	0.424	0.53	0.53
	500	0.0315	0.9917	0.420	0.83	0.00
	1000	0.0369	0.9863	0.412	1.37	0.54
	2000	0.0443	0.9789	0.401	2.11	0.74
Add water	2000	0.0457	0.9775	0.399	2.25	0.14
	4000	0.0539	0.9693	0.388	3.07	0.82
	1000	0.0492	0.9740	0.394	2.60	-0.47
	250	0.0462	0.9770	0.399	2.30	-0.30



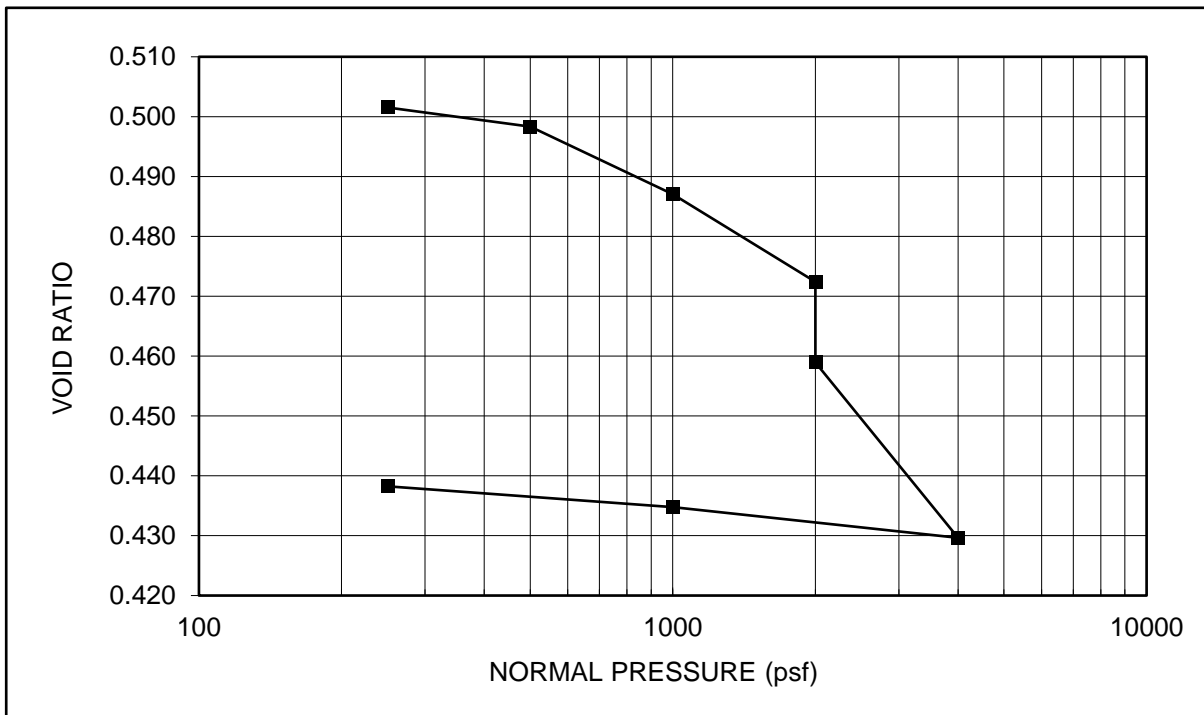


**ONE-DIMENSIONAL CONSOLIDATION  
ASTM D2435**

Project ID: 07-240341-0  
 Location: HA-2  
 Depth: 10 feet  
 Soil Description: Silty Sand

Dry Unit Weight (pcf): 109.5	Initial Dial Reading: 0.0553	inch
Initial Moisture: 15.9%	Initial Specimen Height: 1.0000	inch
Final Moisture: 17.6%	Initial Void Ratio: 0.510	
Initial Saturation: 82.6%	Final Void Ratio: 0.438	
Final Saturation: 99.3%	Specific Gravity : 2.65	

Moisture Condition	Load (psf)	Final Dial Reading (inches)	Sample Height (inches)	Void Ratio	Strain (%)	ΔStrain (%)
In Situ	250	0.0610	0.9943	0.502	0.57	0.57
	500	0.0631	0.9922	0.498	0.78	0.00
	1000	0.0706	0.9847	0.487	1.53	0.75
	2000	0.0803	0.9750	0.472	2.50	0.97
Add water	2000	0.0892	0.9661	0.459	3.39	0.89
	4000	0.1086	0.9467	0.430	5.33	1.94
	1000	0.1052	0.9501	0.435	4.99	-0.34
	250	0.1029	0.9524	0.438	4.76	-0.23



**APPENDIX C**

**GENERAL EARTHWORK  
AND  
GRADING SPECIFICATIONS**

## GENERAL EARTHWORK AND GRADING SPECIFICATIONS

### C-1.00 GENERAL DESCRIPTION

#### C-1.01 Introduction

These specifications present our general recommendations for earthwork and grading as shown on the approved grading plans for the subject project. These specifications shall cover all clearing and grubbing, removal of existing structures, preparation of land to be filled, filling of the land, spreading, compaction and control of the fill, and all subsidiary work necessary to complete the grading of the filled areas to conform with the lines, grades and slopes as shown on the approved plans.

The recommendations contained in the geotechnical report of which these general specifications are a part of shall supersede the provisions contained hereinafter in case of conflict.

#### C-1.02 Laboratory Standard and Field Test Methods

The laboratory standard used to establish the maximum density and optimum moisture shall be ASTM D1557.

The insitu density of earth materials (field compaction tests) shall be determined by the sand cone method (ASTM D1556), direct transmission nuclear method (ASTM D2922) or other test methods as considered appropriate by the geotechnical consultant.

Relative compaction is defined, for purposes of these specifications, as the ratio of the in-place density to the maximum density as determined in the previously mentioned laboratory standard.

### C-2.00 Clearing

#### C-2.01 Surface Clearing

All structures marked for removal, timber, logs, trees, brush and other rubbish shall be removed and disposed of off the site. Any trees to be removed shall be pulled in such a manner so as to remove as much of the root system as possible.

#### C-2.02 Subsurface Removals

A thorough search should be made for possible underground storage tanks and/or septic tanks and cesspools. If found, tanks should be removed and cesspools pumped dry.

Any concrete irrigation lines shall be crushed in place and all metal underground lines shall be removed from the site.

#### C-2.03 Backfill of Cavities

All cavities created or exposed during clearing and grubbing operations or by previous use of the site shall be cleared of deleterious material and backfilled with native soils or other materials approved by the soil engineer. Said backfill shall be compacted to a minimum of 90% relative compaction.

### **C-3.00 ORIGINAL GROUND PREPARATION**

#### **C-3.01 Stripping of Vegetation**

After the site has been properly cleared, all vegetation and topsoil containing the root systems of former vegetation shall be stripped from areas to be graded. Materials removed in this stripping process may be used as fill in areas designated by the soil engineer, provided the vegetation is mixed with a sufficient amount of soil to assure that no appreciable settlement or other detriment will occur due to decaying of the organic matter. Soil materials containing more than 3% organics shall not be used as structural fill.

#### **C-3.02 Removals of Non-Engineered Fills**

Any non-engineered fills encountered during grading shall be completely removed and the underlying ground shall be prepared in accordance to the recommendations for original ground preparation contained in this section. After cleansing of any organic matter the fill material may be used for engineered fill.

#### **C-3.03 Overexcavation of Fill Areas**

The existing ground in all areas determined to be satisfactory for the support of fills shall be scarified to a minimum depth of 6 inches. Scarification shall continue until the soils are broken down and free from lumps or clods and until the scarified zone is uniform. The moisture content of the scarified zone shall be adjusted to within 2% of optimum moisture. The scarified zone shall then be uniformly compacted to 90% relative compaction.

Where fill material is to be placed on ground with slopes steeper than 5:1 (H:V) the sloping ground shall be benched. The lowermost bench shall be a minimum of 15 feet wide, shall be a minimum of 2 feet deep, and shall expose firm material as determined by the geotechnical consultant. Other benches shall be excavated to firm material as determined by the geotechnical consultant and shall have a minimum width of 4 feet.

Existing ground that is determined to be unsatisfactory for the support of fills shall be overexcavated in accordance to the recommendations contained in the geotechnical report of which these general specifications are a part.

### **C-4.00 FILL MATERIALS**

#### **C-4.01 General**

Materials for the fill shall be free from vegetable matter and other deleterious substances, shall not contain rocks or lumps of a greater dimension than is recommended by the geotechnical consultant, and shall be approved by the geotechnical consultant. Soils of poor gradation, expansion, or strength properties shall be placed in areas designated by the geotechnical consultant or shall be mixed with other soils providing satisfactory fill material.

#### **C-4.02 Oversize Material**

Oversize material, rock, or other irreducible material with a maximum dimension greater than 12 inches shall not be placed in fills, unless the location, materials, and disposal methods are specifically approved by the geotechnical consultant. Oversize material shall be placed in such a manner that nesting of oversize material does not occur and in such a manner that the oversize material is completely surrounded by fill material compacted to a minimum of



90% relative compaction. Oversize material shall not be placed within 10 feet of finished grade without the approval of the geotechnical consultant.

#### **C-4.03 Import**

Material imported to the site shall conform to the requirements of Section 4.01 of these specifications. Potential import material shall be approved by the geotechnical consultant prior to importation to the subject site.

### **C-5.00 PLACING AND SPREADING OF FILL**

#### **C-5.01 Fill Lifts**

The selected fill material shall be placed in nearly horizontal layers which when compacted will not exceed approximately 6 inches in thickness. Thicker lifts may be placed if testing indicates the compaction procedures are such that the required compaction is being achieved and the geotechnical consultant approves their use.

Each layer shall be spread evenly and shall be thoroughly blade mixed during the spreading to insure uniformity of material in each layer.

#### **C-5.02 Fill Moisture**

When the moisture content of the fill material is below that recommended by the soils engineer, water shall then be added until the moisture content is as specified to assure thorough bonding during the compacting process.

When the moisture content of the fill material is above that recommended by the soils engineer, the fill material shall be aerated by blading or other satisfactory methods until the moisture content is as specified.

#### **C-5.03 Fill Compaction**

After each layer has been placed, mixed, and spread evenly, it shall be thoroughly compacted to not less than 90% relative compaction. Compaction shall be by sheepsfoot rollers, multiple-wheel pneumatic tired rollers, or other types approved by the soil engineer.

Rolling shall be accomplished while the fill material is at the specified moisture content. Rolling of each layer shall be continuous over its entire area and the roller shall make sufficient trips to insure that the desired density has been obtained.

#### **C-5.04 Fill Slopes**

Fill slopes shall be compacted by means of sheepsfoot rollers or other suitable equipment. Compacting of the slopes may be done progressively in increments of 3 to 4 feet in fill height. At the completion of grading, the slope face shall be compacted to a minimum of 90% relative compaction. This may require track rolling or rolling with a grid roller attached to a tractor mounted side-boom.

Slopes may be over filled and cut back in such a manner that the exposed slope faces are compacted to a minimum of 90% relative compaction.



The fill operation shall be continued in six inch (6") compacted layers, or as specified above, until the fill has been brought to the finished slopes and grades as shown on the accepted plans.

#### **C-5.05 Compaction Testing**

Field density tests shall be made by the geotechnical consultant of the compaction of each layer of fill. Density tests shall be made at locations selected by the geotechnical consultant.

Frequency of field density tests shall be not less than one test for each 2.0 feet of fill height and at least every one thousand cubic yards of fill. Where fill slopes exceed four feet in height their finished faces shall be tested at a frequency of one test for each 1000 square feet of slope face.

Where sheepfoot rollers are used, the soil may be disturbed to a depth of several inches. Density reading shall be taken in the compacted material below the disturbed surface. When these readings indicate that the density of any layer of fill or portion thereof is below the required density, the particular layer or portion shall be reworked until the required density has been obtained.

### **C-6.00 SUBDRAINS**

#### **C-6.01 Subdrain Material**

Subdrains shall be constructed of a minimum 4-inch diameter pipe encased in a suitable filter material. The subdrain pipe shall be Schedule 40 Acrylonitrile Butadiene Styrene (ABS) or Schedule 40 Polyvinyl Chloride Plastic (PVC) pipe or approved equivalent. Subdrain pipe shall be installed with perforations down. Filter material shall consist of 3/4" to 1 1/2" clean gravel wrapped in an envelope of filter fabric consisting of Mirafi 140N or approved equivalent.

#### **C-6.02 Subdrain Installation**

Subdrain systems, if required, shall be installed in approved ground to conform the approximate alignment and details shown on the plans or herein. The subdrain locations shall not be changed or modified without the approval of the geotechnical consultant. The geotechnical consultant may recommend and direct changes in the subdrain line, grade or material upon approval by the design civil engineer and the appropriate governmental agencies.

### **C-7.00 EXCAVATIONS**

#### **C-7.01 General**

Excavations and cut slopes shall be examined by the geotechnical consultant. If determined necessary by the geotechnical consultant, further excavation or overexcavation and refilling of overexcavated areas shall be performed, and/or remedial grading of cut slopes shall be performed.

#### **C-7.02 Fill-Over-Cut Slopes**

Where fill-over-cut slopes are to be graded the cut portion of the slope shall be made and approved by the geotechnical consultant prior to placement of materials for construction of the fill portion of the slope.

## **C-8.00 TRENCH BACKFILL**

### **C-8.01 General**

Trench backfill within street right of ways shall be compacted to 90% relative compaction as determined by the ASTM D1557 test method. Backfill may be jetted as a means of initial compaction; however, mechanical compaction will be required to obtain the required percentage of relative compaction. If trenches are jetted, there must be a suitable delay for drainage of excess water before mechanical compaction is applied.

## **C-9.00 SEASONAL LIMITS**

### **C-9.01 General**

No fill material shall be placed, spread or rolled while it is frozen or thawing or during unfavorable weather conditions. When the work is interrupted by heavy rain, fill operations shall not be resumed until field tests by the soils engineer indicate that the moisture content and density of the fill are as previously specified.

## **C-10.00 SUPERVISION**

### **C-10.01 Prior to Grading**

The site shall be observed by the geotechnical consultant upon completion of clearing and grubbing, prior to the preparation of any original ground for preparation of fill.

The supervisor of the grading contractor and the field representative of the geotechnical consultant shall have a meeting and discuss the geotechnical aspects of the earthwork prior to commencement of grading.

### **C-10.02 During Grading**

Site preparation of all areas to receive fill shall be tested and approved by the geotechnical consultant prior to the placement of any fill.

The geotechnical consultant or his representative shall observe the fill and compaction operations so that he can provide an opinion regarding the conformance of the work to the recommendations

**APPENDIX D**

**REFERENCES**

## REFERENCES

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