Phelan Piñon Hills CSD
Proposed Park Site
South of Phelan Road and East of
Sheep Creek Road
Phelan, California

Geotechnical Investigation Report

Prepared For: Phelan Piñon Hills CSD
Prepared By: Merrell Johnson

April 29, 2022

George Cardenas
PPHCSD Engineering Manager
4176 Warbler Road
PO BOX 294049
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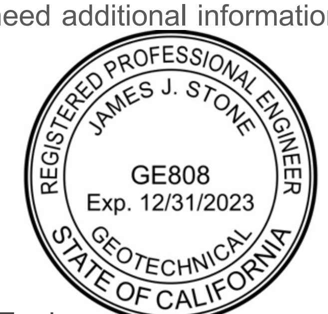
Re: Geotechnical Investigation
Phelan Piñon Hills CSD
Proposed Park Site
South of Phelan Road and East of Sheep Creek Road
Phelan, California

George Cardenas:

In accordance with your request and authorization, we performed a geotechnical investigation for a portion of the park site proposed by Phelan Piñon Hills CSD. The site is located south of Phelan Road and east of Sheep Creek Road in Phelan, California. The investigation was planned and performed using the plan prepared by Steeno Design Studios dated Dec. 2021, discussions with Phelan Piñon Hills CSD, and a site reconnaissance.

If you have any questions or need additional information, please do not hesitate to contact our firm.

Sincerely,
Merrell Johnson Companies



James J. Stone, Geotechnical Engineer
RGE 808 Exp. 12/31/2023

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Table of Contents

Attachments

Introduction

This report presents the results of the geotechnical investigation Merrell Johnson (MJ) performed for a portion of the park site proposed by Phelan Piñon Hills CSD. The site is located south of Phelan Road and east of Sheep Creek Road in Phelan, California. The investigation was planned and performed using the plan prepared by Steeno Design Studios dated Dec. 2021, discussions with Phelan Piñon Hills CSD, and a site reconnaissance. The location of the proposed development is shown on Figure 1 in Appendix A.

The scope of the Geotechnical Investigation is limited to Phase 3 (14.22-acres). Phase 4 is not a part; except that a separate infiltration test report has been prepared for the proposed infiltration basins. The field exploration program was planned when there was an aquatic center and an equestrian center with covered bleachers proposed within the Phase 4 area. These structures have now been eliminated from the plan. That is why there are borings in the Phase 4 area. Only the restroom building in Phase 3 is proposed at this time.

Two percolation test borings were drilled next to the restroom building; however, no laboratory testing was performed in these borings.

Scope of Services

The scope of work for this project consisted of field exploration, laboratory testing, engineering analyses, and preparation of this report. The results of the field exploration and laboratory test programs were analyzed to develop conclusions and recommendations regarding:

- Subsurface conditions underlying the site
- Site preparation and grading
- Excavation conditions
- Foundation support for the new structure along with preliminary geotechnical engineering criteria for foundation design
- Support for slab-on-grade floors
- Flexible pavement structural sections
- Corrosivity of site soils with respect to reinforced concrete and ferrous metals.

Field Exploration and Laboratory Testing

Field Exploration

Subsurface conditions were explored by drilling 5 test borings 15 feet deep at the locations shown on Figure 2 in Appendix A. The borings were logged by an MJ representative, who also collected samples of the materials encountered for examination and laboratory testing.

Bulk samples were collected from drill cuttings. Relatively undisturbed samples were obtained by driving a 2.5-inch inside diameter modified California sampler with a 140-pound hammer falling 30 inches in accordance with American Society for Testing and Materials (ASTM) Standard D3550. Blow counts required to drive the sampler each 6 inches of an 18-inch (or less) drive are noted on the boring logs as “N” value.

Standard Penetration Tests (SPTs) were performed at selected depths by driving a 1.4-inch inside diameter sampler 18 inches with a 140-pound hammer falling 30 inches in accordance with ASTM D1586. The blow counts required to drive the sampler each 6 inches of the drive are noted on the boring logs as “N” value. Disturbed samples were collected from the SPT sampler at the time of driving.

The logs of the test borings are in Appendix B. Soils are described according to the Unified Soil Classification System explained in Appendix B.

Laboratory Testing

The laboratory program included the following tests:

- In-place moisture content and dry density
- Grain size analysis
- R- (Resistance) Value.

The results of the laboratory tests are summarized in Appendix C.

Site and Subsurface Conditions

Site Conditions

The surface of the site is vacant with some scattered desert vegetation. The ground slopes gently downward from south to north, with a fall of about 20 feet from the southern boundary to Phelan Road.

Subsurface Conditions

The site is underlain primarily by medium dense silty sand and poorly graded sand. No groundwater was encountered in the test borings drilled for this investigation. United States Geological Survey data indicate that the groundwater level is expected to be on the order of several hundred feet deep.

Site Class, Site Coefficient and Seismic Design Category

The soils underlying the site are classified as Site Class D-Default according to the California Building Code (CBC) due to the lack of site-specific subsurface information to a depth of 100 feet. The Design Acceleration Parameters were determined according to Chapter 11 of ASCE 7-16 and are provided in the table below.

California Building Code – Seismic Parameters

MCE_R ground motion. (for 0.2 second period)	S_S = 1.531
MCE_R ground motion. (for 1.0s period)	S_1 = 0.628
Site-modified spectral acceleration value	S_{MS} = 1.837
Site-modified spectral acceleration value	S_{M1} = null
Numeric seismic design value at 0.2 second SA	S_{DS} = 1.225
Numeric seismic design value at 1.0 second SA	S_{D1} = null
Site amplification factor at 0.2 second	F_a = 1.2
Site amplification factor at 1.0 second	F_v = null
MCE_G peak ground acceleration	PGA = 0.671
Site amplification factor at PGA	F_{PGA} = 1.2
Site-modified peak ground acceleration	PGA_M = 0.805

Conclusions and Recommendations

Conclusions

The existing surface soils in some areas of the site have been disturbed by traffic and weather. Below a depth of about 1 foot, the on-site soils appear generally undisturbed. No active or potentially active faults are shown to cross the site on the Fault Activity Map of California published by the California Geological Survey. The granular soils are medium dense, and groundwater is deep below this site. The liquefaction potential consequently is very low. The potential for dynamically induced settlement of the granular soils is also very low. In addition, the soils have a very low potential for expansion due to changes in moisture content. The potential for encountering groundwater within the anticipated relatively shallow excavations is minimal. There is a potential for minor amounts of water to enter open excavations as a result of direct rainfall and runoff.

Earthwork

Debris, vegetation, and other deleterious materials should be stripped and removed from the site prior to grading work. Organic materials should be disposed of off-site in accordance with the owner's instructions.

Areas to receive fill supporting new structures, slabs-on-grade, and pavements should be scarified to a depth of 12 inches, brought to within 2 percentage points above or below optimum moisture content, and compacted to a minimum of 90% relative compaction based on the ASTM D1557 laboratory test method. All references to optimum moisture content and relative compaction in this report are based on this test method.

Compacted Fill Material

Fill material should consist of clean soils containing no rocks or other particles with a maximum dimension larger than 6 inches. The on-site soils, less any oversize particles, debris, and organic matter, can be used as fill.

Imported soils should consist of predominantly granular material with an expansion index less than 20 when tested in accordance with ASTM D4829 and should have a minimum R-value of 40. Imported material should be inspected and approved by an MJ representative prior to being brought to the site.

Compacted Fill Placement

Fill that will support new structures, slabs-on-grade, and pavements should be placed in 8-inch-thick loose lifts, moisture conditioned to within 2 percentage points above or below optimum moisture content and compacted to a minimum of 90% relative compaction.

Foundation Support

Existing soils below new foundations, and extending at least 5 feet beyond perimeter foundation lines, should be excavated to a depth of 18 inches below planned foundation bottom grades. The exposed surface should be scarified to a depth of at least 6 inches, moisture conditioned to within 2 percentage points above or below optimum moisture content and compacted to a minimum of 90% relative compaction. Excavated soils should be replaced and compacted as described above for compacted fill placement.

New structures can be supported on shallow spread footings with bottom levels at a minimum depth of 18 inches below the lowest adjacent finished grade. A minimum width of 18 inches is recommended for continuous footings. Isolated footings should be at least 24 inches wide. Footings can be designed for an allowable bearing pressure of 2000 pounds per square foot (psf) for dead plus long-term live loads. This value can be increased by ⅓ when considering the total of all loads, including wind or seismic forces.

Total post-construction settlement is estimated to be approximately ¾ inch. Post-construction differential settlements are anticipated to be ½ inch or less between isolated footings, and between the middle and end of a continuous footing.

Footing excavations should be observed by an MJ representative to check bearing materials and cleaning.

Lateral Loading

Resistance to lateral loads will be provided by passive earth pressure against the faces of footings and other structural elements below grade, and by friction along the bases of footings and slabs. Passive earth pressure can be taken as 350 pounds per square foot (psf) per foot of depth. Base friction can be taken as 0.35 times the actual dead load. Base friction and passive earth pressure can be combined without reduction. Retaining structures free to rotate at the top should be designed for an active equivalent fluid pressure of 35 psf per foot of height, plus any additional building or equipment surcharge. MJ should be notified if retaining walls greater than 10 feet in height, restrained walls, or tieback walls are planned so that geotechnical recommendations specific to wall conditions can be developed.

Slabs-on-Grade

Existing soils below new slabs-on-grade, and extending at least 5 feet beyond perimeter slab lines, should be excavated to a depth of 12 inches below planned slab bottom grades. The exposed surface should be scarified to a depth of at least 6 inches, moisture conditioned to within 2 percentage points above or below optimum moisture content and compacted to a minimum of 90% relative compaction. Excavated soils should be replaced and compacted as described above for compacted fill placement.

Slabs-on-grade should be underlain by a 4" thick blanket of clean, poorly graded, coarse sand or crushed rock. A moisture vapor retarder/barrier should be placed beneath slabs where floor coverings will be installed. Typically, plastic is used as a vapor retarder/barrier. If plastic is used, a minimum 10 mil is recommended. The plastic should comply with ASTM E 1745. Plastic installation should comply with ASTM E 1643.

Current construction practice typically includes placement of a 2-inch thick sand cushion between the bottom of the concrete slab and the moisture vapor retarder/barrier. This cushion can provide some protection to the vapor retarder/barrier during construction, and may assist in reducing the potential for edge curling in the slab during curing. However, the sand layer also provides a source of moisture vapor to the underside of the slab that can increase the time required to reduce moisture vapor emissions to limits acceptable for the type of floor covering placed on top of the slab. The floor-covering manufacturer should be contacted to determine the volume of moisture vapor allowable and any treatment needed to reduce moisture vapor emissions to acceptable limits for the particular type of floor covering to be installed.

Reinforcing for slabs-on-grade should consist of at least #3 bars at 12 inches on-center each way placed at mid-height in the slab. Reinforcing should extend down into the footings.

Surface Drainage

It is important that water be kept a minimum of 5 feet from structures and slabs. No ponding adjacent to buildings and structures should be allowed. Final surfaces should have a positive 2 percent minimum slope away from structures.

Retaining walls should be designed to resist hydrostatic pressures or be provided with a backdrain, weep holes or other drainage facilities. If a basement or underground structure is constructed, a subsurface drainage system is recommended.

Utility Excavations

Excavations should be made in accordance with California Administrative Code, Title 8, Industrial Relations, Chapter 4, Division of Industrial Safety, Subchapter 4, Construction Safety Orders, Article 6. Temporary excavations should be shored or sloped in accordance with Cal OSHA requirements. On-site soils can be considered Type C for purposes of excavation design.

In general, temporary excavations in on-site soils should be sloped no steeper than 1.5:1 for excavations up to 20 feet in depth. Compound excavations with vertical sides in lower portions should be properly shielded to a minimum height of 18 inches above the top of the vertical side, with the upper portion having a maximum slope of 1.5:1. A Registered Professional Engineer should design slopes or benching for excavations greater than 20 feet in depth.

Temporary excavation slopes should be inspected twice daily by the contractor’s competent person before personnel are allowed to enter the excavation. If sloughing, raveling or other evidence for slope instability is noted, corrective measures should be implemented.

Temporary shoring will be required for those excavations where temporary cut slopes as described above are not feasible. Cantilever shoring, and shoring with 1 level of bracing, can be designed to resist an equivalent fluid pressure of 30 psf per foot of depth. For shoring with multiple levels of bracing, a uniform lateral pressure equal to 25H in psf, where H is the height of shoring in feet, should be used. The recommended soil pressure applies to level soil conditions behind the shoring. Where a combination of sloped embankment and shoring is used, the soil pressure will be greater and should be evaluated for actual conditions.

Flexible Pavement Structural Sections

Existing soils below new flexible pavement structural sections and extending at least 5 feet beyond perimeter pavement lines except where constrained by property lines or existing improvements, should be excavated to a depth of 12 inches below planned the bottom of the pavement structural section. The exposed surface should be scarified to a depth of at least 6 inches, moisture conditioned to within 2 percentage points above or below optimum moisture content and compacted to a minimum of 95% relative compaction. Excavated soils should be replaced and compacted as described above for compacted fill placement.

Parking areas supporting automobiles and light trucks can be designed using a Traffic Index (TI) of 5. Main access roads and areas where occasional heavy trucks will pass can be designed for a TI of 7. The pavement support characteristics of on-site soils were evaluated by laboratory test. An R-value of 69 was measured in the test, indicating good pavement support characteristics.

Preliminary flexible pavement structural sections are summarized in the following table.

Traffic Index	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
5 (Parking Areas)	2	4
7 (Driveways and Aisles)	2-½	4

It is recommended that areas in front of trash containers where refuse trucks will make frequent stopping, backing, and turning movements be paved with portland cement concrete (PCCP). The PCCP should be at least 6 inches thick and underlain by at least 6 inches of Class 2 aggregate base or equivalent.

Limitations

The recommendations in this report are based on results of the field exploration and laboratory test programs, combined with interpolation and extrapolation of subsurface conditions between and beyond boring locations. The nature and extent of variations in these conditions may not become evident until construction. If variations are encountered during construction, MJ should be notified so these variations can be reviewed and the recommendations in this report modified if necessary. If changes in the nature, design or location of the structures are planned, these changes should be reviewed by MJ so that modifications to the recommendations in this report can be made if needed.

Our professional services have been performed using the degree of care and skill ordinarily exercised under similar circumstances by reputable engineering consultants practicing in this or similar localities. No other warranty, express or implied, is made as to the professional advice or data included in this report. This report has not been prepared for use by other parties, and may not contain sufficient information for purposes of other parties or other uses.