Engineering a Better Tomorrow

Geotechnical Evaluation Report

Proposed French Valley Public Library
31526 Skyview Road (APN 480-160-021)
Winchester, California

Prepared for:

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October 18, 2019
Project No.: 190759.3

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Subject: **Geotechnical Evaluation Report**
Proposed French Valley Public Library
31526 Skyview Road (APN 480-160-021)
Winchester, California

Dear Mr. Collins,

In accordance with your request and authorization, we are presenting the results of our geotechnical investigation for the Proposed French Valley Public Library project located at 31526 Skyview Road in Winchester, California (APN 480-160-021). The purpose of our investigation has been to evaluate the subsurface conditions at the site and to provide geotechnical engineering recommendations for the construction of the proposed project. This report was prepared in accordance with the requirements of the 2016 California Building Code.

Based on our findings, the proposed project is geotechnically feasible, provided that the recommendations in this report are incorporated into the design and are implemented during construction of the project.

We appreciate the opportunity to be of service on this project. Should you have any questions regarding this report or if we can be of further service, please do not hesitate to contact the undersigned.

Respectfully submitted,
TWINING, INC.

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1. INTRODUCTION

This report presents the results of the geotechnical investigation performed by Twining, Inc. (Twining) for the Proposed French Valley Public Library project located at 31526 Skyview Road in Winchester, California. A description of the site and the proposed development is provided in the following section. The objectives of this study have been to evaluate subsurface conditions at the site and to provide geotechnical recommendations for design and construction of the proposed development, including recommendations for foundations and earthwork.

2. PROJECT DESCRIPTION

The proposed project is to construct a single-story public library branch approximately 25,000 square feet on a portion of Assessor's Parcel Number (APN) 480-160-021 located at 31526 Skyview Road in Winchester, California. The location of the site is depicted on Figure 1 – Site Location Map. The approximate site coordinates are latitude 33.608773°N and longitude 117.108073°W, and the site is located on the Bachelor Mountain, California 7½-Minute Quadrangle, based on the United States Geological Survey (USGS) topographic map (USGS 2018).

The site is currently unpaved and unoccupied. It is bounded on the east by a creek and related rip rap embankment, a flood control easement, and a 100-year floodplain; on the south by Skyview Road, on the west and north by Winchester Road (Highway 79).

Proposed structures will consist of reinforced masonry block walls and structural steel and/or wood-framed truss roof systems and will be supported on reinforced concrete shallow foundations. It is also proposed to include other appurtenant improvements such as parking spaces, a stormwater infiltration basin, hardscape, light poles, and utility pipelines. The size and depth of the infiltration basin are to be determined.

The site plan and borings performed during this evaluation are shown in Figure 2 – Site Plan and Boring Location Map.

The site plan shows that a portion of the proposed building footprint will be built on an approximately 10-foot-high slope. A cut-and-fill transition is anticipated to occur below the building pad, due to the existing surface conditions discussed in Section 4.2 of this report. Approximately 10 feet of engineered fill will be placed to create a uniform building pad, which will create 2H:1V (horizontal : vertical) fill slopes up to 10 feet high along the north and east sides of the pad.

3. SCOPE OF WORK

Our scope of work included review of background information, pre-field activities and field exploration, laboratory testing, engineering analyses and report preparation. These tasks are described in the following subsections.

3.1. Literature Review

We reviewed readily available background data including published geologic maps, topographic maps, seismic hazard maps and literature, and flood hazard maps relevant to the subject site. Relevant information has been incorporated into this report.

3.2. Pre-Field Activities and Field Exploration

Before starting our exploration program, we performed a geotechnical site reconnaissance to observe the general surficial conditions at the site and to select field exploration locations. After exploration locations were delineated, Underground Service Alert was notified of the planned locations a minimum of 72 hours prior to excavation. The approximate locations of the borings are shown on Figure 2, Site Plan and Exploration Location Map.

The field exploration was conducted on September 30, 2019 and consisted of drilling, testing, sampling, and logging 4 exploratory hollow-stem-auger (HSA) borings (B-1 through B-4) and percolation testing in four hand-auger borings (P-1 through P-4). The HSA borings (B-1 through B-4) were advanced to approximate depths of 16.5 to 51.5 feet below ground surface (bgs) using a CME-85 truck-mounted drill rig equipped with 8-inch-diameter HSAs. The hand-auger borings (P-1 through P-4) were drilled to approximately 5 feet bgs for percolation testing. The approximate locations of the borings are shown on Figure 2, Site Plan and Boring Location Map.

Drive samples of the soils were obtained from the HSA borings using a Standard Penetration Test (SPT) sampler without room for liner and a modified California split spoon sampler. The samplers were driven using a 140-pound automatic hammer falling approximately 30 inches. The blow-counts to drive the samplers were recorded, and subsurface conditions encountered in the borings were logged by a Twining field engineer. Soil samples obtained from the borings were transported to Twining Laboratories for examination and testing.

Percolation tests were performed in the 5-foot hand-auger borings (P-1 through P-4) according to the boring percolation test guidance provided in the Riverside County Design Handbook for Low Impact Development Best Management Practices. Testing was performed to provide estimates of infiltration rate of the site soils for use in preliminary design of the stormwater infiltration facility.

Upon completion of drilling or percolation testing, the borings were backfilled by the drilling subcontractor using drilled soil cuttings.

Detailed descriptions of the field exploration, soils encountered during drilling, and the percolation tests are presented in Appendix A – Field Exploration.

3.3. Geotechnical Laboratory Testing

Laboratory tests were performed on selected samples obtained from the borings to aid in the soil classification and to evaluate the engineering properties of site soils. The following tests were performed in general accordance with ASTM standards:

- In-situ moisture and density;
- #200 Wash
- Atterberg Limits;
- Expansion Index;
- Maximum density and optimum moisture;
- Direct shear;
- Consolidation;
- R-Value; and

- Corrosivity.

Detailed laboratory test procedures and results are presented in Appendix B – Laboratory Testing.

3.4. Engineering Analyses and Report Preparation

We compiled and analyzed the data collected from our field exploration and laboratory testing. We performed engineering analyses based on our literature review and data from field exploration and laboratory testing programs. Our analyses included the following:

- Site geology, and subsurface conditions;
- Groundwater conditions;
- Geologic hazards and seismic design parameters;
- Liquefaction potential and seismic settlement;
- Soil corrosion potential;
- Soil collapse and expansion potential;
- Site preparation and earthwork;
- Foundation design parameters including bearing capacity, settlement, and lateral resistance;
- Modulus of subgrade reaction for slab design;
- Pole foundations for light poles, street lights and similar structures;
- Pavement section recommendations; and
- Stormwater infiltration rates.

We prepared this report to present our conclusions and recommendations from this investigation.

4. SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1. Regional Geology

According to the Morton Geologic Map of the Bachelor Mountain quadrangle (Morton, 2003), the site is underlain by very old alluvial valley deposits that are early to middle Pleistocene in age (geologic map symbol: $Qvov_a$) consisting of moderately to well-indurated, reddish-brown, mostly very dissected gravel, sand, silt, and clay-veering alluvium. A portion of the geologic map is reproduced as Figure 3 – Regional Geologic Map.

4.2. Surface and Subsurface Conditions

The site was vacant and unpaved at the time of our field exploration. Based on our review of aerial photos (Figure 4), it appears that the north portion of the site was cut between 2009 and 2011 to approximately 1,364 feet to 1,371 feet above mean sea level (msl), about 10 feet below adjacent ground surface with an average elevation of approximately 1375 feet msl. There are large trees along the slopes formed by the cut.

During our field investigation, we noticed a depressed area occupied by large trees in the proposed parking lot area in the southern portion of the site between boring P-1 and the proposed building, and another depressed area in the proposed infiltration facility area in the north portion of the site. In 2011, the depressed areas appeared as ponds on the 2011 aerial photo (Figure 4).

Subsurface conditions encountered during the field exploration consisted of interbedded layers of silt, clay, silty sand and clayey sand in the upper 20 feet and predominantly clay below 20 feet. The silt and clay layers were very stiff to hard, and the silty and clayey sand layers were dense to very dense.

4.3. Groundwater Conditions

During drilling, groundwater was encountered at approximately 30 to 45 feet bgs in our borings. In about two hours after the end of drilling, the water level rose to about 16 feet bgs, or approximate elevation 1,358 feet msl.

Historically high groundwater level at the project site is 10 to 20 feet bgs based on the Seismic Hazard Zone Report 120 of California Geological Survey (CGS) for the Bachelor Mountain quadrangle (CGS, 2018). Based on groundwater level data measured in 1968 in wells adjacent to the site in the California Water Data Library (CWDL), the groundwater level at the site in 1968 appeared at approximate elevation 1,355 feet msl. It may be assumed that the historic high groundwater at the site is 10 feet bgs or at elevation 1,365 feet msl.

Groundwater conditions may vary across the site due to stratigraphic and hydrologic conditions and may change over time as a consequence of seasonal and meteorological fluctuations, or of activities by humans at this and nearby sites.

5. GEOLOGIC HAZARD AND SEISMIC DESIGN CONSIDERATIONS

The site is located in a seismically active area, as is the majority of southern California, and the potential for strong ground motion in the project area is considered high during the design life of the proposed development. The hazards associated with seismic activity in the vicinity of the site area discussed in the following sections.

5.1. Surface Fault Rupture

As shown on Figure 5, the project site is not located within a State of California Earthquake Fault Zone (formerly known as a Special Studies Zone) or an area with the potential for earthquake-induced landslides (CGS, 2018). The nearest known active faults belong to the Elsinore fault zone located about 6.4 miles southwest of the site. Based on our review of geologic and seismologic literature and our site evaluation, it is our opinion that the likelihood of surface fault rupture and earthquake-induced landslides at the site during the life of the proposed improvements is low.

5.2. Landslides

The area of the project site is not within an area with the potential for earthquake-induced landslides. Considering the site is relatively flat and not close to significant slopes, the potential for earthquake-induced landslides to occur at the site is considered very low.

5.3. Liquefaction and Seismic Settlement Potential

The project site is not within a zone of required investigation for liquefaction according to CGS (2018). The Riverside Liquefaction Map shows liquefaction susceptibility of the site is low. Considering these results, the site subsurface conditions discussed above, and the site seismic shaking intensity discussed below, liquefaction potential at the site is considered low, and seismically induced settlement is negligible.

5.4. CBC Seismic Design Parameters

Based on the 2006 CGS Site Classification Map, the average shear wave velocity in the top 30 meters (or approximately 100 feet) of the soil profile ($V_{s,30}$) at the site is about 349 meters per second (or approximately 1,145 feet per second). Based on global $V_{s,30}$ from topographic slope (Wald & Allen 2008), the site $V_{s,30}$ is about 303 meters per second (or approximately 994 feet per second). The site $V_{s,30}$ values and the subsurface conditions discussed above suggest the site seismic class is D consisting of a stiff soil profile.

Our recommendations for seismic design parameters have been developed in accordance with the 2016 California Building Code (2016 CBC) and ASCE 7-10 (ASCE, 2010) standards. Table 1 presents the seismic design parameters for the site.

Table 1 – 2016 California Building Code Design Parameters

Design Parameters	Value
Site Class	D
Mapped Spectral Acceleration Parameter at Period of 0.2-Second, S_s (g)	1.5
Mapped Spectral Acceleration Parameter at Period 1-Second, S_1 (g)	0.6
Site Coefficient, F_a	1.0
Site Coefficient, F_v	1.5
Adjusted MCE_R[1] Spectral Response Acceleration Parameter, S_{MS} (g)	1.5
Adjusted MCE_R[1] Spectral Response Acceleration Parameter, S_{M1} (g)	0.9
Design Spectral Response Acceleration Parameter, S_{DS} (g)	1.0
Design Spectral Response Acceleration Parameter, S_{D1} (g)	0.6
Peak Ground Acceleration, PGA_M[2] (g)	0.544
Seismic Design Category	D
Notes: [1] Risk-Targeted Maximum Considered Earthquake [2] Peak Ground Acceleration adjusted for site effects	

Using the USGS Seismic Hazard Interactive Reaggregation Tool, a modal moment earthquake magnitude of 7.7 and a modal seismic source distance of 6.4 miles (10.3 kilometers) were obtained for a peak acceleration of 0.68 g at the site, which corresponds to a probability of exceedance of 2% in 50 years.

6. GEOTECHNICAL ENGINEERING RECOMMENDATIONS

Based on the results of our literature review and the field exploration, laboratory testing, and engineering analyses, it is our opinion that the proposed construction is feasible from a geotechnical standpoint, provided that the recommendations in this report are incorporated into the design plans and are implemented during construction.

6.1. General Considerations

Geotechnical engineering recommendations presented in this report for the proposed project are based on our understanding of the proposed development, subsurface conditions encountered during our field exploration, the results of laboratory testing on soil samples taken from the site, and our engineering analyses.

Key geotechnical considerations for the project are as follows:

- A cut/fill transition will occur under the building pad;
- Construction of the building pad will create a 10-foot-high 2H:1V fill slope;
- Subsurface materials consist predominantly of fine-grained soils;
- Relatively high groundwater at approximately 1,358 to 1,365 feet msl.

The following sections present our conclusions and recommendations pertaining to the engineering design for this project. If the design substantially changes, then our geotechnical engineering recommendations would be subject to revision based on our evaluation of the changes.

6.2. Soil Expansion and Collapse Potential

Based on our field exploration and laboratory test results, the risk of soil expansion and collapse is low at the site and will not adversely affect the design and construction of the project.

6.3. Corrosive Soil Evaluation

The potential for the near-surface on-site materials to corrode buried steel and concrete improvements was evaluated. Laboratory testing was performed on one selected near-surface soil to evaluate pH and electrical resistivity, as well as chloride and sulfate contents. The pH and electrical resistivity tests were performed in accordance with California Test 643, and the sulfate and chloride tests were performed in accordance with California Tests 417 and 422, respectively. These laboratory test results are presented in Appendix B.

In accordance with the County of Los Angeles (2014) criteria, corrosive soil is defined as the soil has minimum electrical resistivity less than 1,000 ohm-centimeters, or chloride concentration greater than 500 ppm, or sulfate concentration in soils greater than 2,000 ppm, or a pH less than 5.5.

6.3.1. Reinforced Concrete

Laboratory tests indicate that the soil has 205 ppm or 0.0205% of water soluble sulfate (SO_4) in soil by weight. Based on ACI 318, concrete in contact with the site soils will have a sulfate exposure class S0.

Test results indicate that the potential for chloride attack of reinforcing steel in concrete structures and pipes in contact with soil is negligible.

6.3.2. Buried Metal

A factor for evaluating corrosivity to buried metal is electrical resistivity. The electrical resistivity of a soil is a measure of resistance to electrical current. Corrosion of buried metal is directly proportional to the flow of electrical current from the metal into the soil. As resistivity of the soil decreases, the corrosivity generally increases. Test results indicate the site soils have minimum electrical resistivity value of 1,000 ohm-centimeters.

Correlations between resistivity and corrosion potential published by the National Association of Corrosion Engineers (NACE, 1984) indicate that the soils have severely corrosive potential to buried metals. As such, corrosion protection for metal in contact with site soils should be considered. Corrosion protection may include the use of epoxy or asphalt coatings. A corrosion specialist should be consulted regarding appropriate protection for buried metals and suitable types of piping.

6.4. Site Preparation and Earth Work

In general, earthwork should be performed in accordance with the recommendations presented in this report. Twining should be contacted for questions regarding the recommendations or guidelines presented herein.

6.4.1. Site Preparation

Site preparation should begin with the removal of utility lines, asphalt, concrete, vegetation, and other deleterious debris from areas to be graded. Tree stumps and roots should be removed to such a depth that organic material is not present. Clearing and grubbing should extend to the outside edges of the proposed excavation and fill areas. We recommend that unsuitable materials such as organic matter or oversized material be removed and disposed offsite. The debris and unsuitable material generated during clearing and grubbing should be removed from areas to be graded and disposed at a legal dump site away from the project area.

Tree stumps, roots, and potentially loose or soft materials are anticipated in the two depressed areas discussed in Section 4.2. The depth of removal of soil materials may be deeper in these areas in order to expose competent native soil.

6.4.2. Excavation and Subgrade Preparation

Temporary excavations for the project are expected. We anticipate that unsurcharged excavations with vertical side slopes less than 4 feet high will generally be stable; however, some sloughing of cohesionless sandy materials encountered at the site should be expected.

Where space is available, temporary, un-surcharged excavation sides over 4 feet in height should be sloped no steeper than an inclination of 1H:1V (horizontal:vertical). Where sloped excavations are created, the tops of the slopes should be barricaded so that vehicles and storage loads are away from the top edge of the excavated slopes with a distance at least equal to the height of the slopes. A greater setback may be necessary when considering heavy vehicles, such as concrete trucks and cranes. Twining should be advised of such heavy vehicle loadings so that specific setback requirements can be established. If the temporary construction slopes are to be maintained during the rainy season, berms are recommended to be graded along the tops of the slopes in order to prevent runoff water from entering the excavation and eroding the slope faces.

Excavations shall not undermine existing adjacent footings. We recommend that excavations for the proposed improvements do not encroach within a 1:1 plane projected from the top outside edge of any existing at-grade or below-grade existing facilities including foundations of existing structures, trenches, underground pipelines. Otherwise, shoring should be implemented to maintain foundation support of the adjacent facilities.

Undocumented fill was not encountered in our borings. However, if undocumented fill materials are encountered during excavations, those materials should be removed to the full depth of fill.

Slopes are anticipated during site grading. Fill placed on slopes should be properly benched and keyed into undisturbed native material. New fill placed against any existing approved fill slopes should be properly benched into the existing fill.

A cut/fill transition and a significant variation in the thickness of fill are anticipated across the building pad. Therefore, the pad should be over-excavated and recompacted a minimum of three feet below the bottom of footings to create a blanket of similar fill under the pad.

For minor structures and slabs-on-grade that are structurally separated from the building, the excavation should extend at least 2 feet below the finished grade or at least 1 feet below the bottom of the footing of the minor structures and slabs-on-grade, whichever is greater. Excavation for pavements and hardscape should be over-excavated at least 1 feet as measured from the bottom of the pavement or hardscape section.

Laterally, excavation should extend beyond the foundation limits a minimum distance equal to two feet or the depth of excavation, whichever is greater. Excavation for other improvements (e.g., concrete walkways, flatwork, pavement) should extend laterally at least two feet beyond the limits of the improvements.

The extent and depths of all removal should be evaluated by Twining's representative in the field based on the materials exposed. Should excavations expose soft or soils considered as unsuitable for use as fill by a Twining representative, additional removals may be recommended.

The exposed excavation bottom should be evaluated and approved by Twining. It should then be scarified to a minimum depth of 6 inches and moisture conditioned to achieve generally consistent moisture contents approximately 2 percent above the optimum moisture content. The scarified bottom should be compacted to at least 90 percent relative compaction in accordance with the latest version of ASTM Test Method D1557 and then evaluated and approved by Twining.

Fill and backfill materials should be compacted fill in accordance with Sections 6.4.3 and 6.4.4 of this report. Prior to placement of any fill, the geotechnical engineer or their representative should review the bottom of the excavation for conformance with the recommendations of this report.

Personnel from Twining should observe the excavations so that any necessary modifications based on variations in the encountered soil conditions can be made. All applicable safety requirements and regulations, including CalOSHA requirements, should be met. Stability of temporary excavations is the responsibility of the contractor.

6.4.3. Materials for Fill

In general, most on-site soils are considered as suitable for use as engineered fill. All fill soils should be free of organics, debris, rocks or lumps over three inches in largest dimension, other deleterious material, and not more than 40 percent larger than ¾ inch. Larger chunks, if generated during excavation, may be broken into acceptably sized pieces or may be disposed offsite.

Any imported fill material should consist of granular soil having a “very low” expansion potential (i.e., expansion index of 20 or less). Import material should also have low corrosion potential (that is, chloride content less than 500 parts per million [ppm], soluble sulfate content of less than 0.1 percent, and pH of 5.5 or higher).

All fill soils should be evaluated and approved by a Twining representative prior to importing or filling.

6.4.4. Compacted Fill

Unless otherwise recommended, the exposed excavation bottom to receive fill should be prepared in accordance with Section 6.4.2 of this report. Prior to placement of compacted fill, the contractor should request Twining to evaluate the exposed excavation bottoms.

Compacted fill should be placed in horizontal lifts of approximately 8 to 10 inches in loose thickness, depending on the equipment used. Prior to compaction, each lift should be moisture conditioned, mixed, and then compacted by mechanical methods. The moisture content should be approximately 2 percent above the optimum moisture content. Fill materials should be compacted to a minimum relative compaction of 95 percent within the upper one foot below new vehicle trafficked pavement sections, and 90 percent in all other areas. The relative compaction should be determined by ASTM D1557. Successive lifts should be treated in the same manner until the desired finished grades are achieved.

6.4.5. Excavation Bottom Stability

In general, we anticipate that bottoms of the excavations will be stable and should provide suitable support for the proposed improvements. Conditions of the excavation bottom should be evaluated by Twining during the scarification and re-compaction efforts. If unstable bottom conditions are encountered, remedial measures would be required to stabilize the bottom. Soft bottom conditions can be identified by surface yielding under rubber-tired equipment loading and the inability to achieve proper compaction.

Unstable bottom conditions may be mitigated by over-excavation of the bottom to suitable depths, and/or replacement with a minimum 1-foot-thick aggregate base underlain by geogrid (Tensar TX7 or equivalent).

As an alternative, excavation bottom stabilization may be achieved by cement treatment for the upper 15 inches below the bottom according to Section 6.4.6 of this report.

Recommendations for stabilizing excavation bottoms should be based on evaluation in the field by the geotechnical consultant at the time of construction.

6.4.6. Cement Treatment

Cement treatment, if needed, should be performed according the following processes under the guidance of a Twining Geotechnical engineer:

- Upon achieving rough grade, cement powder is spread on the surface at a rate that is dependent upon the thickness of the treated section. We recommend cement-treatment by 5 to 7 percent cement (by dry weight). The cement powder is then dry mixed with the pulverizer into the subgrade to a depth of at least 12 inches below the rough grade surface. From the time the material is wet mixed, the material should be fully compacted within no more than 2 hours.

- Compaction is performed using a large sheepsfoot compactor. Depending on the type of equipment, a section as thick as 18 inches can be compacted in one lift. The type of equipment proposed for use should be approved by the engineer based on the lift thickness prior to bringing the equipment on site. The cement-treated section should be compacted to 92 percent of the maximum density as determined by ASTM D 1557.

- Upon completion of compaction with the sheepsfoot compactor, the surface is bladed and finish-rolled with a smooth drum roller.

- The surface of the treated material is wetted at least twice daily (possibly more depending on weather) to promote hydration of the cement.

- For at least 24 hours, traffic on the surface after completion of compaction should be minimized to the maximum extent possible and heavy construction equipment traffic should be completely avoided to prevent breakdown of the treated material prior to the curing process being completed. After 24 hours, the surface can be proof-rolled and checked for yielding under heavy rubber-tire vehicle loads (such as a fully-loaded water truck). If the surface indicates signs of yielding or instability, an additional 24 hours of cure time should be implemented while again minimizing traffic loading

6.4.7. Backfill for Utility Trench

Utility trench excavations to receive backfill shall be free of trash, debris or other unsatisfactory materials at the time of backfill placement.

At locations where the trench bottom is yielding or otherwise unstable, pipe support may be improved by placing 12 inches of ¾-inch crushed rock as defined in Section 200-1.2 of the "Greenbook" Standard Specifications for Public Works Construction. Remedial earthwork at the trench bottom should be performed where oversize materials (rocks or clods greater than 3 inches) are present. Removal of oversize materials to a depth of 6 inches below the bottom of the pipeline and replacement with fill compacted to at least 90% relative compaction is recommended. Alternatively, ¾-inch crushed rock may be used.

The trench should be bedded with clean sand extending to at least one foot over the top of pipe. Pipe bedding as specified in SSPWC can be used. Bedding material should consist of clean sand having a sand equivalent (SE) of 30 or greater. Alternative materials meeting the intent of the bedding specifications are also acceptable. Samples of materials proposed for use as bedding should be provided to the engineer for inspection and testing before the material is imported for use on the project. The onsite materials can only be used following the requirement of "Greenbook" bedding specification when the SE is not less than 30. The pipe bedding material should be placed over the full width of the trench. After placement of the pipe, the bedding should be brought up uniformly on both sides of the pipe to reduce the potential for unbalanced loads. No void or uncompacted areas should be left beneath the pipe haunches.

Above pipe bedding, trench backfill may be onsite soils and should not contain rocks or lumps over 3 inches in largest dimension. Larger chunks, if generated during excavation, may be broken into acceptably sized pieces or may be disposed offsite. The moisture content should be approximately 2 percent above the optimum moisture content.

Backfill may be placed and compacted by mechanical means and should be compacted to 90 percent of the laboratory maximum dry density as per ASTM Standard D1557. Where pavement is planned, the top 12 inches of subgrade soils and the overlying aggregate base should be compacted to 95 percent.

Jetting or flooding of pipe bedding and backfill material is not recommended.

6.4.8. Rippability

The earth materials underlying the site should be generally excavatable with heavy-duty earthwork equipment in good working condition. Some gravels, cobbles and man-made debris should be anticipated.

6.4.9. Construction Dewatering

As discussed earlier, groundwater was at approximately 1,358 feet msl. Construction of the project is anticipated to occur above the groundwater. The possibility to encounter groundwater is low during earthwork and foundation preparation for the proposed structures, and the need for dewatering is not anticipated for construction of structures and utility trenches.

If needed, considerations for construction dewatering should include anticipated drawdown, volume of pumping, potential for settlement of nearby structures, and groundwater discharge. Disposal of groundwater should be performed in accordance with guidelines of the Regional Water Quality Control Board.

6.5. Foundation Recommendations

Based upon the excavation/over-excavation and backfill recommendations, the proposed structures may be supported on continuous strip footings or isolated footings designed in accordance with the geotechnical recommendations presented below. Structural design of foundations should be performed by the structural engineer and should conform to the 2016 California Building Code.

6.5.1. Building Foundation Bearing Capacity and Settlement

Footings for the building should be placed on the subgrade prepared in accordance the requirements for the building pad as described in Section 6.4. Geotechnical design parameters for these footings presented in Table 2 may be used, assuming less than 25 kips on shallow spread footings and less than 5 kips per lineal foot on perimeter foundations. Twining should be contacted for footing dimensions, allowable bearing pressures, and settlements that are outside the indicated applicable ranges.

The total lateral resistance can be taken as the sum of the friction at the base of the footing and passive resistance. The upper one foot of soil should be neglected when calculating the passive resistance. The passive resistance value may be increased by one-third when transient loads from wind or earthquake.

Table 2 - Geotechnical Design Parameters for Shallow Foundations

Minimum Footing Dimensions	• Continuous footings: 12 inches in width. • Square footings: 24 inches in width. • Minimum embedment: 12 inches measured from the lowest adjacent grade to the bottom of the footing.
Allowable Bearing Pressure	• Footings should be supported on at least 3 feet of compacted fill. • Continuous footings: an allowable bearing pressure of 2,500 pounds per square foot (psf) may be used. The allowable may be increased by 75 psf for each additional foot of width and 220 psf for each additional foot of embedment, up to a maximum allowable capacity of 3,000 psf. • Square footings: an allowable bearing pressure of 3,000 psf may be used. The allowable may be increased by 60 psf for each additional foot of width and 220 psf for each additional foot of embedment, up to a maximum allowable capacity of 4,000 psf. • The allowable bearing values may be increased by one-third for transient loads from wind or earthquake.
Estimated Static Settlement	• Approximately one inch of total settlement with differential settlement estimated to be on the order of ½ inches over 50 feet. • Most static settlement of foundation system is expected to occur immediately upon application of loading. Long term total and differential settlement is expected to be less than one inch and ½ inches, respectively.
Allowable Coefficient of Friction Below Footings	0.30
Allowable Lateral Passive Resistance	Increases with depth at a rate of 200 psf per foot (200 pcf equivalent fluid pressure)

6.6. Retaining Walls

Recommendations for wall lateral loads, backfill, and drainage are provided below. Lateral resistance may be based on 6.5 of this report. Retaining walls should be designed to have a factor of safety of 1.5 for static stability and 1.1 for stability due to transient loads from wind or seismic.

6.6.1. Backfill and Drainage of Walls

The backfill material behind walls should consist of granular non-expansive material and be approved by the project geotechnical engineer. Based on the soil materials encountered during our exploration, some on-site soils will meet this requirement.

Wall backfill should be adequately drained. Adequate backfill drainage is essential to provide a free-drained backfill condition and to limit hydrostatic buildup behind walls. Drainage behind walls may be provided by a geosynthetic drainage composite such as TerraDrain, MiraDrain, or equivalent, attached to the outside perimeter of the wall and installed in accordance with the manufacturer's recommendations. The drainage system should meet the minimum requirements of Sections 1805.4.2 and 1805.4.3 of 2016 CBC.

6.6.2. Lateral Earth Pressure

The values presented below assume that the supported grade is level and that surcharge loads are not applied. The recommended design lateral earth pressure is calculated assuming that a drainage system will be installed behind retaining walls in accordance with Sections 1805.4.2 and 1805.4.3 of 2016 CBC and that external hydrostatic pressure will not develop behind the walls. Where wall backfill does not have adequate drainage, the full hydrostatic pressure should be added to the lateral earth pressures provided below in design.

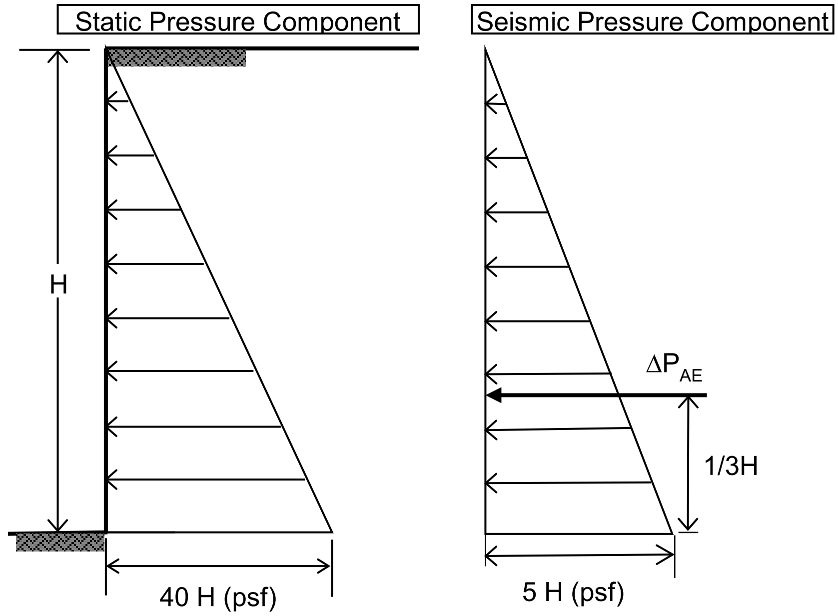
Walls that are free to move and rotate at the top (such as cantilevered walls) and have adequate drainage may be designed for the active earth pressure equivalent to a fluid weighting 50 pcf.

Walls that are restricted to move horizontally at the top (such as by a floor deck) and have adequate drainage may be designed for the "at-rest" earth pressure equivalent to a fluid weighing 72 pcf.

Vertical surcharge loads within a 1:1 plane projected from the bottom of the wall distributed over retained soils should be considered as additional uniform horizontal pressures acting on the wall. These additional pressures can be estimated as approximately 40% and 60% of the magnitude of the vertical surcharge pressures for the "active" and "at-rest" conditions, respectively.

6.6.3. Seismic Lateral Earth Pressure

Walls retaining more than 6 feet high earth should be designed for seismic lateral earth pressure. The seismic pressure distribution may be considered a triangle with the maximum pressure at the bottom. The combination of static and incremental seismic pressures shown in the following diagram may be used for seismic design for both cantilever and restrained walls.



where H is in feet

Seismic Earth Pressure Distribution on Walls

6.7. Concrete Slabs

Slabs should be supported on non-expansive engineered fill in accordance with Section 6.4 of this report. For design of concrete slabs, a base modulus of subgrade reaction (*k*) of 150 pounds per cubic inch (pci) may be used provided it is modified by the formulas below based on slab dimensions.

$$k_1 = 150\ pci$$

$$k(BxB) = k_1 \left(\frac{B+1}{2B}\right)^2$$

$$k(BxL) = k_{BxB}\left(\frac{1+0.5\ {}^{B}/_{L}}{1.5}\right)$$

Where:

$$k_1 = Modulus\ for\ 1x1\ plate$$

$$B = Width\ of\ Square\ Foundation$$

$$L = Length\ of\ Rectangular\ Foundation$$

Floor slabs should be designed and reinforced in accordance with the structural engineer's recommendations. In moisture sensitive areas, the floor slabs should be dampproofed in accordance with Section 1805.2 of 2016 CBC. Specific recommendations can be provided by a waterproofing consultant.

6.8. Fence Poles and Sign Posts

The Project may involve fence poles and sign posts. Geotechnical recommendations for conditions with and without lateral constraint provided at the ground surface conditions are provided below based on 2016 CBC.

6.8.1. Non-Constrained Ground

The embedment of sign posts where no lateral constraint is provided at or above the ground surface should be calculated using Equation 18-1 of 2016 CBC (shown below) or a minimum 3 feet below the ground surface, whichever is deeper.

$$\mathrm{d} = \frac{\mathrm{A}}{2}\left(1 + \sqrt{1 + \frac{4.36\mathrm{h}}{\mathrm{A}}}\right) \qquad \text{(Equation 18-1 of 2016 CBC)}$$

where:

- A = $2.34P/(S_1 * b)$
- b = Diameter of round post or footing or diagonal dimension of square post or footing, feet
- d = Depth of embedment in earth in feet but not over 12 feet for purpose of computing lateral pressure.
- h = Distance in feet from ground surface to point of application of "P".
- P = Applied lateral force in pounds.
- S_1 = Allowable lateral soil-bearing pressure based on a depth of one-third the depth of embedment in pounds per square foot.

An allowable passive earth pressure of 200 pcf up to a maximum of 2,000 psf may be used for design provided the upper one foot of passive resistance is neglected in the structural design.

6.8.1. Constrained Ground

The embedment of sign posts where lateral constraint is provided at the ground surface, such as by a rigid floor or pavement, should be calculated using Equation 18-2 of 2016 CBC (shown below) or a minimum 3 feet below the ground surface, whichever is deeper.

$$d = \sqrt{\frac{4.24Ph}{S_3 b}} \qquad \text{(Equation 18-2 of 2016 CBC)}$$

where:

- b = Diameter of round post or footing or diagonal dimension of square post or footing, feet
- d = Depth of embedment in earth in feet but not over 12 feet for purpose of computing lateral pressure.
- h = Distance in feet from ground surface to point of application of "P".

P = Applied lateral force in pounds.

S_3 = Allowable lateral soil-bearing pressure based on a depth of one-third the depth of embedment in pounds per square foot.

An allowable passive earth pressure of 200 pcf up to a maximum of 2,000 psf may be used for design provided the upper one foot of passive resistance is neglected in the structural design.

6.9. Flexible Pavement Design

Our pavement structural design is in accordance with Chapter 630 of the Caltrans Highway Design Manual, which is based on a relationship between the gravel equivalent (GE) of the pavement structural materials, the traffic index (TI), and the R-value of the underlying subgrade soil. Our laboratory test results indicate an R value of 12, which was used in our asphalt pavement structural calculations. On this basis, Table 3 provides recommended minimum thicknesses for hot mix asphalt (HMA) and aggregate base sections for different traffic indices. These minimum thicknesses may be adjusted based on additional R-value tests during construction.

The asphalt pavement section should be constructed on top of properly prepared subgrade in accordance with Section 6.4 of this report and aggregate base section compacted to 95 percent of the maximum dry density in accordance with ASTM D1557.

Table 3 – Recommended Minimum HMA and Base Section Thicknesses

Traffic Index	5.0	6.0	7.0
HMA Thickness (in)	4.0	4.0	5.0
Aggregate Base Thickness (in)	7.0	11.0	12.0

6.10. Rigid Pavement Design

For preliminary design of rigid pavement section, Table 4 provides minimum thicknesses for Jointed Plain Concrete Pavement (JPCP) section and Class 2 Aggregate Base (AB) section for different traffic indices. Final design of rigid pavement should be performed by the project Civil Engineer based on field observations and additional R-value tests during construction. The subgrade should be prepared in accordance with Section 6.4.2 of this report. The AB section should be compacted to 95 percent of the maximum dry density in accordance with ASTM D1557.

Table 4 – Recommended Rigid Pavement Minimum Thicknesses

Traffic Index	5.0	6.0	7.0
JPCP Thickness (in)	4	5.5	7.0
Aggregate Base Thickness (in)	4	4	4
Maximum Joint Spacing (feet)	15.0	15.0	15.0

The above pavement section is based on a minimum 28-day concrete compressive strength of 3,500 psi. Positive drainage should be provided away from all pavement areas to prevent seepage of surface and/or subsurface water into the pavement base and/or subgrade.

6.11. Stormwater Infiltration Facility

The design of stormwater infiltration facility should be based on percolation test results with an appropriate factor of safety.

Our percolation test results may be used in preliminary design. Details of the percolation tests are presented in Appendix A. Infiltration rates with a factor of safety of 3 from our percolation tests are summarized in Table 5. The proposed infiltration facility should have a minimum setback from property lines and foundations recommended in Table 6.

However, the Riverside County requires a minimum of 10 feet between the bottom of the infiltration facility and the historical high groundwater. The historic high groundwater is about 10 feet bgs at the site, and thus site does not appear suitable for the proposed infiltration facility.

Table 5 – Infiltration Rate with a Factor of Safety of 3

Test Location	Depth of Test Borehole (feet)	Design Infiltration Rate (inch/hour)
P-1	5	Testing was abandoned due to negligible water level drop during pre-soaking
P-2	5	
P-3	5	
P-4	5	1.2

Table 6 – Recommended Minimum Infiltration Facility Setback

Setback from	Distance
Property lines	10 feet
Foundations	15 feet or outside of 1:1 plane drawn up from the bottom of foundation, whichever is greater.

6.12. Drainage Control

The control of surface water is essential to the satisfactory performance of the building and site improvements. Surface water should be controlled so that conditions of uniform moisture are maintained beneath the improvements, even during periods of heavy rainfall. The following recommendations are considered minimal:

- Ponding and areas of low flow gradients should be avoided.
- If bare soil within 5 feet of the structure is not avoidable, then a gradient of 5 percent or more should be provided sloping away from the improvement. Corresponding paved surfaces should be provided with a gradient of at least 1 percent.

- The remainder of the unpaved areas should be provided with a drainage gradient of at least 2 percent.
- Positive drainage devices, such as graded swales, paved ditches, and/or catch basins should be employed to accumulate and to convey water to appropriate discharge points.
- Concrete walks and flatwork should not obstruct the free flow of surface water.
- Brick flatwork should be sealed by mortar or be placed over an impermeable membrane.
- Area drains should be recessed below grade to allow free flow of water into the basin.
- Enclosed raised planters should be sealed at the bottom and provided with an ample flow gradient to a drainage device. Recessed planters and landscaped areas should be provided with area inlet and subsurface drain pipes.
- Planters should not be located adjacent to the structures wherever possible. If planters are to be located adjacent to the structures, the planters should be positively sealed, should incorporate a subdrain, and should be provided with free discharge capacity to a drainage device.
- Planting areas at grade should be provided with positive drainage. Wherever possible, the grade of exposed soil areas should be established above adjacent paved grades. Drainage devices and curbing should be provided to prevent runoff from adjacent pavement or walks into planted areas.
- Gutter and downspout systems should be provided to capture discharge from roof areas. The accumulated roof water should be conveyed to off-site disposal areas by a pipe or concrete swale system.

Landscape watering should be performed judiciously to preclude either soaking or desiccation of soils. The watering should be such that it just sustains plant growth without excessive watering. Sprinkler systems should be checked periodically to detect leakage and they should be turned off during the rainy season.

6.13. Slope Stability

Slope stability analyses were performed to evaluate the static and seismic stability of the fill slopes. Seismic stability was evaluated using the pseudo-static method with a horizontal seismic coefficient of 0.15. Results of the analysis shown in Appendix C indicate that the slopes have adequate factors of safety.

It should be noted that a small portion of the toe of the slope at the east corner extends to the 100-year floodplain. It is recommended that riprap be placed against the toe as a protection against the 100-year flood event.

7. DESIGN REVIEW AND CONSTRUCTION MONITORING

Geotechnical review of plans and specifications is of paramount importance in engineering practice. The poor performance of many structures has been attributed to inadequate geotechnical review of construction documents. Additionally, observation and testing of the subgrade will be important to the performance of the proposed development. The following sections present our recommendations relative to the review of construction documents and the monitoring of construction activities.

7.1. Plans and Specifications

The design plans and specifications should be reviewed by Twining, Inc. prior to bidding and construction, as the geotechnical recommendations may need to be reevaluated in the light of the actual design configuration and loads. This review is necessary to evaluate whether the recommendations contained in this report and future reports have been properly incorporated into the project plans and specifications. Based on the work already performed, this office is best qualified to provide such review.

7.2. Construction Monitoring

Site preparation, removal of unsuitable soils, assessment of imported fill materials, fill placement, foundation installation, and other site grading operations should be observed and tested, as appropriate. The substrata exposed during the construction may differ from that encountered in the test excavations. Continuous observation by a representative of Twining, Inc. during construction allows for evaluation of the soil conditions as they are encountered and allows the opportunity to recommend appropriate revisions where necessary.

8. LIMITATIONS

The recommendations and opinions expressed in this report are based on Twining, Inc.'s review of available background documents, on information obtained from field explorations, and on laboratory testing. It should be noted that this study did not evaluate the possible presence of hazardous materials on any portion of the site. In the event that any of our recommendations conflict with recommendations provided by other design professionals, we should be contacted to aid in resolving the discrepancy.

Due to the limited nature of our field explorations, conditions not observed and described in this report may be present on the site. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation and laboratory testing can be performed upon request. It should be understood that conditions different from those anticipated in this report may be encountered during grading operations, for example, the extent of removal of unsuitable soil, and that additional effort may be required to mitigate them.

Site conditions, including groundwater elevation, can change with time as a result of natural processes or the activities of man at the subject site or at nearby sites. Changes to the applicable laws, regulations, codes, and standards of practice may occur as a result of government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which Twining, Inc. has no control.

Twining's recommendations for this site are, to a high degree, dependent upon appropriate quality control of subgrade preparation, fill placement, and foundation construction. Accordingly, the recommendations are made contingent upon the opportunity for Twining to observe grading operations and foundation excavations for the proposed construction. If parties other than Twining are engaged to provide such services, such parties must be notified that they will be required to assume complete responsibility as the geotechnical engineer of record for the geotechnical phase of the project by concurring with the recommendations in this report and/or by providing alternative recommendations.

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This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Twining should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

This report has been prepared for the exclusive use by the client and its agents for specific application to the proposed project. Land use, site conditions, or other factors may change over time, and additional work may be required with the passage of time. Based on the intended use of this report and the nature of the new project, Twining may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the Client or anyone else will release Twining from any liability resulting from the use of this report by any unauthorized party.

Twining performed its evaluation using the degree of care and skill ordinarily exercised under similar circumstances by reputable geotechnical professionals with experience in this area in similar soil conditions. No other warranty, either express or implied, is made as to the conclusions and recommendations contained in this report.

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FIGURES

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APPENDIX A
FIELD EXPLORATION

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Appendix A
Field Exploration

General

The subsurface exploration program for the proposed project consisted of drilling, testing, sampling and logging four hollow-stem-auger (HSA) exploratory borings (B-1 through B-4) and percolation testing in four hand-auger borings (P-1 through P-4) at the site on September 30, 2019.

The HSA Borings (B-1 through B-4) were advanced to depths of approximately 16½ to 51½ feet below ground surface (bgs). Drilling operation for the HSA borings was performed using a truck-mounted CME-85 hollow-stem-auger drill rig by Baja Exploration of Escondido, California. Borings P-1 through P-4 were advanced to a depth of approximately 5 feet bgs using a 5-inch diameter hand auger.

The approximate locations of the borings are shown on Figure 2, Site Plan and Boring Location Map.

Drilling and Sampling

An explanation of the boring logs is presented as Figure A-1. The boring logs are presented as Figures A-2 through A-7. The boring logs describe the earth materials encountered, samples obtained, and show the field and laboratory tests performed. The logs also show the boring number, drilling date, and the name of the logger and drilling subcontractor. The borings were logged by an engineer using the Unified Soil Classification System. The boundaries between soil types shown on the logs are approximate because the transition between different soil layers may be gradual. Drive and bulk samples of representative earth materials were obtained from the borings.

Disturbed samples were obtained from selected depths using a Standard Penetration Test (SPT) sampler. This sampler consists of a 2-inch O.D., 1.4-inch I.D. split barrel shaft without room for liner. Soil samples obtained by the SPT sampler were retained in plastic bags. A California modified sampler was also used to obtain drive samples of the soils from selected depths. This sampler consists of a 3-inch outside diameter (O.D.), 2.4-inch inside diameter (I.D.) split barrel shaft. The samples were retained in brass rings for laboratory testing.

When the boring was drilled to the selected depth, the sampler was lowered to the bottom of the boring and then driven a total of 18-inches into the soil using an automatic hammer weighing 140 pounds dropped from a height of approximately 30 inches. The number of blows required to drive the samplers the final 12 inches is presented on the boring logs.

Upon completion of the borings, the boreholes were backfilled with drilled soil cuttings.

Percolation Testing

Percolation testing was performed on September 30, 2019 in the 5-foot-deep borings (P-1 through P-4) in accordance with the procedures of the Riverside County Design Handbook for Low Impact Development Best Management Practices. After installing pipe and filter rock, the boreholes were filled with water to approximately one foot bgs and presoaked for two consecutive 25-minute sessions prior to testing. At the end of each presoak session, water level change in borings P-1 through P-3 was negligible, and the testing was terminated. In P-4, water level change in boring was less than 6 inches.

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After presoaking, the boreholes were filled with water to depths approximately 0.9 to 1.9 feet bgs. Measurements were recorded at 10-minute intervals for a total of 7 readings. The last reading was used to determine the percolation rate at each test location.

Our calculated design infiltration rates are presented in Table A-1 below with a factor safety of 3. Detailed test data is attached at the end of this appendix.

Table A-1 – Design Infiltration Rates with a Factor of Safety of 3

Test Location	Depth of Test Borehole (feet)	Design Infiltration Rate (inch/hour)
P-1	5	Testing was abandoned due to negligible water level drop during pre-soaking
P-2	5	
P-3	5	
P-4	5	1.2

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APPENDIX B
LABORATORY TESTING

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Appendix B
Laboratory Testing

Laboratory Moisture Content and Density Tests

The moisture content and dry densities of selected driven samples obtained from the exploratory borings were evaluated in general accordance with the latest version of ASTM D 2937. The results are shown on the boring logs in Appendix A, and also summarized in Table B-1.

No. 200 Wash Sieve

The amount of fines passing the No. 200 sieve was evaluated in accordance with ASTM D 1140. The results are presented in Table B-2.

Atterberg Limits

Tests were performed on selected representative fine-grained soil samples to evaluate the liquid limit, plastic limit, and plasticity index in general accordance with ASTM D 4318. These test results were utilized to evaluate the soil classification in accordance with the Unified Soil Classification System. The test results are summarized in on Figure B-1 and Table B-3.

Resistance Value (R-value)

R-value testing was performed on a select bulk sample of the near-surface soils encountered at the site. The test was performed in general accordance with ASTM D 2844. The results are summarized in Table B-4.

Expansion Index

The expansion index of a select soil sample was evaluated in general accordance with ASTM D 4829. The specimen was molded under a specified compactive energy at approximately 50 percent saturation. The prepared 1-inch thick by 4-inch diameter specimen was loaded with a surcharge of 144 pounds per square foot and was inundated with tap water. Readings of volumetric swell were made for a period of 24 hours. The result of Expansion Index test is presented in Table B-5.

Direct Shear

Direct shear tests were performed on a remolded sample and select modified-California soil samples in general accordance with the latest version of ASTM D 3080 to evaluate the shear strength characteristics of the selected materials. The remolded sample was prepared to a relative compaction of 90% according to the maximum density as determined by ASTM D1557. The samples were inundated during shearing to represent adverse field conditions. Test results are presented on Figures B-2 through B-4.

Maximum Density and Optimum Moisture

A Modified Proctor test was performed on near-surface soils to determine the maximum dry density and optimum water content for compaction. The test was performed in accordance with ASTM D 1557 Method A. The curve is attached to this appendix as Figure B-5.

Consolidation

Consolidation tests were performed on select modified-California soil samples in general accordance with the latest version of ASTM D2435. The samples were inundated during testing

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to represent adverse field conditions. The percent consolidation for each load cycle was recorded as a ratio of the amount of vertical compression to the original height of the sample. The results of the tests are attached to this appendix. The tests were performed by Twining and Hushmand Associates, Inc. (HAI) of Irvine, California. The test results are presented in Figure B-6 and the HAI report included in this appendix.

Corrosivity

Soil pH and resistivity tests were performed by Anaheim Test Lab, Inc. (ATLI) of Anaheim, California on a representative soil sample. The resistivity of the soil assumes saturated soil conditions. The chloride and sulfate contents of the selected samples were evaluated in general accordance with the latest versions of Caltrans test methods CT417, CT422, and CT 643. The test results are presented on Table B-6 and the ATLI report included in this appendix.

Table B-1
Moisture Content and Dry Density

Boring No.	Depth (feet)	Moisture Content (%)	Dry Density (pcf)
B-1	5	9.8	125.0
B-1	15	16.0	115.9
B-1	25	25.1	104.2
B-1	35	14.5	112.3
B-1	45	17.2	112.4
B-2	10	20.9	101.1
B-3	5	5.5	126.9
B-3	15	26.3	99.0
B-4	10	7.5	121.2
B-4	20	16.5	114.8
B-4	30	22.3	105.0
B-4	40	15.2	116.9
B-4	50	13.0	118.7

Table B-2
Number 200 Wash Results

Boring No.	Depth (feet)	Percent Passing #200
B-1	0-5	67.5
B-1	20	73.2
B-1	30	43.4
B-2	5	50.9
B-4	15	69.0

Table B-3
Atterberg Limits Results

Boring No.	Depth (feet)	Liquid Limit	Plastic Limit	Plasticity Index	U.S.C.S. Classification
B-1	20	33	17	16	CL
B-1	30	32	14	18	CL
B-2	5	25	13	12	CL
B-4	15	42	14	28	CL

Table B-4
Resistance Value (R-value)

Boring No.	Depth (feet)	R Value
B-1	0 – 5	12

Table B-5
Expansion Index

Boring No.	Depth (feet)	Expansion Index	Expansion Potential
B-3	0 – 5	42	low

Table B-6
Corrosivity Test Results

Boring No.	Depth (feet)	pH	Water Soluble Sulfate (ppm)	Water Soluble Chloride (ppm)	Minimum Resistivity (ohm-cm)
B-1	0-5	7.4	205	106	1,000

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Appendix C
Slope Stability Analysis

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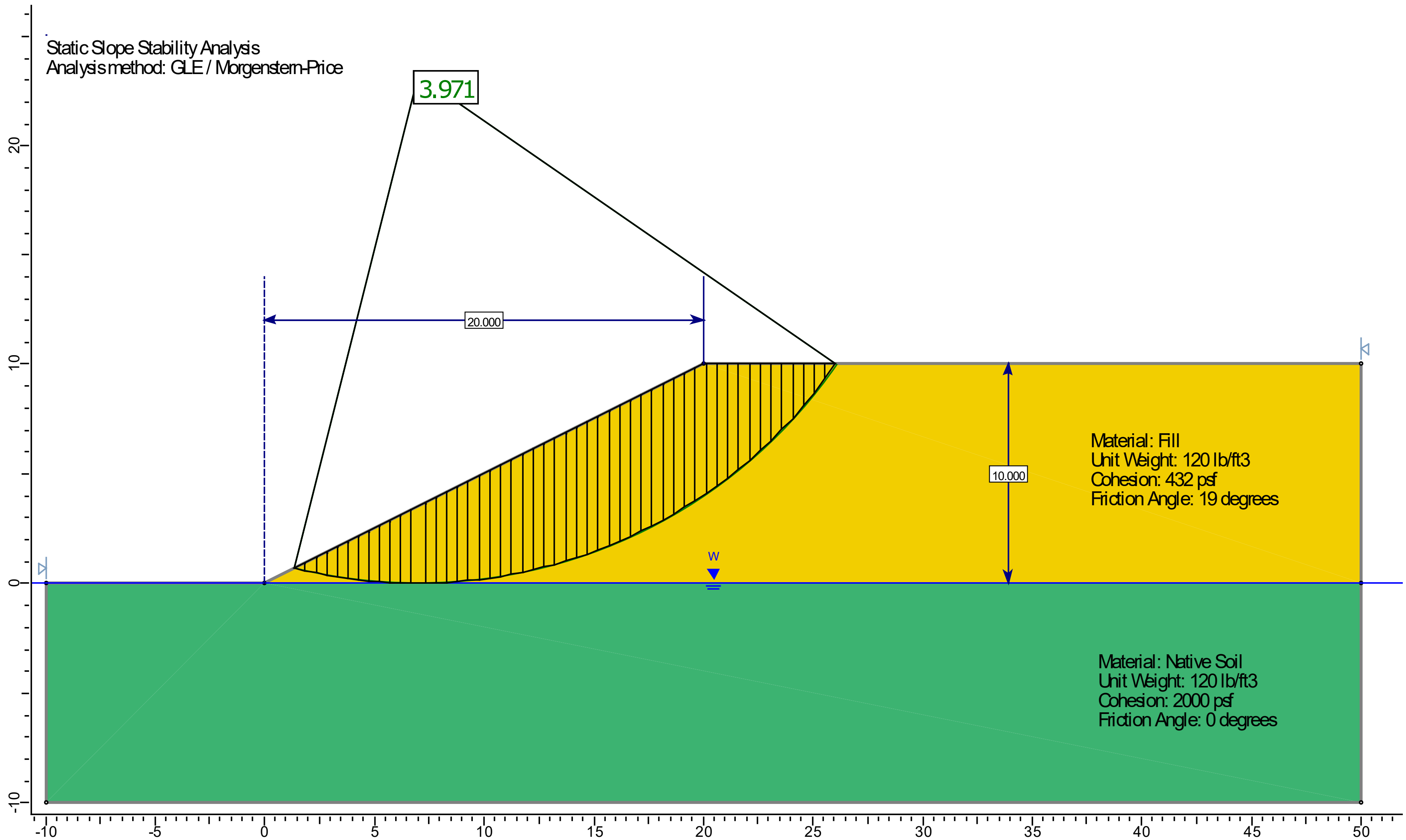

Figure C-1 Static Slope Stability Analysis

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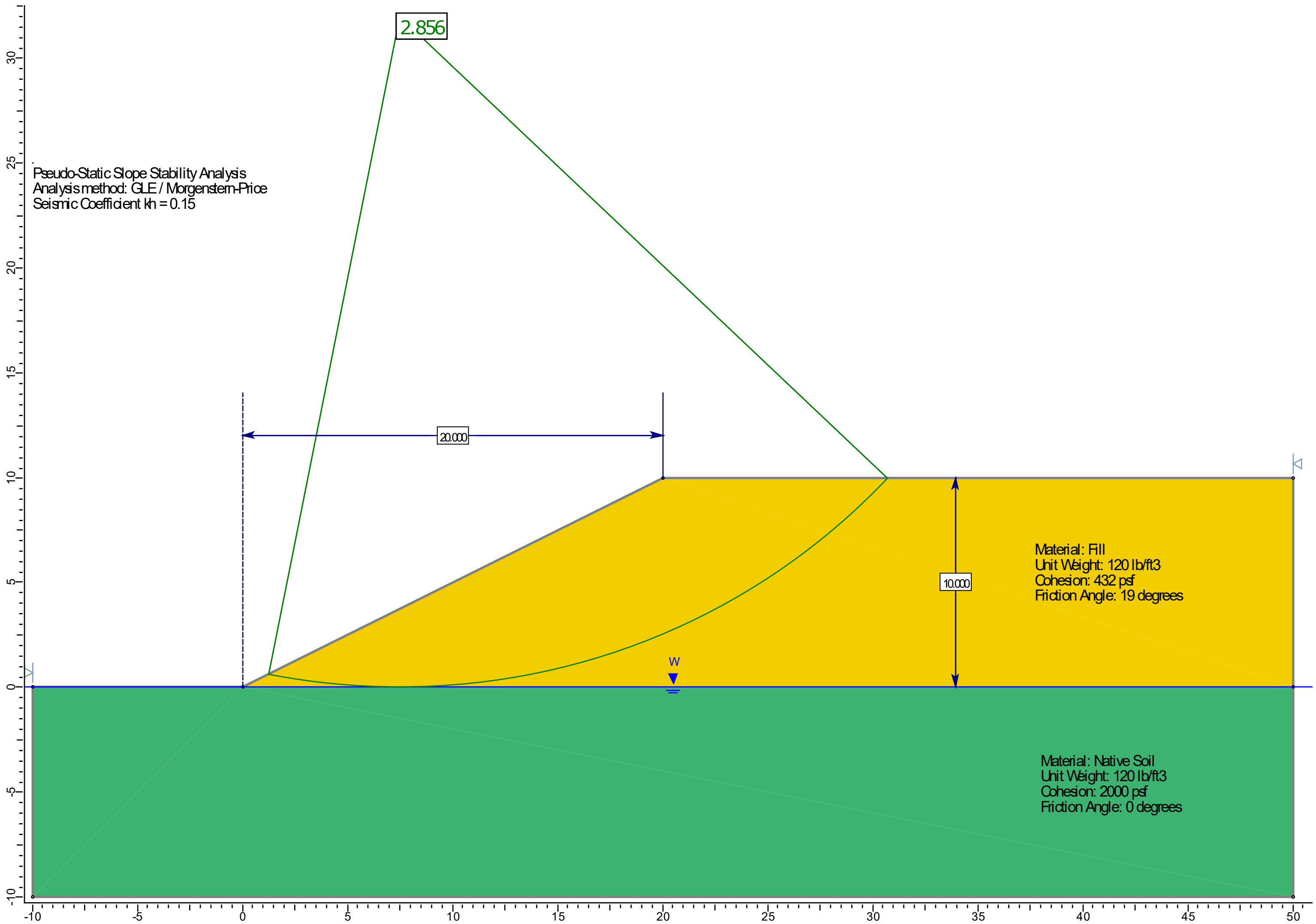

Figure C-1 Pseudo-Static Slope Stability Analysis