DESIGN-PHASE GEOTECHNICAL INVESTIGATION
WHITEWATER PRESERVE FLOOD CONTROL PROJECT
A PORTION OF ASSESSOR PARCEL NUMBERS (APNs) 514-240-004 AND 006
9160 WHITEWATER CANYON ROAD
WHITEWATER AREA OF RIVERSIDE COUNTY, CALIFORNIA

Q3 CONSULTING

July 27, 2020
J.N. 19-423

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Q3 CONSULTING
27042 Towne Centre Drive, Suite 110
Foothill Ranch, California 92610

Attention: Mr. John McCarthy

Subject: Design-Phase Geotechnical Investigation, *Whitewater Preserve Flood Control Project*, a Portion of Assessor Parcel Numbers (APNs) 514-240-004 and 006, 9160 Whitewater Canyon Road, Whitewater Area of Riverside County, California

Dear Mr. McCarthy:

In accordance with your request and authorization, **Petra Geosciences, Inc. (Petra)** is submitting this design-phase geotechnical investigation report for the proposed levee structure within the subject site located at 9160 Whitewater Canyon Road, in the Whitewater area of Riverside County, California. This work was performed in general accordance with the scope of work outlined in our Agreement for Project No. 40.039.000, dated December 4, 2019. The purposes of our assessment were to obtain information regarding relevant geotechnical and geologic condition of the site, information on the nature of the previous land usage, evaluate the potential geologic constraints that may affect development of the proposed levee, and to provide a design-phase geotechnical recommendations for the proposed site development.

This design-phase assessment also included a review of available published and unpublished geotechnical literature, available aerial photos and geologic maps pertaining to active and potentially active faults and other geologic hazards which may have an impact on the proposed development.

It has been a pleasure to be of service to you on this project. Should you have questions regarding the contents of this report, or should you require additional information, please contact this office.

Respectfully submitted,

PETRA GEOSCIENCES, INC.

Alan V. Pace, CEG
Senior Associate Geologist

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1.0 INTRODUCTION

Petra Geosciences, Inc. (Petra) is presenting herein the results of our design-phase geotechnical investigation for the proposed development situated at the boundary of two adjacent parcels, designated as APN's 514-240-004 and 514-240-006, at 9160 Whitewater Canyon Road, in the Whitewater Area of Riverside County, California. The purpose of this study was to obtain preliminary information on the general geologic and geotechnical conditions within the project area in order to provide conclusions and recommendations for the feasibility of the proposed project and design-phase geotechnical recommendations. This investigation included a review of published and unpublished literature, site reconnaissance and subsurface exploration, as well as a review of geotechnical maps pertaining to geologic hazards which may have an impact on the development of levee structure.

1.1 Scope of Work

The scope of our investigation consisted of the following.

- Collecting and reviewing available published and unpublished data, concerning geologic and soil conditions within the site and nearby area, that could have an impact on the proposed development.
- Review of readily available topographic maps and aerial photographs of the site and surrounding area.
- Coordination with the local underground utility locating service (i.e., Underground Service Alert USA) to obtain an underground-utility clearance, prior to commencement of the preliminary subsurface exploration.
- Excavation, logging and sampling five backhoe test pits to a maximum depth of 8 feet below the existing ground surface.
- Logging and field-classification of soil materials encountered in each test pits in accordance with the visual-manual procedures outlined in the Unified Soil Classification System and the American Society for Testing and Materials (ASTM) Procedure D 2488-90.
- Collecting representative bulk and undisturbed soil samples for laboratory analysis. Undisturbed samples were retrieved utilizing a 2.4-inch inside diameter, modified-California split-spoon sampler. In addition, some bulk samples were taken from the cuttings of excavation.
- Performing appropriate laboratory analysis on representative soil samples (bulk and undisturbed) obtained from the borings and soil-cement prepared from onsite soil to determine their engineering properties. Testing included determination of in-situ and maximum dry density; in-situ and optimum moisture content; sieve analysis; direct shear characteristics; unconfined compressive strength; soluble sulfate content; soil acidity (pH) and resistivity.

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- Preforming engineering and geologic analysis of the data with respect to the proposed site development.

- Preparation of this design-phase geotechnical report presenting the results of our evaluation of geotechnical conditions associated with site improvements including but not limited to information on site work, embankments, geology, soil erosion, channel lining, fill placement, over-excavation, groundwater, and levee construction and providing recommendations for the proposed site development in general conformance with the requirements of the 2019 California Building Code (2019 CBC) and U.S. Army Corps of Engineers (USACE) , as well as in accordance with applicable state and local jurisdictional requirements.

1.2 Location and Site Description

The subject flood control project includes the construction of a levee structure that is proposed to protect the Whitewater Preserve Visitor Center facilities against any future flood hazard. The visitor center is located at 9160 Whitewater Canyon Road in the Whitewater area of Riverside County and is comprised of a few maintenance buildings, native trails, parking lots, and appurtenant structures.

The subject site is located within the valley area of Whitewater Canyon. It is bounded by Whitewater Canyon Road and San Bernardino Mountains to the east, Whitewater River to the east and vacant lands to the north and south. The Whitewater River is a small permanent stream in central Riverside County, California, with some upstream tributaries in southwestern San Bernardino County. The river's headwaters are in the San Bernardino Mountains and it terminates at the Salton Sea in the Colorado Desert. The area drained by the Whitewater River is part of the larger endorheic Salton Sea drainage basin. The site exists at approximate elevation of 2180± feet above mean sea level. A map showing the location of the site is provided in Figure 1.

It is our understanding that the river banks within the site area may have been subject to several episodes of construction, with the materials that were derived from the river deposits, in the last century with the most recent likely being in the 1980’s. At the time of our field exploration, the area was covered occasionally with a medium growth of surface vegetation.

1.3 Proposed Development

Our review of the improvement plans prepared by Q3 Consulting (Q3) for Wildlands Conservancy dated June 9, 2020 indicates that the flood control system includes construction of a 2:1 (H:V) levee embankment behind the parking lot area and approximately 600 feet away from access road alignment. The proposed levee will be constructed on the east bank and along the Whitewater river and have a northwest-southeast trending. A portion of the study area, depicting the approximate alignment of the proposed levee and the Whitewater Preserve facilities is shown in Figure 2.

Based on the referenced plans, the levee will be an earthen embankment that will be covered by an 8-foot wide soil-cement layer. The total length of levee will be in the order of 1875 feet plus a 50-foot turn around area located at the southern terminus of structure. The levee crest, which will be used as an access road, has a width of 16 feet that increases to 50 feet at the turnaround pad. The maximum height of the levee is about 31 feet. For placement of the soil-cement layer, a trapezoidal-shaped channel with a 12-foot wide base will be excavated. This excavation will create a false slope on the opposite side of the levee that varies in height up to 34 feet. After the placement of soil-cement section, a portion of the exposed, soil-cement protective layer will be backfilled with compacted native soils. The finished soil-cement slope surface above the compacted fill will be protected with a rip-rap layer of native rocks.

Per United State Army Corps of Engineers performance guideline No. 1110-2-1913 (USACE), this levee is classified as a mainline levee that lie along a mainstream. A typical cross-section of the levee, its protective soil-cement layer, compacted fill and rip-rap is schematically shown in Figure 3.

1.4 Literature Review

Petra researched and reviewed available published and unpublished geologic data pertaining to regional geology, faulting and geologic hazards that may affect the site. A summary of the results of this review are included within the Findings section of this report.

1.5 Aerial Photo and Historical Map Analysis

USGS topographical maps dating from 1901, 1940, 1955, 1984, 1996, 2012 and 2018 were reviewed. Additionally, Goggle Earth® aerial photographs from 1995 to 2019 were also reviewed. As indicated in topographical plan, from 1901 to date, the Whitewater Canyon Road have had the same alignment and terminated at the current preserve center. The center, with a few interior streets and a trout pond, can be recognized in 1955 map. The topographical features of the area around the facility has not been changed significantly. However, minor changes in contour lines were observed that may be due to the erosion or dredging.

Generally, based on the review of Goggle Earth® aerial photographs from 1995 to 2019, the site appears to have the same features and land use to present day. With the passage of time, the Whitewater Trout Farm's historic building has been transformed into a visitor facility and ranger station. As a result of this

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transformation, the number of water ponds has been decreased. Aerial photos also indicate that the flows of water typically take through 2 or 3 small narrow streams. The alignment of these streams has been changed over the years. From 2015 to 2018, a major and a minor stream were running along the river thalweg. More recently, the major stream was found to be located closer to the western bank of river and the minor stream, which is a tributary of major stream, was found to be flowing close to eastern bank of the river in the vicinity of the proposed development. In 2019, the major stream has changed its alignment at the southern portion of proposed levee and joined the minor stream.

1.6 Field Exploration and Testing

A subsurface investigation was performed by Petra as part of this design-phase geotechnical investigation. Field exploration was performed on May 27, 2020 and included the excavation of 5 test pits (identified herein as TP-1 through TP-5) ranging in approximate depths of 6 to 8½ feet below the surface using a backhoe. The approximate locations of the test pits are shown on the attached Test Pit Location Map (Figure 2). The materials encountered within the exploratory test pits were classified and logged in accordance with the visual-manual procedures of the Unified Soil Classification System (USCS). Descriptive logs of the borings are presented in Appendix A.

Associated with the subsurface exploration was the collection of bulk samples and relatively undisturbed samples of soil materials for classification, laboratory testing and geotechnical engineering analyses. Bulk samples consisted of selected materials obtained at various depth intervals from the test pits. Relatively undisturbed samples were tried to obtain from the test pits using a 3-inch outside diameter (OD) modified California split-spoon soil sampler lined with brass rings in different location and depth, but only 1 sample was grabbed due to existence of numerous amount of cobble and boulders. The central portions of the driven core sample was placed in sealed containers and transported to Petra's laboratory for testing.

1.7 Laboratory Testing

To assist in a preliminary evaluation of the engineering properties of the on-site earth materials and their mixture with cement for the proposed development, laboratory testing was performed on selected representative bulk and relatively undisturbed samples of native soil materials obtained during the field evaluation and the mixture of soil-cement materials. Laboratory testing included determination of the following for pure soil and soil-cement mixture:

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- ✓ Native Soil
  - In-situ dry density and moisture content
  - Maximum dry density and optimum moisture content
  - Grain size analysis
  - Soluble sulfate, pH and resistivity
  - Direct shear
- ✓ Soil-Cement
  - Maximum dry density and optimum moisture content
  - Direct shear
  - Unconfined compressive strength

A description of laboratory test methods and laboratory testing are presented in Appendix B. The results of in-situ moisture content and dry density tests are summarized in the boring logs (Appendix A).

It should be noted that, due to the sandy nature of site soil and existence of oversize materials, obtaining relatively undistributed samples were not easily achievable. As such, certain assumptions and laboratory remolding of a few relatively undisturbed and disturbed samples had to be made to estimate the in-place characteristics of native soils.

2.0 FINDINGS

2.1 Regional Geologic Setting

The site is located in Whitewater River Area of the upper Coachella Valley at the juncture of three natural geomorphic provinces of California; the Transverse Ranges, the Peninsular Ranges, and the Colorado Desert. The Coachella Valley lies within the northern portion of the Salton Trough. This large northwest-trending structural depression extends approximately 180 miles from San Gorgonio Pass to the Gulf of California. Part of this basin, including the Salton Sea, lies below sea level and has progressively been filling with sediments eroded from local bounding mountain ranges, deposits from the Colorado River, and by incursions by the Gulf of California since at least the late-Miocene Epoch. Deposits within the Salton Trough are estimated to be over two to five miles thick (Kohler and Fuis, 1986; Fuis and Kohler, 1984; Biehler, et. al., 1964). It is considered the dominant feature of the Colorado Desert Geomorphic Province. It is well known for its exposures of the San Andreas Fault and related fault systems that form the margin between the Pacific and North American Plates.

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The western end of the San Gorgonio Pass is somewhat elusive in definition. Already several miles wide at Beaumont, it loses its identity as it merges with the Beaumont Upland. The Beaumont Upland, which extends almost to Redlands, is an alluvial plain, or terrace-like structure built up by streams carrying sand and gravel south from the eastern San Bernardino Mountains. This old erosion surface is a flat, smooth, gently sloping plain into which broad, steep-walled, flat-floored arroyos have been cut to a depth more than 50 feet below the surface level. Interstate 10 traverses the upland surface, dipping in several places with the gullies. Also visible from the freeway, recent stream rejuvenation has incised new gullies about 10 feet below this surface. The eastern end of the pass enters the Coachella Valley at Whitewater Canyon. It does so as a well-formed gradual slope and is about 1.5 miles wide measured between Windy Point and Whitewater Hill.

The San Bernardino Mountains are an elevated and faulted block, thrust upward from a region of low relief to their present height during Pleistocene time, about two million years ago. Inland from the ridges forming the valley edge, Joshua Tree National Monument occupies most of the interior section. Structurally, the flat upland plateau in the western section of the Little San Bernardino Mountains, including most of Joshua Tree National Monument, is a tilted block uplifted uniformly between the Mission Creek fault and the Morongo Valley fault. The northern margin of the block lies roughly parallel to Twentynine Palms Highway. The Little San Bernardino Mountains are considerably lower in elevation than either the Santa Rosa Mountains or the San Jacinto Mountains to the west. The most striking aspect of the mountains is the uniquely flat and uniform crestline. This is apparent from any viewpoint in Palm Springs or Palm Desert. This is the western margin of an ancient desert upland; an old erosion surface averaging 4,000 feet in elevation which is discussed in the following section. The eastern mountains are made up of the oldest rocks in the area, the Chuckwalla Complex of metamorphic rocks. This assemblage is of Precambrian age, about 1.7 billion years old.

Whitewater Canyon is a closed canyon with access only at its mouth. The east side of the canyon is an abrupt wall, with little vegetation. The western side is more sloping, with considerable vegetation. The closed northern end of the canyon is dominated by cliffs of bare, brown rock. The west side of the canyon displays the darker rocks of the ancient metamorphic Chuckwalla Complex. Rocks of the east wall are the younger Miocene Coachella Fanglomerate overlain by the early Pliocene Imperial Formation. A splendid exposure of the fanglomerate can be observed at the terminus of the road adjacent to the preserve center about 5 miles from the mouth of the canyon. The Whitewater River channel, containing abundant whitish boulders in its stream bed, generally lies close to the east side of the canyon. The Colorado Aqueduct crosses the canyon near its mouth, and here for part of the year, excess water is diverted from the aqueduct for

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recharge of the groundwater system. The stream crosses the valley to the spreading ponds of the Coachella Aquifer. About 1.5 miles into the canyon, the road crosses the Banning fault, considered by some to be the main strand of the San Andreas fault. The fault trace is marked by lush riparian vegetation in the stream channel, contrasting sharply with the stark canyon walls. Whitewater Canyon is the only remaining unspoiled canyon in the Coachella Valley.

2.2 Local Geology and Subsurface Soil Conditions

A regional geologic map of the subject property and vicinity maps the majority of the site as being underlain by young alluvial deposits exist in the main channel of the Whitewater River where the base of the levee will be founded. This layer is sitting on the top of older alluvial fan of San Gorgonio Pass. The stream channel alluvial materials are described as un-indurated and undissected gravelly cobbly sand with occasional boulders along stream valley.

Where encountered in our test pits, earth materials onsite consisted of artificial fill which are similar in character to the young alluvium. It is our understanding that this material occasionally derived from the stream deposits and stocked on the riverbank. As far as we know, this area was constructed and reconstructed several times in the last century with the most recent likely being in the 1980's. Since the fill material and stream channel alluvial are the same, fill/native soils contact and, therefore, fill thickness was not clearly detectable. In addition, excavation into the discussed materials was hard due to the existence of large size boulders (up to 4 feet). The fill and/or native materials encountered within the test pits consisted typically of dry to slightly moist, loose to medium-dense fine- to coarse-grained sands with gravel and numerous amounts of cobbles and boulders. Laboratory testing on a representative sample of the finer matrix materials yielded a dry density of 123.3 pounds per cubic foot, and a moisture content of 3.0 percent. Test Pit locations are presented on Figure 2. A regional surficial soils/geologic map of the site area is provided on Figure 4.

2.3 Groundwater

The site is located within the Indio (7-021.01) Groundwater Basin (California Department of Water Resources [CDWR], Water Data Library, 2020). The Regional Water Board Watershed basin is reported as the Whitewater-Coachella-Indio Basin. Groundwater depth varies within the area due to the rough topography and flow direction beneath the subject site is believed to be toward the south and lower area.

No seepage or static groundwater was encountered to the maximum depths explored by Petra (8½ feet below the ground surface). Based on our review of published literature and monitoring wells data which

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located within a radius of 10 miles (gathered from://wdl.water.ca.gov/waterdatalibrary), the depth to historically high groundwater in the area of the subject site is generally considered to be very deep and greater than 100 feet below the ground surface. Additionally, reviewing the Bulletin No. 108 prepared by Department of Water Resources for Coachella Valley area confirmed this scenario.

2.4 Tectonic Setting

2.4.1 Faulting

Based on our review of published and unpublished geotechnical maps and literature pertaining to site geology, no active or potentially active faults are known to project through the site and the site does not lie within the bounds of an "Earthquake Fault Zone" as defined by the State of California in the Alquist-Priolo (AP) Earthquake Fault Hazard Zoning Act (Whitewater-R95 Quadrangle from Bryant and Hart, 2007). State of California Seismic Hazard Zone maps created for this area also indicate no earthquake fault zones within or adjacent to the property (CGS, 1995). However, according to the Riverside County Parcel Report, the site is located within the county fault zone with high sensitivity. (Riverside County, 2014, 2019a).

As the geology map shown, the Whitewater Fault which was reported in Riverside County integrated project source by California Division of Mines and Geology (CDMG, 1980) and Dibblee (1981), lies within the east canyon wall, almost parallel to the canyon, and juxtaposes old alluvium against Coachella fanglomerate of late Miocene age. This fault was first mapped by Allen (1954). As he described, it is a relatively minor but continuous fault which separates crystalline rocks, Coachella fanglomerate, and quaternary gravels along the east wall of lower Whitewater Canyon. In 1957, Allen shows the fault concealed beneath Cabazon fanglomerate and it is also concealed by recent alluvium where it crosses the Whitewater River. Displacement on this steeply to moderately east-dipping fault is relatively up on the east. The Whitewater Fault is well-expressed by aligned drainages and saddles; however, this expression is principally fault line geomorphology and there is no expression of fault within the Holocene alluvium. According to State of California fault definitions, an "active" fault has had displacement within the Holocene epoch (i.e., the last 11,000 years). Based on the fault evaluation report No. FER-235 (CDMG, 1994), since there is no indication of Holocene activity along the Whitewater Fault, this fault in inactive.

However, it should be noted that according to the USGS Unified Hazard Tool website and/or 2010 CGS Fault Activity Map of California, the nearest active fault (design fault for the site) is the South Branch of the San Andreas Fault zone (San Bernardino Mountains section), which is located approximately 3.48 ± miles on both north and south side of the site. The subject site is located at a distance of less than {6.25 miles (10 km)} from the surface projection this fault system, which is capable of producing a magnitude 7

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or larger events with a slip rate along the fault greater than 0.04 inch per year. As such, the site should be considered as a **Near-Fault Site** in accordance with ASCE 7-16, Section 11.4.1.

In spite of the active tectonic regime, earthquakes in the Whitewater Canyon region within historical times (i.e., the past couple hundred years) have been infrequent and of small magnitude. A listing of historical earthquakes published by the National Earthquake Information Center (2006) indicates that the largest earthquake occurring within a radius of approximately 62 miles (100 kilometers) of the site was the Magnitude 7.3 Landers earthquake in 1992. This event, along with the associated aftershocks, occurred approximately 31 miles northeast of the subject property. The closest documented earthquake equal to or greater than magnitude 6.0, was a magnitude 6.0 Morongo Valley earthquake that occurred approximately 3.1 miles northeast of the site in 1986.

Some of the more significant historic seismic events in the recent 100 years with magnitude of 6 or greater and within 100 kilometers of subject site are listed in Table 1, along with the corresponding approximate epicentral distances to the subject site and the calculated moment magnitude based on various published earthquake databases.

TABLE 1

Notable Historical Earthquakes (M $\geq$ 6) within 100 kilometers of Project

Date	Location	Approximate Distance from Site (km)	Magnitude
1999	16km SW of Ludlow, CA	77	7.1
1992	7km SSE of Big Bear City, CA	29	6.3
1992	Landers, California Earthquake	31	7.3
1992	17km NNE of Thousand Palms, California	32	6.1
1986	6km SSW of Morongo Valley, CA	5	6.0
1954	12km W of Salton City, CA	94	6.4
1948	16km E of Desert Hot Springs, CA	30	6.0
1937	16km WSW of Oasis, CA	76	6.0
1918	Southern California	41	6.8

Based on our review of aerial photographs for the site and vicinity, photo lineaments were not observed traversing the site. While fault rupture would most likely occur along previously established fault traces, fault rupture could occur at other locations. However, as discussed above, the potential for active fault rupture at the site is considered to be very low.

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2.4.2 Nearby Seismic Sources

Published geologic maps and literature indicate that the site lies within 50 miles of a number of significant active and potentially active faults (including the various segments of the San Andreas Fault zone) that are considered capable of generating strong ground motion at the subject site. The names and locations of these faults relative to the subject property are provided in Table 2.

TABLE 2

Significant Nearby Seismic Sources(1)

Fault Name	Approximate Distance/ Direction from Site	Slip Rate (mm/yr)[1]	Maximum Magnitude[1]
South San Andreas (San Gorgonio Pass-Garnet Hill Segment)	2.97 miles south	>5.0	7.74
North San Andreas (Mill Creek Segment)	3.34 miles north	>5.0	7.91
Mission Creek	3.35 miles north	>5.0	7.26
Pinto Mountain	5.67 miles north	2.5	7.3
Burnt Mountain	12.84 miles east	0.6	6.8
Eureka Peak	17.42 miles east	0.6	6.7
Landers	18.32 miles northeast	0.6	7.4
San Jacinto (Coyote Creek)	20.75 miles southwest	14.0	7.1
San Jacinto (Anza)	22.59 miles southwest	9.0	7.3
North Frontal (East)	23.26 miles northeast	0.5	7.0
Lenwood-Lockhart-Old Woman Springs	24.93 miles southeast	0.9	7.5
Johnson Valley	25.11 miles north	0.6	6.9
So. Emerson-Copper Mountain	28.9 miles east	0.6	7.1
Calico-Hidalgo	31.49 miles east	1.8	7.4
San Jacinto (San Jacinto Valley)	35.34 miles southwest	18.0	7.0
Pisgah-Bullion Mountain	36.94 miles northeast	0.8	7.3
Cleghorn	37.40 miles northwest	3.0	6.8
Elsinore (Julian Section)	43.26 miles southwest	3.0	7.4
Cucamonga	46.94 miles west	5.0	6.7
Elsinore (Glen Ivy Section)	48.33 miles southwest	3.0	7.49

Note: [1] Per USGS 2008 National Seismic Hazard Maps – Source Parameters
http://geohazards.usgs.gov/cfusion/hazfaults_2008_search

As indicated above, the San Gorgonio Pass-Garnet Hill and mill creek segments of the San Andreas Fault zone are located on south and north of the subject site, respectively. This fault is among the most active in California and has accordingly been placed within a State of California Alquist-Priolo Earthquake Fault Zone (Bryant and Hart, 2007; CGS 2015). According to State of California fault definitions, an "active" fault has had displacement within the Holocene epoch (i.e., the last 11,000 years). A "potentially active" fault is a fault that does not have evidence of movement within the last 11,000 years, but has moved within Quaternary period, the last 2.6 million years. "Potentially active" faults are not placed within Alquist-Priolo Earthquake Fault Zones, but are considered when conducting siting studies for such critical structures as dams and nuclear power plants, etc.

It should be noted that, based on our research and evaluation, any number of faults within the Salton Sea region and the Colorado Desert Geomorphic Province could generate severe site ground motions. The major contributor to the deterministic minimum component of the ground motion models, however, is San Bernardino segment of San Andreas Fault. Riverside County, however, has identified the San Jacinto Valley segment of the San Jacinto Fault zone with a higher probability (43 percent vs. 22 percent) of an earthquake occurring on a fault segment in the next 30 years than the Coachella segment of the San Andreas Fault zone (Riverside County, 2014).

2.5 Secondary Seismic Effects

Secondary effects of seismic activity normally considered as possible hazards to a site include several types of ground failure. Various general types of ground failures, which might occur as a consequence of severe ground shaking at the site, include ground subsidence, ground lurching and lateral spreading. The probability of occurrence of each type of ground failure depends on the severity of the earthquake, distance from faults, topography, subsoil and groundwater conditions, in addition to other factors.

Based on the site conditions and location with respect to the Whitewater Canyon walls, landsliding, significant ground lurching, and lateral spreading are considered unlikely at the site. The types of seismically induced flooding that are generally considered as potential hazards to a particular site normally include flooding due to a tsunami (seismic sea wave), a seiche, or failure of a major reservoir or other water retention structure upstream of the site. Since the site lies at approximately 2180 feet above the Pacific Ocean and does not lie in close proximity to an enclosed body of water or downstream of a major reservoir retention structure, the probability of flooding from a tsunami, seiche, or dam-break inundation is considered non-existent. Additionally, the site is not located within a Tsunami Inundation Area on the Tsunami Inundation Map for Emergency Planning, produced by the State of California (2009).

The potential for ground subsidence due to seismic shaking is anticipated to be Moderate. The seismic-related subsidence (dynamic settlement) is discussed later in this report.

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2.6 Seismic Design Parameters

Earthquake loads on earthen structures and buildings are a function of ground acceleration which may be determined from the site-specific ground motion analysis. Alternatively, a design response spectrum can be developed for certain sites based on the code guidelines. To provide the design team with the parameters necessary to construct the design acceleration response spectrum for this project, we used two computer applications. Specifically, the first computer application, which was jointly developed by Structural Engineering Association of California (SEAOC) and California's Office of Statewide Health Planning and Development (OSHPD), the SEA/OSHPD Seismic Design Maps Tool website, https://seismicmaps.org, is used to calculate the ground motion parameters. The second computer application, the United Stated Geological Survey (USGS) Unified Hazard Tool website, https://earthquake.usgs.gov/hazards/interactive/, is used to estimate the earthquake magnitude and the distance to surface projection of the fault. It should be noted that the contents of this section, in general, applies to any appurtenant structures, if any, that may be constructed in conjunction with the proposed levee development.

To run the above computer applications, site latitude and longitude, seismic risk category and knowledge of site class are required. The site class definition depends on the direct measurement and the ASCE 7-16 recommended procedure for calculating average small-strain shear wave velocity, Vs30, within the upper 30 meters (approximately 100 feet) of site soils.

Tentatively, a seismic risk category of II (subject to verification) was assigned to the proposed structure, if any, in accordance with 2019 CBC, Table 1604.5. No shear wave velocity measurement was performed at the site, however, the subsurface materials at the site appears to exhibit the characteristics of stiff soils condition for Site Class D designation. Therefore, an average shear wave velocity of 600 feet per second for the upper 100 feet was assigned to the site based on engineering judgment and geophysical experience. As such, in accordance with ASCE 7-16, Table 20.3-1, Site Class D (D- Default as per SEA/OSHPD software) has been assigned to the subject site.

The following table, Table 3, provides parameters required to construct the seismic response coefficient, C_s, curve based on ASCE 7-16, Article 12.8 guidelines, for design of buildings. A printout of the computer output is attached in Appendix C.

TABLE 3

Seismic Design Parameters

Ground Motion Parameters	Specific Reference	Parameter Value	Unit
Site Latitude (North)	-	33.9883	°
Site Longitude (West)	-	-116.6570	°
Site Class Definition	Section 1613.2.2 [1], Chapter 20 [2]	D-Default [4]	-
Assumed Risk Category	Table 1604.5 [1]	II	-
M_w - Earthquake Magnitude	USGS Unified Hazard Tool [3]	7.68 [3]	-
R – Distance to Surface Projection of Fault	USGS Unified Hazard Tool [3]	4.78 [3]	km
S_s - Mapped Spectral Response Acceleration Short Period (0.2 second)	Figure 1613.2.1(1) [1]	2.403 [4]	g
S_1 - Mapped Spectral Response Acceleration Long Period (1.0 second)	Figure 1613.2.1(2) [1]	0.901 [4]	g
F_a – Short Period (0.2 second) Site Coefficient	Table 1613.2.3(1) [1]	1.2 [4]	-
F_v – Long Period (1.0 second) Site Coefficient	Table 1613.2.3(2) [1]	Null [4]	-
S_{MS} – MCE_R Spectral Response Acceleration Parameter Adjusted for Site Class Effect (0.2 second)	Equation 16-36 [1]	2.884 [4]	g
S_{M1} - MCE_R Spectral Response Acceleration Parameter Adjusted for Site Class Effect (1.0 second)	Equation 16-37 [1]	Null [4]	g
S_{DS} - Design Spectral Response Acceleration at 0.2-s	Equation 16-38 [1]	1.923 [4]	g
S_{D1} - Design Spectral Response Acceleration at 1-s	Equation 16-39 [1]	Null [4]	g
$T_o = 0.2\ S_{D1}/S_{DS}$	Section 11.4.6 [2]	Null	s
$T_s = S_{D1}/S_{DS}$	Section 11.4.6 [2]	Null	s
T_L - Long Period Transition Period	Figure 22-14 [2]	8 [4]	s
PGA - Peak Ground Acceleration at MCE_G [*]	Figure 22-9 [2]	0.980	g
F_{PGA} - Site Coefficient Adjusted for Site Class Effect [2]	Table 11.8-1 [2]	1.2 [4]	-
PGA_M –Peak Ground Acceleration [2] Adjusted for Site Class Effect	Equation 11.8-1 [2]	1.176 [4]	g
Design PGA ≈ (⅔ PGA_M) - Slope Stability [†]	Similar to Eqs. 16-38 & 16-39 [2]	0.784	g
Design PGA ≈ (0.4 S_{DS}) – Short Retaining Walls [‡]	Equation 11.4-5 [2]	0.769	g
C_{RS} - Short Period Risk Coefficient	Figure 22-18A [2]	0.899 [4]	-
C_{R1} - Long Period Risk Coefficient	Figure 22-19A [2]	0.883[4]	-
SDC - Seismic Design Category [§]	Section 1613.2.5 [1]	Null [4]	-

References:
[1] California Building Code (CBC), 2019, California Code of Regulations, Title 24, Part 2, Volume I and II.
[2] American Society of Civil Engineers/Structural Engineering Institute (ASCE/SEI), 2016, Minimum Design Loads and Associated Criteria for Buildings and Other Structures, Standards 7-16.
[3] USGS Unified Hazard Tool - https://earthquake.usgs.gov/hazards/interactive/
[4] SEI/OSHPD Seismic Design Map Application – https://seismicmaps.org

Related References:
Federal Emergency Management Agency (FEMA), 2015, NEHERP (National Earthquake Hazards Reduction Program) Recommended Seismic Provision for New Building and Other Structures (FEMA P-1050).

Notes:
* PGA Calculated at the MCE return period of 2475 years (2 percent chance of exceedance in 50 years).
† PGA Calculated at the Design Level of ⅔ of MCE; approximately equivalent to a return period of 475 years (10 percent chance of exceedance in 50 years).
‡ PGA Calculated for short, stubby retaining walls with an infinitesimal (zero) fundamental period.
§ The designation provided herein may be superseded by the structural engineer in accordance with Section 1613.2.5.1, if applicable.

2.6.1 Discussion

Owing to the characteristics of the subsurface soils, as defined by Site Class D-Default designation, and proximity of the site to the sources of major ground shaking, the site is expected to experience strong ground shaking during its anticipated life span. Under these circumstances, where the code-specified design response spectrum may not adequately characterize site response, the 2019 CBC typically requires a site-specific seismic response analysis to be performed. This requirement is signified/identified by the “null” values that are output using SEA/OSHPD software in determination of short period, but mostly, in determination of long period seismic parameters (see Table 3).

For conditions where a “null” value is reported for the site, a variety of structural design approaches are permitted by 2019 CBC and ASCE 7-16 in lieu of a site-specific seismic hazard analysis. For any specific site, these alternative design approaches, which include Equivalent Lateral Force (ELF) procedure, Modal Response Spectrum Analysis (MRSA) procedure, Linear Response History Analysis (LRHA) procedure and Simplified Design procedure, among other methods, are expected to provide results that may or may not be more economical than those that are obtained if a site-specific seismic hazards analysis is performed. These design approaches and their limitations should be evaluated by the project structural engineer.

2.6.1.1 Seismic Design Category

Please note that the Seismic Design Category, SDC, is also designated as “null” in Table 3. For Risk Category I, II or III structures, where the mapped spectral response acceleration parameter at 1 – second period, S1, is greater than or equal to 0.75, the 2019 CBC, Section 1613.2.5.1 requires that these structures be assigned to Seismic Design Category E.

2.6.1.2 Equivalent Lateral Force Method

Should the Equivalent Lateral Force (ELF) method be used for seismic design of structural elements, the value of Constant Velocity Domain Transition Period, T_s, is estimated to be 0.531 seconds and the value of Long Period Transition Period, T_L, is provided in Table 3 for construction of Seismic Response Coefficient – Period (C_s -T) curve that is used in the ELF procedure.

As stated herein, the subject site is within a Site Class D-Default. A site-specific ground motion hazard analysis is not required for structures on Site Class D-Default with $S_1 \geq 0.2$ provided that the Seismic Response Coefficient, C_s, is determined in accordance with ASCE 7-16, Article 12.8 and structural design is performed in accordance with Equivalent Lateral Force (ELF) procedure.

2.7 Seismically Induced Liquefaction and Dry Sand Settlement

2.7.1 General

Liquefaction occurs when strong seismic shaking of a saturated sand or silt causes intergranular fluid (pore-water) pressures to increase to levels where grain-to-grain contact is lost, and material temporarily behaves as a viscous fluid. Liquefaction can cause settlement of the ground surface, loss of bearing, settlement and tilting of structures, flotation and buoyancy of buried structures and fissuring of the ground surface. A common surface manifestation of liquefaction is the formation of sand boils – short-lived fountains of soil and water that emerge from fissures or vents and leave freshly deposited, usually conical mounds of sand or silt on the ground surface.

For sandy soils above the water table, strong seismic shaking can also result in rearrangement of the granular soil structure leading to densification of sandy soils, ground settlement and settlement and tilting of superstructures.

Assessment of liquefaction or dry sand settlement potential for a particular site requires knowledge of a number of regional as well as site-specific parameters, including the estimated design earthquake magnitude, and the associated probable peak horizontal ground acceleration at the site, subsurface stratigraphy and soil characteristics. Parameters such as estimated probable peak horizontal ground acceleration can readily be determined using published references, or by utilizing a commercially available computer program specifically designed to perform a probabilistic analysis. On the other hand, stratigraphy and soil characteristics can only be accurately determined by means of a site-specific subsurface investigation combined with appropriate laboratory analysis of representative samples of onsite soils.

2.7.2 Governmental Approach

In April 1991, the State of California enacted the Seismic Hazards Mapping Act (Public Resources Code, Division 2, Chapters 7-8). This act requires an assessment of liquefaction among other seismic hazards prior to new construction for most projects where geological conditions warrant. The purpose of the Act is to protect the public safety from the effects of strong ground shaking, liquefaction, landslides, or other ground failure.

Where liquefaction potential is established, it is required to be mitigated to acceptable levels of risk. The Act defines mitigation as "… those measures that are consistent with established practice and reduce seismic risk to acceptable levels." Acceptable level of risk is defined as "that level that provides reasonable protection of the public safety, though it does not necessarily ensure continued structural integrity and

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functionality of the project [California Code of Regulations; Section 3721 (a)]." It is, therefore, interpreted that in the context of the Act, mitigation of the potential liquefaction hazards at the site, to appropriate levels of risk, can be accomplished through appropriate development design.

More specifically, the 2019 California Building Code in Section 1803.5.11 and 1803.5.12, for structures within Seismic Design Categories C through F, requires the specific assessment of liquefaction hazards at a site. It also requires provision of recommendations for mitigation if a hazard exists.

2.8 Site-Specific Liquefaction and Dry Sand Settlement Hazard Analysis

Review of the County of Riverside Environmental Impact Report No. 521 (public review draft) indicates that the property is located within an area that has been designated as having a Moderate potential for earthquake-induced liquefaction (Riverside County, 2014). However, based upon a relatively deep historic high groundwater level (100+ feet), the liquefaction potential at the site is considered negligible. As such, surface manifestation of liquefaction such as ground fissures, sand boils, loss of bearing, liquefaction-induced settlement, etc. is considered negligible. Additionally, according to the intended purpose of the subject levee, except during a flooding event, the subject levee is not expected to be exposed to the significant body of water. Therefore, the liquefaction potential for levee embankment was not considered in analyzing the stability of levee because of the low probability of earthquake coinciding with periods of flood.

Due to the absence of high groundwater level and based on the sandy nature of the site soil encountered in the Petra's exploration pits, the most likely scenario for dynamic settlements is considered the dry sand settlement. This is due primarily to the presence of unconsolidated granular sandy soils and to the proximity of seismic sources. The undisturbed onsite soils, as tested in our laboratory, indicate to exist at approximately 85 percent of laboratory maximum dry density. The existing native materials will form the majority of levee embankment and its foundation. Based on our experience in this area, the total seismic settlement is anticipated to be on the order of 2 - 3 inches. Furthermore, the differential dynamic settlement is estimated to be on the order of 1 – 1 ½ inches over a span of 30 feet. It should be noted that our estimated settlement is for free field condition. Depending on proposed levee geometry, height and stiffness, the actual settlement during the design earthquake may vary from those estimated herein due to the proposed geometry and various levee components interactions.

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2.9 Geotechnical Issues Not Related to Seismicity

2.9.1 Subsidence

Subsidence is the settlement or deformation of the land surface caused by several different conditions (including tectonic activity and petroleum production); however, it is most commonly associated with changes in groundwater levels. Long-term withdrawal of groundwater in the area of the subject site has lowered the water table considerably, and this has resulted in subsidence in some areas of the Coachella Valley (Sneed, Brandt and Solt, 2014). Although partial recovery of the settlement may be possible if the water table is recharged.

According to Section 4.12 of the County of Riverside Environmental Impact Report No. 521 (public review draft dated March 2014), the subject site lies within an area that is susceptible to subsidence. According to Chapter 6.0 of the County of Riverside General Plan (County of Riverside, 2008), Policy S-3.8, requires that a geotechnical evaluation of subsidence be performed if a site lies within a documented subsidence area, or an area that is susceptible to subsidence as shown on Figure S-7 of that document. As stated in the plan *"differential displacement and fissures occur at or near the valley margin, and along faults. In the County of Riverside, the worst damage to structures, as a result of regional subsidence, may be expected at the valley margins"*.

Based on the topographical features of development area, any activity on the upstream portion of this segment of Whitewater River will lead to changes in the groundwater level. However, due to the relatively deep underground water level, we do not anticipate additional settlement at the site any subsidence to be significant.

2.9.2 Wind Erosion

Figure 4.12.6 of Section 4.12 of the Riverside County Draft Environmental Impact Report (EIR) indicates that the site is located in an area that is categorized as "High" for wind erodibility (County of Riverside, 2014). Development plans should account for the potential effects of wind-blown sand.

2.9.3 Landslides and Rock Falls

The site area is surrounded by the steep slopes which can increase the possibility of seismically induced landslides or rockfalls. However, the proposed development is located in the valley portion of Whitewater Canyon that is far enough, on the order of 200+ feet, from the both steep slopes of canyon sidewalls and,

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therefore, is considered less susceptible to these types of hazards. Based on these findings, it is our opinion that the probability of the site being affected by landslides or rock falls is considered very low.

2.9.4 Expansive Soils

Our visual and tactile classification of onsite soil materials indicates that expansive soils are not likely to be present at the site near the surface. We did not find any clayey soils in the test pits that were excavated for this project to be indicative of the presence of expansive soils. Foundations and exterior flatwork, if any, should be designed based on the appropriate soil's expansive characteristics.

2.9.5 Flooding Not Related to Seismicity

As part of this investigation, we conducted an independent review of the applicable FEMA flood insurance rate map for the area of the subject site (Map No. 06065C0860G, effective August 28, 2008). This map indicates that the site of the proposed construction is located within an area that is designated as Flood Zone X, which signifies one or more of the following conditions could be present (FEMA, 2009):

- The site is located within an area having a 0.2 percent annual chance flood;
- The site is located within an area having a 1 percent annual chance flood with an average flood depth of less than 1 foot or with a drainage area of less than 1 square mile;
- The site is located within an area protected from the 1 percent annual chance flood by a levee system.

It should be noted that the County of Riverside Environmental Impact Report No. 521 ((Riverside County, 2014) reports that the site is not located within the 100-year food zone, Riverside County parcel report indicates that a flood plain review maybe required. With respect to the purpose of this project, which is to protect the preserve center against the 500-year flood, we are under the assumption that flood plain study was already conducted for this site and its conclusions will be considered in the proposed development.

3.0 CONCLUSIONS AND RECOMMENDATIONS

3.1 General Feasibility

From a soils engineering and engineering geologic point of view, the subject property is considered suitable for the proposed construction provided the following conclusions and recommendations are incorporated into the design criteria and project specifications. In addition, the proposed grading and construction are not expected to affect the stability of adjoining properties in an adverse manner provided grading and construction are performed in accordance with current standards of practice, all applicable grading ordinances and the recommendations presented in this report.

3.2 Grading Plan Review

This report has been prepared with reference to an improvement plan prepared by Q3 and plot dated June 9, 2020. As stated above, the development on the subject site includes the grading of existing river embankment to accommodate a flood control levee that is planned to protect the Whitewater Preserve Visitor Center facilities. As stated earlier, the levee is an earthen embankment which will be stabilized by a soil-cement layer extending from the embankment crown to the toe of water face slope. The total length of levee will be in the order of 1875 feet plus a 50-foot turnaround area located at the southern terminus of structure. The levee will be constructed on the existing river deposits, whether native soils or potentially previously placed fills, which will stay in place and will be covered with a layer of compacted fill that, in turn, will be protected by an 8-foot thick soil-cement layer. The crest that will be used as an access road to the turnaround pad.

For placement of the soil-cement layer, a trapezoidal-shaped channel, with a 12-foot wide base, will be excavated. This excavation will create a false slope on the opposite side of the levee that varies in height up to 34 feet. In some segments along the levee structure, where the existing ground elevation at the levee crest is lower than finish subgrade level, after processing the existing native soil the grade will bring up to the desired level using the compacted fill material. This operation also will provide a suitable subgrade layer for the access road. Further, this grading operation will create a low-height, 2:1 (H:V) slope with compacted fill within the landward area. Eventually, this fill slope will be covered with relatively loose native material (lightly compacted under the construction equipment wheels) with gradient flatter than 2:1.

3.3 Geotechnical Concerns

The geotechnical concerns related to the proposed development of levee embankment are considered as follows:

- Static Settlement
- Seismically Induced Settlement
- Permeability and Seepage
- Slope Stability

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3.3.1 Static Settlement

In general, evaluation of the post-construction settlement that can occur from consolidation and/or compression of both embankment and foundation materials is of importance if the settlement would result in loss of freeboard of the levee or damage to structures embedded within the embankment. As such, it is a common practice to overbuild the levee by a given percent of its height to take into account the impact of the anticipated settlement. Common allowances are 0 to 5 percent for compacted fill, 5 to 10 percent for semi-compacted fill, 15 percent for uncompacted fill, and 5 to 10 percent for hydraulic fill.

Based on the levee configuration presented on the improvement plan, construction details, intended use and the result of our laboratory test result, we anticipate that the static settlement of this embankment under the embankment loading to be less than 5 percent of total height of levee (i.e., 1 to 2 inches). Owing to the cohesionless nature of the site soils, a major portion of this anticipated settlement is expected to occur during construction.

3.3.2 Seismically Induced Settlement

The site is located within an active tectonic area of southern California with several significant faults capable of producing moderate to strong earthquakes. The South San Andreas Fault zone (consisting of the Banning and San Bernardino segments) is in close proximity of the site and capable of producing strong ground motions. The site will likely be subjected to many strong, seismically related ground shaking during the anticipated life span of the project and structures within the site should, therefore, be designed and constructed to resist the effects of strong ground motion.

Based on a review of the Riverside County General Plan (Safety Element) and Draft EIR, the site lies within zone that is moderate susceptible to liquefaction (Riverside County, 2014, 2019a). Typically, liquefaction occurs in areas where groundwater lies within the upper 50± feet of the ground surface. However, a relatively deep historic high groundwater level (in excess of 100 feet below ground surface), the liquefaction potential at the site is considered negligible. Considering the absence of shallow groundwater and based on the relatively dry state of the sandy site soils encountered during site exploration, the most likely scenario for dynamic settlements is the dry sand settlement. This is due primarily to the presence of unconsolidated granular sandy soils and to the proximity of seismic sources.

Based on our experience in this area, the total seismic settlement is anticipated to be on the order of 2 – 3 inches. Furthermore, the differential dynamic settlement is estimated to be on the order of 1 - 1½ inches over a span of 30 feet.

3.3.3 Permeability and Seepage

Levee seepage is when water moves away from the river channel, either below (foundation) or through the levee and surrounding land surface. Typically, the following conditions may develop:

- Under Seepage - In pervious foundations beneath levees, under-seepage may result in excessive hydrostatic pressures beneath an impervious top stratum on the landside and may cause sand boils and piping beneath the levee itself. In general, these problems are most acute where a pervious substratum underlies a levee and extends both landward and riverward of the levee and where a relatively thin top stratum exists on the landside of the levee. Principal seepage control measures for foundation under-seepage are cutoff trenches, riverside impervious blankets, landside seepage berms, pervious toe trenches, and pressure relief wells.

- Through Seepage – For conditions where through-seepage in an embankment emerge on the landside slope, it can soften fine grained fill in the vicinity of the landside toe, cause sloughing of the slope, or even lead to piping (internal erosion) of fine sand or silt materials. Seepage exiting on the landside slope would also result in high seepage forces, decreasing the stability of the slope. The efficient means of through-seepage control includes pervious toe drains and horizontal or inclined drainage layers.

As discussed previously, the proposed levee will be designed to protect the landside during the flood events and, because of the deep groundwater surface, the levee structure is not expected to experience the steady state seepage condition . Additionally, the proposed elevation of landside area with respect to the finished level of levee crest is expected to significantly reduce the possibility of occurrence for any phenomena described above during flood. With all this in mind, the impact of any type of seepage on the proposed levee structure is considered negligible.

3.3.4 Slope Stability

Stability of the proposed slope depends on geometry of the levee and its various components, shear strength of the components, various construction sequences, and external loading conditions. The effect of each of these parameters are discussed below.

3.3.4.1 Geometry

Based on the development plan and the sequences of construction, two different cross-sections, considered most critical representatives of the levee alignment, are considered in our slope stability analyses. These cross-sections are 1) the levee embankment at Station 26+00 that presents the highest cross-section with a total height of 34 feet and, 2) the false slope created during construction at Station 18+00 with a height of 31 feet.

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<u>3.3.4.2 Shear Strength Parameters</u>

Direct shear tests were performed on reconstituted samples of native site soils in our laboratory. The samples reconstituted to represent the anticipated native soils, compacted fill and compacted soil-cement conditions. Test results i.e. shear strength parameters are presented in Appendix A and a summary of design shear strength parameter are presented in Table 4 below.

TABLE 4

Shear Strength Parameters

Material	Peak Value		Ultimate Value	
	Cohesion (C, psf)	Angle of Internal Friction (Ø, degrees)	Cohesion (C, psf)	Angle of Internal Friction (Ø, degrees)
Native Soil	90	31	72	28
Compacted Fill	10	36	10	36
Compacted Soil-Cement (with 5% cement)	426	44	5	34

<u>3.3.4.3 Acceptable Slope Stability Criteria and Slope Stability Analysis</u>

The USACE has established minimum factor of safety (FS) thresholds for levee slope stability. The FS against slope failure is estimated by calculating the forces resisting slope failure divided by the forces causing slope failure. Thus, a FS of greater than 1 implies a stable slope, a FS less than 1 implies a failing state, and a FS equal to 1 implies that a slope is on the verge of failure. Allowable minimum FS values, presented in Table 5 below, have been used to evaluate various stability conditions for levee slopes.

TABLE 5

Minimum Acceptable Factor of Safety Values

Material	Minimum Factor of Safety
Static - Temporary	1.25
Static – Long Term	1.40
Pseudo - Static	1.00
Rapid Drawdown	1.00

The principal methods used to analyze levee embankments for static stability against shear failure assume either a sliding surface having the shape of a circular arc within the embankment or a planar failure surface along a relatively weak zone through the soil mass. Based on the levee geometry and relatively homogeneous nature of the site soils, a circular failure surface is considered most suitable.

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Pseudo-static analysis is one of the simplest approaches used to analyze the seismic response of soil embankments and slopes. Selection of an appropriate seismic coefficient is the most important, and yet a difficult aspect of a pseudo-static stability analysis. In theory, the seismic coefficient values should depend on some measure of the amplitude of the inertial force induced in the slope by the dynamic forces generated during an earthquake. Because soil slopes are not rigid and the peak acceleration generated during an earthquake last for only a very short period of time, seismic coefficients used in practice generally correspond to acceleration values well below the predicted peak ground accelerations (Kramer, 1996). However, the choice of coefficients used in the slope stability analysis is very subjective and lacks a clear rationale. Due to the existence of significant nearby seismic sources (San Andreas Fault) and strong ground motions generated during the 1992 Landers Earthquake, the horizontal seismic coefficient value of 0.2 is considered in our pseudo-static analysis.

Rapid drawdown condition represents the condition whereby a prolonged flood stage saturates at least the major part of the levee embankment and then the flood water falls faster than the soil can drain. This causes the development of excessive driving forces, i.e. saturated zones and the presence of excess pore water pressure that, in turn, reduces the resisting effective stress in the soil mass, the combination of which may result in the slope becoming unstable.

In light of the above discussion, four slope stability conditions are considered as follows:

- During Construction
- Immediately after Construction
- During Operation
- Post Scour (Post Full Flood Stage)

The results of our analysis are summarized and presented in Appendix D and will be discussed below.

<u>3.3.4.4 During Construction</u>

As stated earlier, it is our understanding that the levee excavation creates two slopes, the main levee and the false slope. As presented in Appendix D, in dry condition, both slopes are anticipated to stay stable during construction.

Since the Whitewater River is a year-around stream, a potential for the presence of shallow groundwater or perched groundwater during excavation was also evaluated. Our stability analyses for determination of a

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critical elevation above which the presence of groundwater is detrimental to stability of excavated slopes indicate that:

1. The slopes will be stable, if the water seepage is observed in the lower 3 and 9 feet of excavated valley on the false and levee slope sides, respectively. (FS≥1.25).

2. The slopes will be on the verge of failure, if seepage exist shallower than 10 or 17 feet above the valley bottom on the on the false and levee slope sides, respectively. (FS≤1.00).

3. For conditions where seepage is observed between the depth determined above (a and b), it means that the slopes are partially stable. (1.00<FS<1.25).

We understand that the flow of the existing streams will be diverted to a new alignment away from the proposed construction areas prior to beginning the construction to keep the water away from the excavation area as much as possible. Notwithstanding and aside from the construction difficulties at the presence of groundwater, if condition "b" or "c" is encountered during excavation, the operation should be halted immediately, and the project soil engineer should be notified for further investigation and providing appropriate recommendations.

3.3.4.5 Immediately after Construction

This condition presents a relatively unlikely scenario whereupon the completed levee and its soil-cement cover is flooded (500-year flood level) while the lower portion of the trapezoidal-shaped trench is not backfilled yet. Under this scenario, except for rapid drawdown condition, granular soils (pervious soils) are modeled using drained shear strength parameters. Slope stability analyses for this case are only performed for levee slope. As presented in Appendix D, static and pseudo-static analysis results satisfied the minimum required FS for temporary condition, however, for the rapid drawdown condition, which is very unlikely, the factor of safety has fallen below the acceptable criteria. The appropriate measures to improve the stability of subject slope with respect to the rapid drawdown condition will be provided later in this report.

3.3.4.6 During Operation

This condition provides an evaluation of the performance of the levee and the false slopes during the normal life span of the project when all components of flood control system are built, as follows:

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- Levee Slope
  - ✓ For dry condition, both static and pseudo-static analysis results meet the required specification.
  - ✓ Prolonged levee inundation condition from full flood stage. In an unlikely event where the downstream valley is somehow dammed, say by flood debris, this condition may occur as the water remains at or near full 500-year flood level long enough so that the embankment becomes fully saturated. We understand that due to the topography of the site and downstream area, this situation is highly unlikely. However, the levee is considered stable under static condition.
  - ✓ When the water ponded in the above condition subsides gradually, the factor of safety of static condition is found to be acceptable.

- False Slope
  - ✓ Based on our communication with Q3, it is our understanding that the stability of false slope after the completion of construction is not required to be considered. However, we evaluated the highest false slope on the upstream portion of levee. The results for static and pseudo-static conditions are greater than minimum requirements.

3.3.4.7 Post Scour

The scour calculation performed by Q3 indicates that when a 500-year flood happens, majority the rip-rap and compacted soil located in front of levee may be washed away, and only minimum of 3 feet of material above protected toe of levee slope may stay in place. Considering this situation, we evaluated the rapid drawdown condition immediately after 500-year flood, along with static and pseudo-static conditions for long term stability after 500-year flood. The results of all analyses are acceptable with respect to the criteria set in Table 5.

3.4 Earthwork Recommendations

3.4.1 General

All earthwork should be performed in accordance with current industry standards of practice in the area, with all applicable requirements of the County of Riverside, as well as with the recommendations provided below and this firm's "Earthwork Specifications" (Appendix E).

3.4.2 Presence of Shallow Groundwater

Owing to the presence of perennial streams, shallow groundwater or perched groundwater may be encountered during construction, which may interfere with conventional earthwork operation. We recommend excavating a few borings and convert them to monitoring wells for frequent measurements of groundwater table well in advance of the construction activities. For conditions where high groundwater at

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levels higher than approximately 4 feet below the deepest level of excavation is anticipated, it is recommended that the contractor prepare plans for dewatering and other appropriate measures.

3.4.3 Clearing and Grubbing

All existing weeds, grasses, brush and similar vegetation existing within areas to be graded should be stripped and removed from the site. Clearing operations should also include the removal of debris, vegetation and similar deleterious materials. Note that deleterious materials may be encountered within the site and may need to be removed by hand (i.e. root pickers), during grading operations.

The project geotechnical consultant should provide periodic observation services during clearing and grubbing operations to document compliance with the above recommendations. In addition, should unusual or adverse soil conditions or buried structures be encountered during grading that are not described herein, these conditions should be brought to the immediate attention of the project geotechnical consultant for corrective recommendations.

It should be noted that the existing stream flow alignment should be relocated away from the construction area prior to beginning the earthwork operation to keep the water away from the excavation area as much as possible.

3.4.4 Ground Preparation

The near-surface soils are loose/weathered and are susceptible to settlement. As such, remedial grading of the near-surface compressible soils will be necessary for the area located behind or below the proposed soil-cement cover where is anticipated to receive compacted fill. It is recommended that all existing low-density, compressible surficial soils in areas to receive compacted fill should be removed to approximately 2 feet below the existing grades.

Additionally, while the access road is planned to be built in the cut area (i.e. sta. 28+00 and 28+70), 2 feet removal below proposed soil-cement layer for the access road is recommended. The lateral limits of the removals should extend at least 2 feet beyond the outside edges of the access road.

Soil removals may need to be locally deeper depending upon the exposed conditions encountered during grading. The actual depths and horizontal limits of removals and over-excavations should be evaluated during grading on the basis of observations and testing performed by the project geotechnical consultant. Prior to placing engineered fill, the exposed bottom surfaces in the removal areas should be approved by a representative of the project geotechnical consultant. The exposed bottom(s) should be scarified to a

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minimum depth of 12 inches, moisture conditioned to achieve at least two percent above optimum moisture content and compacted with a heavy vibratory roller prior to placement of additional fill. Minimum compaction of the upper 12 inches of the removal bottom should meet or exceed 90 percent relative compaction.

3.4.5 Fill Placement

All fills should be placed in 6- to 8-inch-thick maximum lifts, watered or air dried as necessary to achieve slightly above-optimum moisture conditions, and then compacted to a minimum relative compaction of 90 percent per ASTM D 1557. The laboratory maximum dry density and optimum moisture content for each change in soil type should be determined in accordance with Test Method ASTM D 1557. Compaction shall be achieved at or slightly above optimum moisture content, and as generally discussed in the attached "Standard Grading Specifications" (Appendix E). Mixing and moisture conditioning will be required in order to achieve the required moisture conditions.

It must be emphasized that the depths of remedial grading provided in the above paragraphs are estimates only and are based on conditions observed at the test pit locations and the similar project experience. Subsurface conditions can and usually do vary between points of exploration. For this reason, the actual removal depths will have to be determined on the basis of in-grading observations and testing performed by a representative of the project geotechnical consultant. Remedial grading and ground preparation should be performed prior to placing any new fills.

3.4.6 Excavation Characteristics

Based on the results of our subsurface evaluation, the native materials within the site are expected to be readily excavatable with conventional earthmoving equipment. However, the presence of oversized materials (cobbles and boulders), which may hamper the excavation process, should be considered.

3.4.7 Fill Slope Construction

Fill slopes that are anticipated for the landside and crest of levee embankment, shall be overfilled to an extent determined by the contractor, but not less than two feet measured perpendicular to the slope face, so that when trimmed back to the compacted core, the required compaction is achieved. Compaction of each fill lift should extend out to the temporary slope face.

As an alternative to overfilling, fill slopes may be built to the finish slope face in accordance with the following recommendations:

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- Compaction of each fill lift shall extend to the proposed face of the slopes.

- Backrolling during grading shall be undertaken at intervals not to exceed four feet in height. Backrolling at more frequent intervals may be required.

- Care shall be taken to avoid spillage of loose materials down the face of the slopes during grading.

- At completion of mass filling, the slope surface shall be watered, shaped and compacted first with a sheepsfoot roller, then with a grid roller operated from a side boom Cat, or equivalent, such that compaction to project standards is achieved to the slope face.

3.4.8 Boundary Conditions

As previously stated, the proposed development area is located away from the property boundary and is surrounded by vacant land. During grading of the site, temporary excavations with sidewalls varying up to approximately 34 feet high will be created during the excavation for the placement of soil-cement protection layer. Based on the relatively loose, non-cohesive nature of on-site soils and slope stability analysis results discussed above, the temporary excavations may not be able to stand vertically and should be sloped at a minimum ratio of at least 1.5:1, horizontal:vertical, or flatter as required to maintain stability.

Sidewalls of temporary excavations that are excavated to the above configurations are expected to remain sufficiently stable during grading. However, all temporary excavations should be observed by a representative of our firm for any evidence of potential instability. Depending upon the results of these observations, revised temporary slope configurations may become necessary. Furthermore, as discussed before, if groundwater or seepage is encountered during excavation at 10 feet above the proposed bottom of excavation on false slope and 17 feet on native levee slope, the operation should be halted immediately, and the project soil engineer should be notified for further investigation and corrective recommendations.

Other factors that should be considered with respect to the stability of temporary excavation sidewalls include construction traffic and storage of materials on or near the tops of the slopes, construction scheduling, and weather conditions at the time of construction. All applicable requirements of the California Construction and General Industry Safety Orders, the Occupational Safety and Health Act of 1970, and the Construction Safety Act should also be followed.

3.4.9 Suitability of On-Site Materials for Use as Engineered Fill

Based on our field observations and subsurface soil conditions encountered in our test pits, the vast majority of soil materials would be suitable for use as engineered fill; however, native alluvial soils are considered susceptible to wind and storm water runoff erosion. Oversize rock may be encountered during site grading.

Any materials exceedingly more than 12 inches in the greatest dimension should be removed from the fill. Depending on the volume of materials less than 12 inches but exceeding 3 inches in greatest dimension, a portion of these materials may need to be removed from the fill at the discretion of the representative of the project geotechnical consultant. As with most grading, the majority of soils exposed at or near the surface would require moisture conditioning to near optimum moisture for use as engineered fill.

3.4.10 Oversized Materials

Oversize rock is defined as hard boulders or rock fragments exceeding 12 inches in maximum dimension. Oversize rock observed during grading operations should be removed from the site or placed in the lower portions of the deeper fills utilizing the typical detail shown on Plate SG-4, Appendix E. Any oversize materials buried on site should be placed individually or in windrows, and in a manner to avoid nesting, and then completely covered with granular on-site earth materials. The granular materials should be thoroughly watered and rolled to ensure closure of all voids. Oversize rock should not be placed within the upper 10 feet of finish grade within the development areas.

3.4.11 Volumetric Changes - Shrinkage and Subsidence

The volume of oversized materials that should be removed from excavated soils prior to the finer materials to be placed as compacted fill is estimated to be in the range of 20-30 percent of the volume of the excavated soils. Several photos taken during our field exploration that shows the relative portion of oversized material, are presented in Appendix F.

Volumetric changes in earth quantities will also occur when onsite soils are excavated and replaced as properly compacted fill. Based on in-place densities of earth materials encountered during our evaluation, a shrinkage factor on the order of 20 to 30 percent may be anticipated during removal and re-compaction. More shrinkage should be anticipated if a portion of material between 3 to 12 inches in greatest dimension have to be removed. The actual shrinkage that will occur during grading will depend on the average degree of relative compaction achieved. A maximum subsidence of approximately 0.2 feet may be anticipated as a result of the scarification and re-compaction of the exposed bottom surfaces within the removal areas.

The above estimates of shrinkage and subsidence are intended for use by project planners in estimating earthwork quantities and should not be considered absolute values. Contingencies should be made for balancing earthwork quantities based on actual shrinkage and subsidence that will occur during site grading.

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3.4.12 Expansive Soil Conditions

Given the sandy nature of the near-surface soils we would expect that they are non-expansive (expansion Index ≤ 20).

3.4.13 Import Soils for Grading

It is our understanding that import soils are not likely needed for the site development. In case if it is required to be used, import soils should be free of deleterious materials, oversize rock and any hazardous materials. The soils should also be non-expansive and essentially non-corrosive and approved by the project geotechnical consultant *prior* to being brought onsite. The geotechnical consultant should visit the potential borrow site and conduct testing of the soil at least three days before the commencement of import operations.

Important Note: Petra recommends that a thorough screening of the prospective borrow site and, in most cases, laboratory testing be performed to determine if hazardous or otherwise toxic materials are present in the fill soil at concentrations that are above the established maximum allowable levels. This screening typically involves significant costs, and turnaround times for laboratory testing can typically range from several days to several weeks depending on the previous land usage at the borrow site location. If it is determined that imported soil will be required to establish the planned finished grades within the subject site, the additional costs and project delays should be taken into consideration well in advance of the start of grading operations. In addition, the project environmental consultant should be notified as soon as practical once the need for imported fill has been established so that they may be prepared to perform the required services at the appropriate time.

3.5 Soil-Cement Protection Layer Construction Guidelines

3.5.1 Material Specification and Design

3.5.1.1 General

The American Concrete Institute defines soil-cement as a mixture of soil and measured amounts of Portland cement and water compacted to a high density. Based on the development plan prepared by Q3, an 8-foot wide soil-cement layer, which is benched in a stair-step fashion into the native soils, is proposed to protect the flood control levee at the subject site. As such, the operation consists of constructing successive horizontal lifts of compacted soil-cement up the slope to the design height for protection of the levee mass. Layer thickness may be from 6 to 12 inches depending on the type of compaction equipment used.

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The following provides guidance on the design and construction of soil-cement slope protection for the levee. This includes guidelines for soil-cement materials, mixture proportioning, design of slope protection, construction, quality control, inspection, and testing. The guideline draws heavily from "Soil Cement Bank Protection Standards" prepared by the County of Los Angeles Department of Public Works (LACoDPW). However, where more stringent specifications are sought, the referenced guidelines are provided therein.

3.5.1.2 Construction Materials

Soil

It is recommended that the soil shall not contain any material retained on a 3-inch sieve or any organics or other deleterious materials. Deleterious materials such as sod, brush, and roots shall be separated and removed from the soil-cement materials. Additionally, the sand equivalent and plasticity index of soil should be greater than 15 and less than 8, respectively. The grain-size requirements or sieve size percent passing (dry weight) of soil aggregate to be used for soil-cement mixture should be within the following range:

- ✓ 3-inch sieve 100%
- ✓ ¾-inch sieve 80% - 100%
- ✓ #4 sieve 60% - 90%
- ✓ #40 sieve 30% - 50%
- ✓ #200 sieve 5% - 20%

Required soil for soil-cement may be obtained from the excavation of on-site soil or from other borrowed areas approved by the geotechnical engineer and stockpiled on the job site as specified later in this report. It's our understanding the onsite material will be used in the levee construction. The results of our laboratory analysis of onsite soil indicates that native soils, after screening, will be suitable to be used in soil-cement mixture. Therefore, material for soil-cement shall be obtained from excavation of the existing west bank where our test pits were located, and the stream channel deposits. Attention is directed to the fact that not all material excavated from the bank alignment will be acceptable for use as the soil component of the soil cement mixture. At the contractors' discretion, excavated area soils shall be screened before or after stockpiling, but prior to mixing to ensure being within acceptable range provided above. The screening results should be provided to the project geotechnical engineer for approval. The unsuitable materials, especially, the oversized materials, shall be separated during the excavation operation. The distribution and gradation of materials in the soil-cement lining shall not result in lenses, pockets, streaks, or layers of

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material differing substantially in texture or gradation from surrounding material. The sieve analysis results performed on the samples taken from our test pits are presented in Appendix B.

Cement

Portland cements meeting specifications of either ASTM C 150, CSA A-5, or AASHTO M85 are suitable. Generally, Type I is used for soil-cement. However, soil cement can be subject to sulfate attack and it is the lime in the cement that is involved in the reaction. Therefore, sulfate bearing soils or water should be avoided. There is no definitive test to determine the threshold sulfate content at which a soil is deemed to be potentially reactive. Because of that, we recommend using of Type II Portland cement for soil-cement mixture. Use of fly ash as a replacement for Portland cement is not recommended since, experience has indicated that fly ash reduces early age compressive strength and durability when used in soil cement.

Water

Most water is acceptable for soil-cement. The quality of water for soil-cement should be similar to that used for mixing concrete. The primary requirement is that water should be free from substances deleterious to hardening of the soil-cement. Specifically, water should be free from objectionable quantities of organic matter, alkali, salts, and other impurities. Presence of soluble sulfates should be of concern. Therefore, water should contain no more than 1,000 parts per million of chlorides as Cl or of sulfates as SO_4. Water shall be sampled and tested in accordance with the requirements of AASTO T26, or be potable water.

3.5.1.3 Proportioning Soil-Cement Mixtures

One of the key factors that accounts for the successful use of soil-cement is the careful predetermination of engineering control factors in the laboratory and their application during construction. The way a given soil reacts with cement is determined by simple laboratory tests conducted on mixtures of cement, soil, and water. These tests determine three fundamental requirements for soil-cement including the minimum cement content needed to harden the soil adequately, the proper moisture content and the density to which the soil-cement must be compacted.

Cement Content

A series of laboratory tests were performed to determine the influence of cement content on various engineering properties of the soil-cement mixture. As initial values, three different percentages of cement content, namely, 1, 3 and 5 percent, by dry weight of soil were chosen. The tests involved in this process included moisture density tests (ASTM D 558) to determine initial design density and optimum moisture content based on a selected initial cement content and direct shear strength tests (ASTM D 3080) to

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determine the strength required to stabilize the levee surface. Further, unconfined compressive strength tests (ASTM D 1632 and D 1633) were conducted on laboratory prepared specimens with final cement content for field strength determination purposes.

Moisture density tests were conducted following procedures indicated in ASTM D 558, Standard Test Methods for Moisture Density Relations of Soil Cement Mixtures to determine maximum density and optimum water content for molding soil-cement samples of direct shear and unconfined shear strength tests and for field compaction control during construction. The results of moisture/density tests are summarized in Table 6 and presented in Appendix B.

TABLE 6

Moisture Density Test Results

Sample Type	Cement Percentage (%)	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
Soil-Cement	1	124.0	10.0
	3	123.5	10.5
	5	123.5	11.0

The values reported in Table 6 were used to prepare remolded samples for direct shear testing. Shear strength parameters determined in our laboratory testing program are shown in Table 7.

TABLE 7

Shear Strength Test Results

Material	Cement Percentage (%)	Peak Value		Ultimate Value	
		Cohesion (C, psf)	Angle of Internal Friction (Ø, degrees)	Cohesion (C, psf)	Angle of Internal Friction (Ø, degrees)
Soil-Cement	1	360	36	168	29
	3	396	40	42	31
	5	426	44	5	34

These shear strength parameters we used in the static slope stability analysis that indicated the soil mixture with 5 percent cement can be used during construction to yield the minimum required factor of safety for stability of protected levee.

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As stated earlier, unconfined compressive strength tests were conducted to establish a procedure for the field soil-cement quality control. The test specimens were prepared and cured in general conformance with ASTM D1632 and D559 and then, compacted soil cement cylinders were tested according to ASTM D1633. It should be noted that we increased the amount of cement by 2 percent based on our previous experiences and engineering judgment, so that the soil-cement specimens for compression test contain 7 percent cement. A minimum density of 95 percent of the maximum ASTM D 558 density is considered suitable for soils-cement layer. As such, duplicate specimens were prepared at this density and tested in accordance with the ASTM procedures previously indicated. According to specification provided by USACE and local California agencies, the specified compressive strength of soil cement should be based on the 28-day test results. Additionally, 7-day test results can be used to monitor early strength gain. Minimum recommended unconfined compressive strength criteria for 7-day and 28-day cured samples are 750 and 875 psi, respectively. The results of compression tests on the samples with 7 percent are in the order of XXX and XXX for 7-day and 28-day samples, respectively.

3.5.1.4 Drainage and Seepage [Tentative section subject to consultation with project civil engineer]

Although no distress to soil cement slope protection due to rapid drawdown has been reported and the current thinking is that drainage is not required unless severe draw down is anticipated, the designer should be aware of the preventative measures can be used.

As stated above the landward slopes will be covered with loose soil which is susceptible to be eroded during flood. Although, this event will not jeopardize the levee structure performance. Three concepts are presented. One is design of the levee so that the least permeable zone is placed adjacent to the soil cement layer. This will provide protection against buildup of excess pore water pressure. A second method is to determine that the weight of the facing is sufficient to resist uplift pressures. Here, there may be some pore pressure relief through shrinkage cracks in the soil cement. Obviously, some estimate must be made of the gross hydraulic conductivity of the soil cement. A third measure is to provide deliberate drainage conduits through the soil cement. In this case, installation of 1 or 2 rows of weep holes drainage, at 20-foot spacing along the levee structure and above the bottom of river is recommended. The drainage system consists of:

- Polyvinyl chloride plastic pipe (PVC), Class 200 which shall conform to the requirements of 207-17.
- Sacked filter material shall consist of one cubic foot of No. 3 concrete aggregate per 200-1.4 in a burlap sack.
- Screen shall be PVC floor drain cover.

In such arrangements, a filter is placed in the area of weep hole before soil cement construction.

3.5.2 Construction Recommendation

The construction of soil-cement protection bank and its controlling measures should be performed in accordance with current industry standards of practice in the area, with all applicable requirements of the County of Riverside, as well as with the recommendations provided in this firm's "Soil-Cement Bank Protection Guidelines" (Appendix G).

3.6 General Corrosivity Screening

As a screening level study, limited chemical and electrical tests were performed on samples considered representative of the onsite soils to identify potential corrosive characteristics of these soils. The common indicators associated with soil corrosivity include water-soluble sulfate level, pH (a measure of acidity), and minimum electrical resistivity. Test methodology and results are presented in Appendix B and Table 8, respectively.

It should be noted that Petra does not practice corrosion engineering; therefore, the test results, opinion and engineering judgment provided herein should be considered as general guidelines only. Additional analyses would be warranted, especially, for cases where buried metallic building materials (such as copper and cast or ductile iron pipes) in contact with site soils are planned for the project. In many cases, the project geotechnical engineer may not be informed of these choices. Therefore, for conditions where such elements are considered, we recommend that other, relevant project design professionals (e.g., the architect, landscape architect, civil and/or structural engineer) also consider recommending a qualified corrosion engineer to conduct additional sampling and testing of near-surface soils during the final stages of site grading to provide a complete assessment of soil corrosivity. Recommendations to mitigate the detrimental effects of corrosive soils on buried metallic and other building materials that may be exposed to corrosive soils should be provided by the corrosion engineer as deemed appropriate.

In general, a soil's water-soluble sulfate levels and pH relate to the potential for concrete degradation; and electrical resistivity is a measure of a soil's corrosion potential to a variety of buried metals used in the building industry, such as copper tubing and cast or ductile iron pipes. Although, the development plan prepared by Q3 does not show any metallic elements used in the proposed structure. Table 8, below, presents a single value of individual test results with an interpretation of current code indicators and guidelines that are commonly used in this industry. The table includes the code-related classifications of the soils as they relate to the various tests, as well as a general recommendation for possible mitigation measures in view of the potential adverse impact on various components of the proposed structures in direct contact with site soils. The guidelines provided herein should be evaluated and confirmed, or modified, in

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their entirety by the project structural engineer, corrosion engineer and/or the contractor responsible for soil-cement production and placement.

TABLE 8

Soil Corrosivity Screening Results

Test	Test Results	Classification	General Recommendations
Soluble Sulfates (Cal 417)	0.0009 percent	S0[1]	Type II cement; min. f'c= 2,500 psi; no water/cement ratio restrictions
pH (Cal 643)	7.96	Moderately Alkaline	No special recommendations
Resistivity (Cal 643)	24,000 ohm-cm	Essentially Noncorrosive[2]	No special recommendations

Notes:
1. ACI 318-14, Section 19.3
2. Pierre R. Roberge, "Handbook of Corrosion Engineering"

4.0 FUTURE IMPROVEMENTS AND GRADING

If additional exterior improvements are considered in the future, our firm should be notified so that we may provide design recommendations to mitigate movement, settlement and/or tilting of the structures. It is further recommended that we be engaged to review the final design drawings, specifications and grading plan prior to any new construction. If we are not provided the opportunity to review these documents with respect to the geotechnical aspects of new construction and grading, it should not be assumed that the recommendations provided herein are wholly or in part applicable to the proposed construction.

5.0 REPORT LIMITATIONS

This report is based on the existing conditions of the subject property, limited subsurface exploration, our geologic and geotechnical research of available maps and data, proposed development and geotechnical data as described herein. As stated, when site plans have been developed, detailed subsurface investigation and geotechnical testing and analysis, may be necessary. The materials encountered on the project site and described in other literature are believed representative of the project area, and the conclusions and recommendations contained in this report are presented on that basis. However, soil materials can vary in characteristics between points of exploration, both laterally and vertically, and those variations could affect the conclusions and recommendations contained herein. As such, observation and testing by our firm during the grading and construction phases of the project are essential to confirming the basis of this report. To

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provide the greatest degree of continuity between the design and construction phases, consideration should be given to retaining Petra Geosciences, Inc. for construction services.

This report has been prepared consistent with that level of care being provided by other professionals providing similar services at the same locale and time period. The contents of this report are professional opinions and as such, are not to be considered a guarantee or warranty. This report should be reviewed and updated after a period of one year or if the project concept changes from that described herein. The information contained herein has not been prepared for use by parties or projects other than those named or described herein. This report may not contain sufficient information for other parties or other purposes.

It has been a pleasure to be of service to you on this project. Should you have questions regarding the contents of this report, or should you require additional information, please contact this office.

Respectfully submitted,

PETRA GEOSCIENCES, INC.

Siamak Jafroudi, Ph.D.
Senior Principal Engineer
GE 2024

Alan Pace
Senior Associate Geologist
CEG 1952

Ali Maroufi
Senior Staff Engineer
EIT

AM/AP/SJ/

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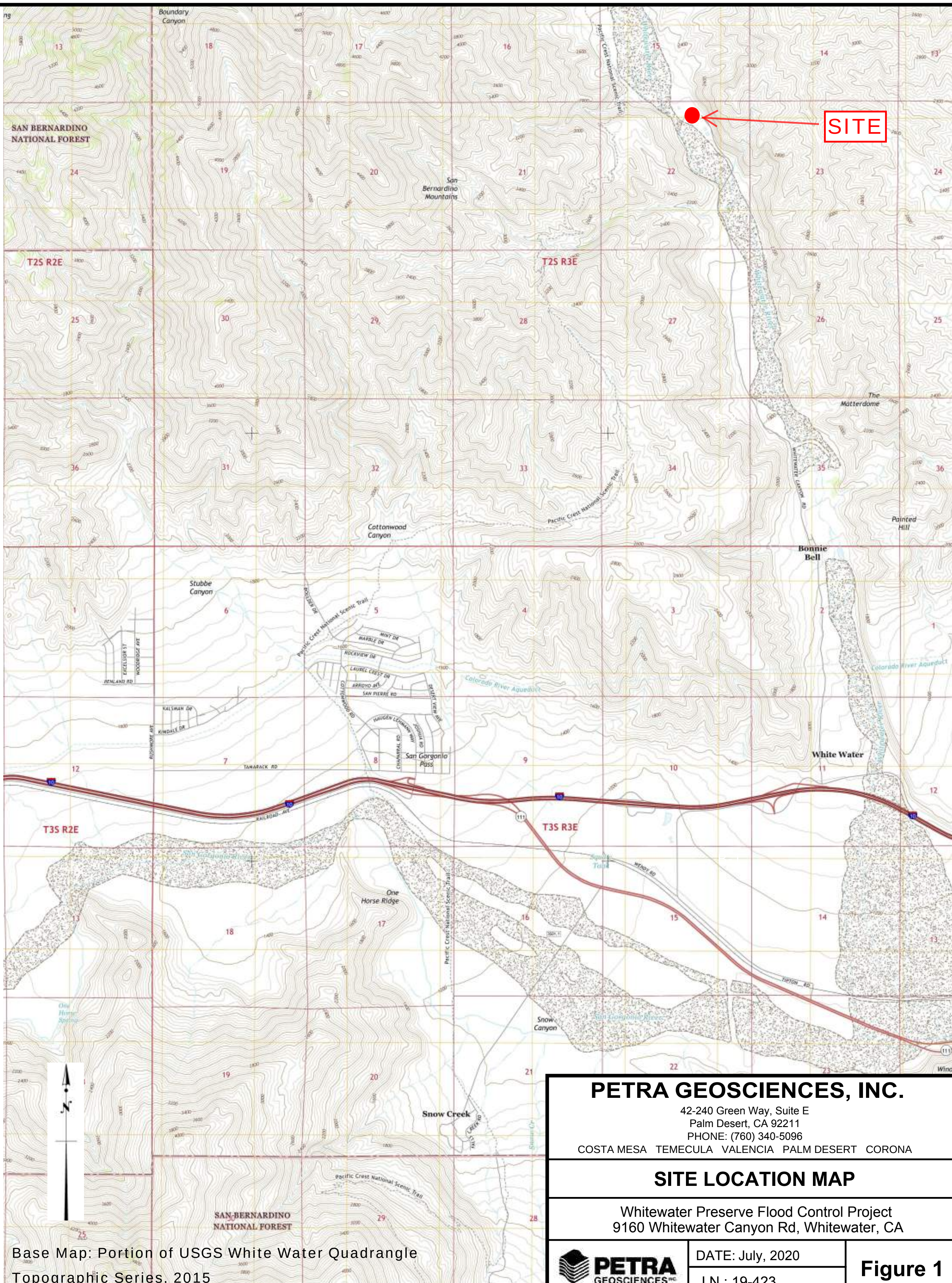


Base Map: Portion of USGS White Water Quadrangle Topographic Series, 2015

PETRA GEOSCIENCES, INC.
42-240 Green Way, Suite E
Palm Desert, CA 92211
PHONE: (760) 340-5096
COSTA MESA TEMECULA VALENCIA PALM DESERT CORONA

SITE LOCATION MAP

Whitewater Preserve Flood Control Project
9160 Whitewater Canyon Rd, Whitewater, CA

PETRA GEOSCIENCES INC.	DATE: July, 2020	**Figure 1**
	J.N.: 19-423	

LEGEND

TP-5 Test Pits Performed by Petra Geosciences, Inc. (2020)

Approximate Limits of the Proposed Levee Alignment

Note: All Locations and Limits are Approximate.

PETRA GEOSCIENCES, INC.

42-240 Green Way, Suite E
Palm Desert, CA 92211
PHONE: (760) 340-5096
COSTA MESA TEMECULA VALENCIA PALM DESERT CORONA

TEST PIT LOCATION MAP

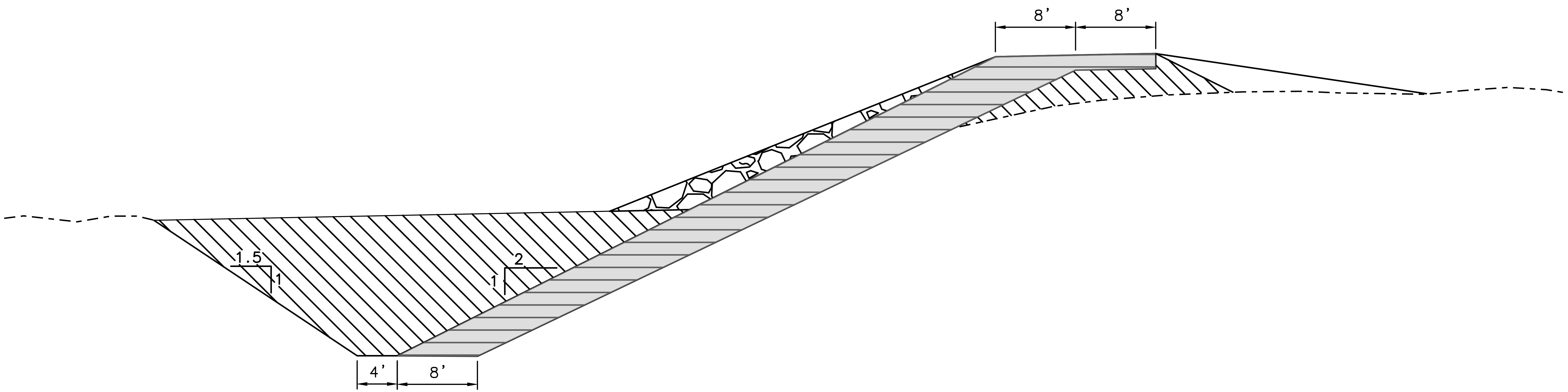
Whitewater Preserve Flood Control Project
9160 Whitewater Canyon Rd, Whitewater, CA

PETRA GEOSCIENCES

DATE: July, 2020

J.N.: 19-432

Figure 2



LEGEND

Schematic Level of Existing Grade

Approximate Configuration of Soil-Cement Protection

Approximate Configuration of Compacted Fill

Approximate Configuration of Rip-Rap Layer of Native Rocks

Note: All Locations and Configurations Are Conceptual.

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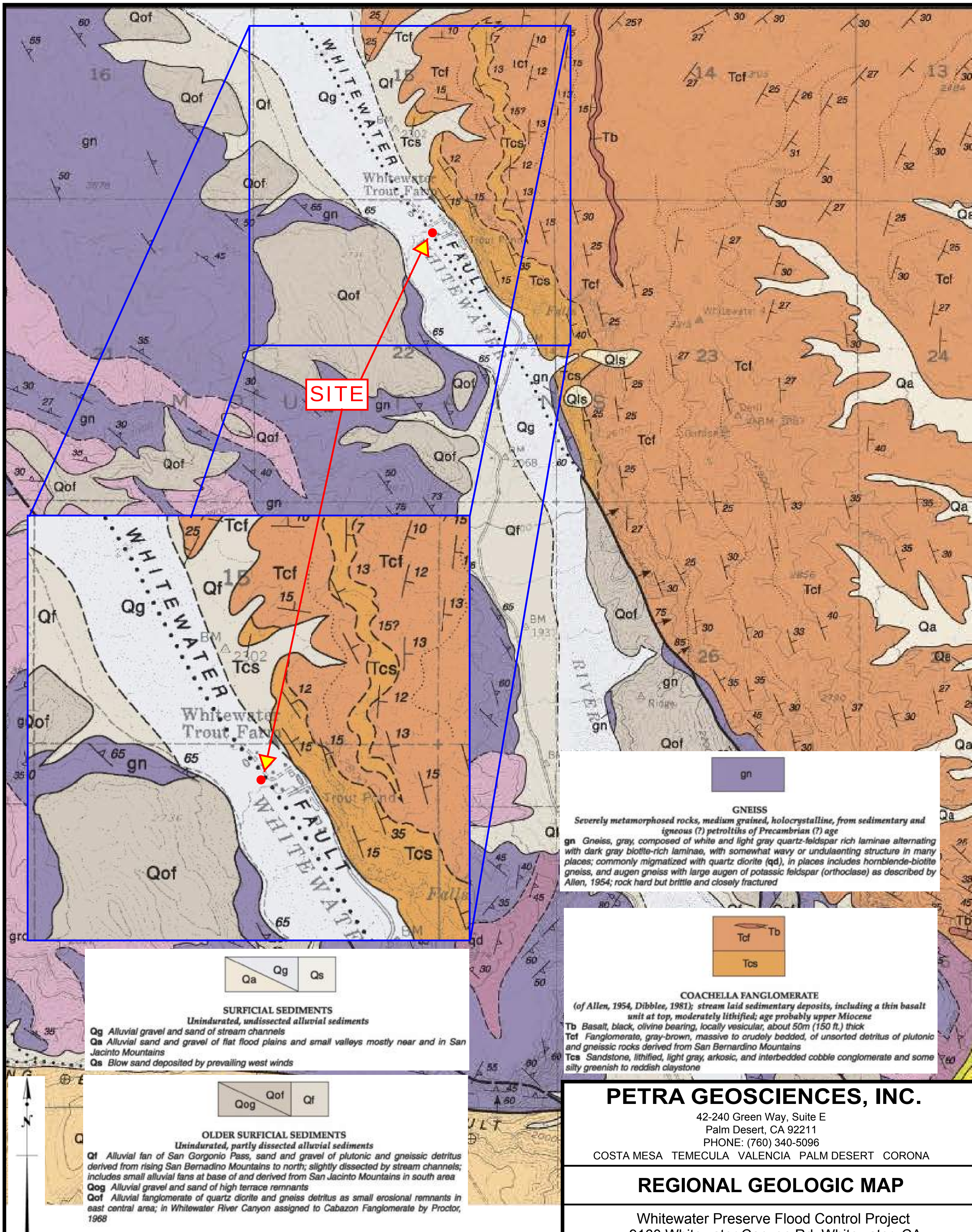
42-240 Green Way, Suite E
Palm Desert, CA 92211
PHONE: (760) 340-5096
COSTA MESA TEMECULA VALENCIA PALM DESERT CORONA

TYPICAL CROSS-SECTION OF THE LEVEE

Whitewater Preserve Flood Control Project
9160 Whitewater Canyon Rd, Whitewater, CA

PETRA GEOSCIENCES	DATE: July, 2020	**Figure 3**
	J.N.: 19-432	



Qa | Qg | Qs

SURFICIAL SEDIMENTS
Unindurated, undissected alluvial sediments
Qg *Alluvial gravel and sand of stream channels*
Qa *Alluvial sand and gravel of flat flood plains and small valleys mostly near and in San Jacinto Mountains*
Qs *Blow sand deposited by prevailing west winds*

Qog | Qof | Qf

OLDER SURFICIAL SEDIMENTS
Unindurated, partly dissected alluvial sediments
Qf *Alluvial fan of San Gorgonio Pass, sand and gravel of plutonic and gneissic detritus derived from rising San Bernadino Mountains to north; slightly dissected by stream channels; includes small alluvial fans at base of and derived from San Jacinto Mountains in south area*
Qog *Alluvial gravel and sand of high terrace remnants*
Qof *Alluvial fanglomerate of quartz diorite and gneiss detritus as small erosional remnants in east central area; in Whitewater River Canyon assigned to Cabazon Fanglomerate by Proctor, 1968*

gn

GNEISS
Severely metamorphosed rocks, medium grained, holocrystalline, from sedimentary and igneous (?) petroltihs of Precambrian (?) age
gn *Gneiss, gray, composed of white and light gray quartz-feldspar rich laminae alternating with dark gray biotite-rich laminae, with somewhat wavy or undulaenting structure in many places; commonly migmatized with quartz diorite (**qd**), in places includes hornblende-biotite gneiss, and augen gneiss with large augen of potassic feldspar (orthoclase) as described by Allen, 1954; rock hard but brittle and closely fractured*

Tcf | Tb
Tcs

COACHELLA FANGLOMERATE
(of Allen, 1954, Dibblee, 1981); stream laid sedimentary deposits, including a thin basalt unit at top, moderately lithified; age probably upper Miocene
Tb *Basalt, black, olivine bearing, locally vesicular, about 50m (150 ft.) thick*
Tcf *Fanglomerate, gray-brown, massive to crudely bedded, of unsorted detritus of plutonic and gneissic rocks derived from San Bernardino Mountains*
Tcs *Sandstone, lithified, light gray, arkosic, and interbedded cobble conglomerate and some silty greenish to reddish claystone*

Base Map: Portion of Geologic map of the Whitewater 15 Minute Quadrangles By Dibblee, T.W., and Minch J.A., 2004 From National Geologic Map Database

PETRA GEOSCIENCES, INC.
42-240 Green Way, Suite E
Palm Desert, CA 92211
PHONE: (760) 340-5096
COSTA MESA TEMECULA VALENCIA PALM DESERT CORONA

REGIONAL GEOLOGIC MAP

Whitewater Preserve Flood Control Project
9160 Whitewater Canyon Rd, Whitewater, CA

PETRA GEOSCIENCES

DATE: July, 2020
J.N.: 19-423

Figure 4

APPENDIX A

BORING LOGS

DRAFT

Key to Soil and Bedrock Symbols and Terms

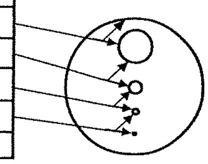
PETRA

Unified Soil Classification System

Coarse-grained Soils > 1/2 of materials is larger than #200 sieve	The No. 200 U.S. Standard Sieve is about the smallest particle visible to the naked eye	GRAVELS more than half of coarse fraction is larger tban #4 sieve	Clean Gravels (less than 5% fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines
				GP	Poorly-graded gravels, gravel-sand mixtures, little or no fines
			Gravels with fines	GM	Silty Gravels, poorly-graded gravel-sand-silt mixtures
				GC	Clayey Gravels, poorly-graded gravel-sand-clay mixtures
		SANDS more than half of coarse fraction is smaller than #4 sieve	Clean Sands (less than 5% fines)	SW	Well-graded sands, gravelly sands, little or no fines
				SP	Poorly-graded sands, gravelly sands, little or no fines
			Sands with fines	SM	Silty Sands, poorly-graded sand-gravel-silt mixtures
				SC	Clayey Sands, poorly-graded sand-gravel-clay mixtures
Fine-grained Soils > 1/2 of materials is smaller than #200 sieve		SILTS & CLAYS Liquid Limit Less Than 50		ML	Inorganic silts & very fine sands, silty or clayey fine sands, clayey silts with sligbt plasticity
				CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
				OL	Organic silts & clays of low plasticity
		SILTS & CLAYS Liquid Limit Greater Than 50		MH	Inorganic silts, micaceous or diatomaceous fine sand or silt
				CH	Inorganic clays of higb plasticity, fat clays
				OH	Organic silts and clays of medium-to-high plasticity
Highly Organic Soils				PT	Peat, humus swamp soils with high organic content

Grain Size

Description		Sieve Size	Grain Size	Approximate Size
Boulders		>12”	>12”	Larger than basketball-sized
Cobbles		3 - 12”	3 - 12”	Fist-sized to basketball-sized
Gravel	coarse	3/4 - 3”	3/4 - 3”	Thumb-sized to fist-sized
	fine	#4 - 3/4”	0.19 - 0.75”	Pea-sized to thumb-sized
Sand	coarse	#10 - #4	0.079 - 0.19”	Rock salt-sized to pea-sized
	medium	#40 - #10	0.017 - 0.079”	Sugar-sized to rock salt-sized
	fine	#200 - #40	0.0029 - 0.017”	Flour-sized to sugar-sized to
Fines		Passing #200	<0.0029”	Flour-sized and smaller



Laboratory Test Abbreviations

MAX	Maximum Dry Density	MA	Mechanical (Particle Size) Analysis
EXP	Expansion Potential	AT	Atterberg Limits
SO4	Soluble Sulfate Contcnt	#200	#200 Screen Wash
RES	Resistivity	DSU	Direct Shear (Undisturbed Sample)
pH	Acidity	DSR	Direct Shear (Remolded Sample)
CON	Consolidation	HYD	Hydrometer Analysis
SW	Swell	SE	Sand Equivalent
CL	Chloride Content	OC	Organic Content
RV	R-Value	COMP	Mortar Cylinder Compression

Modifiers

Trace	< 1 %
Few	1 - 5%
Some	5 - 12 %
Numerous	12 - 20 %

Sampler and Symbol Descriptions

- Approximate Depth of Seepage
- Approximate Depth of Standing Groundwater
- Modified California Split Spoon Sample
- Standard Penetration Test
- Bulk Sample
- Shelby Tube
- No Recovery in Sampler

Bedrock Hardness

Soft	Can be crushed and granulated by hand; “soil like” and structureless
Moderately Hard	Can be grooved with fingernails; gouged easily with butter knife; crumbles under light hammer blows
Hard	Cannot break by hand; can be grooved with a sharp knife; breaks with a moderate hammer blow
Very Hard	Sharp knife leaves scratch; chips with repeated hammer blows

Notes:

Blows Per Foot: Number of blows required to advance sampler 1 foot (unless a lesser distance is specified). Samplers in general were driven into the soil or bedrock at the bottom of the bole with a standard (140 lb.) hammer dropping a standard 30 inches unless noted otherwise in Log Notes. Drive samples collected in bucket auger borings may be obtained by dropping non-standard weight from variable heigbts. When a SPT sampler is used tbe blow count conforms to ASTM D-1586

EXPLORATION LOG

Project:	Whitewater Preserve Flood Control	Boring No.:	TP-1
Location:	9160 Whitewater Canyon Road, Whitewater, Riverside County, California	Elevation:	+/-2224'
Job No.: 19-423	Client: Q3 Consulting	Date:	03/27/2020
Drill Method: Excavated by Backhoe	Driving Weight: Machine Driven	Logged By:	KTM

Depth (Feet)	Lith-ology	Material Description	WATER	Samples: Blows per 6 in.	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0		**ARTIFICIAL FILL (af) Stream Channel Deposit** Gravelly Sand (GP-SP): Grayish-brown, slightly moist, loose to medium dense, fine- to coarse-grained sand, poorly graded, rounded gravel and cobbles. @0.5': Very large boulder along side of pit down to 4.5', 18" long, 16" thick. From 1' to 2.5': pieces of garbage.							
2		@2': railroad tie.							
		Approximately: 15% Boulders (granite) 1' to 3' 20% Cobbles 3" - 12" 10% gravel 55% sand.							
4		No Recovery - too rocky.		10					
6		Test Pit terminated at 6' No groundwater encountered The pit was backfilled with cuttings and a large boulder was left in the pit (too large to remove).							
8									
10									
12									
14									

PLATE

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EXPLORATION LOG

Project:	**Whitewater Preserve Flood Control**	Boring No.:	**TP-2**
Location:	**9160 Whitewater Canyon Road, Whitewater, Riverside County, California**	Elevation:	**+/-2218'**
Job No.: **19-423**	Client: **Q3 Consulting**	Date:	**03/27/2020**
Drill Method: **Excavated by Backhoe**	Driving Weight: **Machine Driven**	Logged By:	**KTM**

Depth (Feet)	Lithology	Material Description	WATER	Samples: Blows per 6 in.	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0		**ARTIFICIAL FILL (af) Stream Channel Deposit** Gravelly Sand (GP-SP): Brown, slightly moist to moist, loose to medium dense, fine- to coarse-grained sand, poorly graded.							
2									
3		@3': No Recovery.		16/11"		■			
4		Becomes grayish-brown, moist, medium- to coarse-grained sand, Rounded Approximately: 10% Boulders (granite) 1' to 3' 20% Cobbles 3" - 12" 10% gravel 60% sand tree roots.				■			
5		@5': increase in cobble contents.		12/12"		■	3.0	123.3	
				22/12"	■	■			
6						■			
7.5		Test Pit terminated at 7.5' (Refusal) No groundwater encountered The pit was backfilled with cuttings.							
8									
10									
12									
14									

PLATE

EXPLORATION LOG

Project:	**Whitewater Preserve Flood Control**	Boring No.:	**TP-3**
Location:	**9160 Whitewater Canyon Road, Whitewater, Riverside County, California**	Elevation:	**+/-2208'**
Job No.: **19-423**	Client: **Q3 Consulting**	Date:	**03/27/2020**
Drill Method: **Excavated by Backhoe**	Driving Weight: **Machine Driven**	Logged By:	**KTM**

Depth (Feet)	Lith-ology	Material Description	WATER	Samples: Blows per 6 in.	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0		**ARTIFICIAL FILL (af) Stream Channel Deposit** Gravelly Sand (GP-SP): Grayish-brown, moist, loose to medium dense, fine-to coarse-grained sand, poorly graded, Approximately: 5% Boulders (granite) 1' to 3' 30% Cobbles 3" - 12" 5% gravel 60% sand.							
2									
		@3': No Recovery.		15/12"					
4		@4': increase in sand contents.		12/12"					
		@5': increase in boulder contents becomes gray.							
6									
		Test Pit terminated at 7.5' No groundwater encountered The pit was backfilled with cuttings.							
8									
10									
12									
14									

PLATE

Petra Geosciences, Inc.

EXPLORATION LOG

Project:	**Whitewater Preserve Flood Control**	Boring No.:	**TP-4**
Location:	**9160 Whitewater Canyon Road, Whitewater, Riverside County, California**	Elevation:	**+/-2187'**
Job No.: **19-423**	Client: **Q3 Consulting**	Date:	**03/27/2020**
Drill Method: **Excavated by Backhoe**	Driving Weight: **Machine Driven**	Logged By:	**KTM**

Depth (Feet)	Lith-ology	Material Description	WATER	Samples: Blows per 6 in.	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0–8		**ARTIFICIAL FILL (af) Stream Channel Deposit** Gravelly Sand to Sandy Gravel (GP-SP): Grayish-brown, moist, loose to medium dense, fine- to coarse-grained sand, poorly graded, Approximately: 0% Boulders (granite) 1' to 3' 5% Cobbles 3" - 12" 20% gravel 75% sand trace roots. @1.5': becomes Approximately: 5% Boulders (granite) 1' to 3' 10% Cobbles 3" - 12" 20% gravel 65% sand. @3': No Recovery. @6': increase in boulder contents.		25/12"					
8		Test Pit terminated at 8.0' (Refusal) No groundwater encountered The pit was backfilled with cuttings.							
10									
12									
14									

PLATE

Petra Geosciences, Inc.

EXPLORATION LOG

Project:	**Whitewater Preserve Flood Control**	Boring No.:	**TP-5**
Location:	**9160 Whitewater Canyon Road, Whitewater, Riverside County, California**	Elevation:	**+/-2177'**
Job No.:	**19-423** Client: **Q3 Consulting**	Date:	**03/27/2020**
Drill Method:	**Excavated by Backhoe** Driving Weight: **Machine Driven**	Logged By:	**KTM**

Depth (Feet)	Lith-ology	Material Description	WATER	Samples: Blows per 6 in.	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0		**ARTIFICIAL FILL (af) Stream Channel Deposit** Silty Gravelly Sand (SM): Grayish-brown, moist, loose, fine- to medium-grained sand.							
		Gravelly Sand with Silt (GP-SP): Grayish-brown, moist, loose to medium dense, medium- to coarse-grained sand, poorly graded, trace roots Approximately: 5% Boulders (granite) 1' to 3' 10% Cobbles 3" - 12" 20% gravel 65% sand.							
5		@5': No Recovery.		20/12"					
8.5		Test Pit terminated at 8.5' (Refusal) No groundwater encountered The pit was backfilled with cuttings.							

PLATE

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APPENDIX B

LABORATORY TEST PROCEDURES

LABORATORY DATA SUMMARY

DRAFT

LABORATORY TESTING

Associated with the subsurface exploration was the collection of bulk and relatively undisturbed samples of soil materials for laboratory testing. The relatively undisturbed samples were obtained using a 3-inch, outside-diameter, modified California split-spoon soil sampler lined with 1-inch-high brass rings. The driven ring samples were placed in sealed containers and transported to our laboratory located at 1251 W. Pomona Road, Unit #103, Corona, CA 92882, for testing.

Our laboratory testing capabilities include Soil Classifications, Moisture Content and In-Situ Moisture Content and Dry Unit Weight, Organic Content, Laboratory Maximum Dry Unit Weight and Optimum Moisture Content, Corrosivity Screening (Soluble Sulfate and Chloride Content, pH, Resistivity) and Direct Shear; all in accordance with the latest procedures of American Society for Testing and Materials (ASTM) and California Department of Transportation (Caltrans).

To evaluate the engineering properties of site soils, laboratory testing was performed on selected samples of soil considered representative of those encountered. Appropriate tests were assigned by the project engineer and geologist based on project plans and specifications including the level of anticipated loads, when available, and subsurface stratigraphy. Test results were reviewed by the laboratory manager and engineer-in-charge of the laboratory or his qualified designee for completeness and accuracy. A description of laboratory test procedures and summaries of the test data are presented in the following pages.

DRAFT

LABORATORY TEST PROCEDURES

Soil Classification

Soils encountered within the exploration borings were initially classified in the field in general accordance with the visual-manual procedures of the Unified Soil Classification System (ASTM D2488). The samples were re-examined in the laboratory and the classifications reviewed and then revised where appropriate. The assigned group symbols are presented in the exploration logs, Appendix A.

In-Situ Moisture and Density

Moisture content and unit dry density of in-place soil samples were determined in accordance with the current version of Test Method ASTM D2435 and Test Method ASTM D2216, respectively. Test data are presented in the exploration logs, Appendix A.

Laboratory Maximum Dry Density

Maximum dry density and optimum moisture content were determined for selected bulk sample of soil and mixture of soil-cement with different percentage of cement in accordance with current version of Method A of ASTM D1557. The results of this test are included on Plates B-1 and presented in Plate B-2.

Corrosivity Tests

Chemical and electrical analyses were performed on selected bulk samples of onsite soils to determine their soluble sulfate content, chloride content, pH (acidity) and minimum electrical resistivity. These tests were performed in accordance with the latest versions of California Test Method Nos. CTM 417 (sulfate), CTM 422 (chloride), and CTM 643 (pH and resistivity) respectively. The results of these tests are included on Plate B-1.

Grain Size Distribution

Grain size analysis was performed on selected bulk samples of onsite soils in accordance with the latest versions of Test Method ASTM D 136 and/or ASTM C 117, or Test Method ASTM D 422 and/or ASTM D 6913. The test result is graphically presented on Plates B-3 through B-4.

Direct Shear

The Coulomb shear strength parameters, i.e., angle of internal friction and cohesion, were determined for selected, reconstituted-bulk samples of onsite soil and compacted soil-cement. This test was performed in general accordance with the current version of Test Method ASTM D3080. Three specimens were prepared for each test. The test specimens were inundated and then sheared under various normal loads at a constant strain rate of 0.005 inch per minute. The results of the direct shear test are graphically presented on Plate B-5 through B-10.

Unconfined Compressive Strength

The compression strength of soil-cement samples with final cement content were tested in conformance with ASTM D1633. The test specimens with final cement content were prepared and cured in general conformance with ASTM 1632 and then, compacted soil cement cylinders were tested according to ASTM D1633. The results of the compressive strength tests are graphically presented on Plate B-11 through B-XX.

DRAFT

LABORATORY DATA SUMMARY

Boring/ Test Pit/ Sample/ Number	Sample Depth (ft.)	Soil/ Bedrock Description[1]	Specific Gravity[2]	Compaction[3]		Expansion[4]		Atterberg Limits[5]			Corrosivity Screening				Percent Passing No. 200 Sieve[9]	Sand Equivalent[10]	R-Value[11]	Organic Content[12] (%)
				Maximum Dry Unit Weight (pcf)	Optimum Moisture (%)	Index	Potential	LL	PL	PI	Soluble Sulfate Content[6] (%)	Chloride Content[7] (ppm)	pH[8] (Acidity)	Minimum Resistivity[8] (Ohm-cm)				
TPT-3	0-8’	Poorly Graded Sand with Gravel (SP)		121.5	10.5						0.0009		7.96	24,000				

Test Procedures:

1 Per Test Method ASTM D 2488
2 Per Test Method ASTM D 854
3 Per Test Method ASTM D 1557
4 Per Test Method ASTM D 4829
5 Per Test Method ASTM D 4318
6 Per California Test Method CTM 417
7 Per California Test Method CTM 422
8 Per California Test Method CTM 643
9 Per Test Method ASTM C 117
10 Per Test Method ASTM D 2419
11 Per California Test Method CTM 301
12 Per Test Method ASTM D 2974

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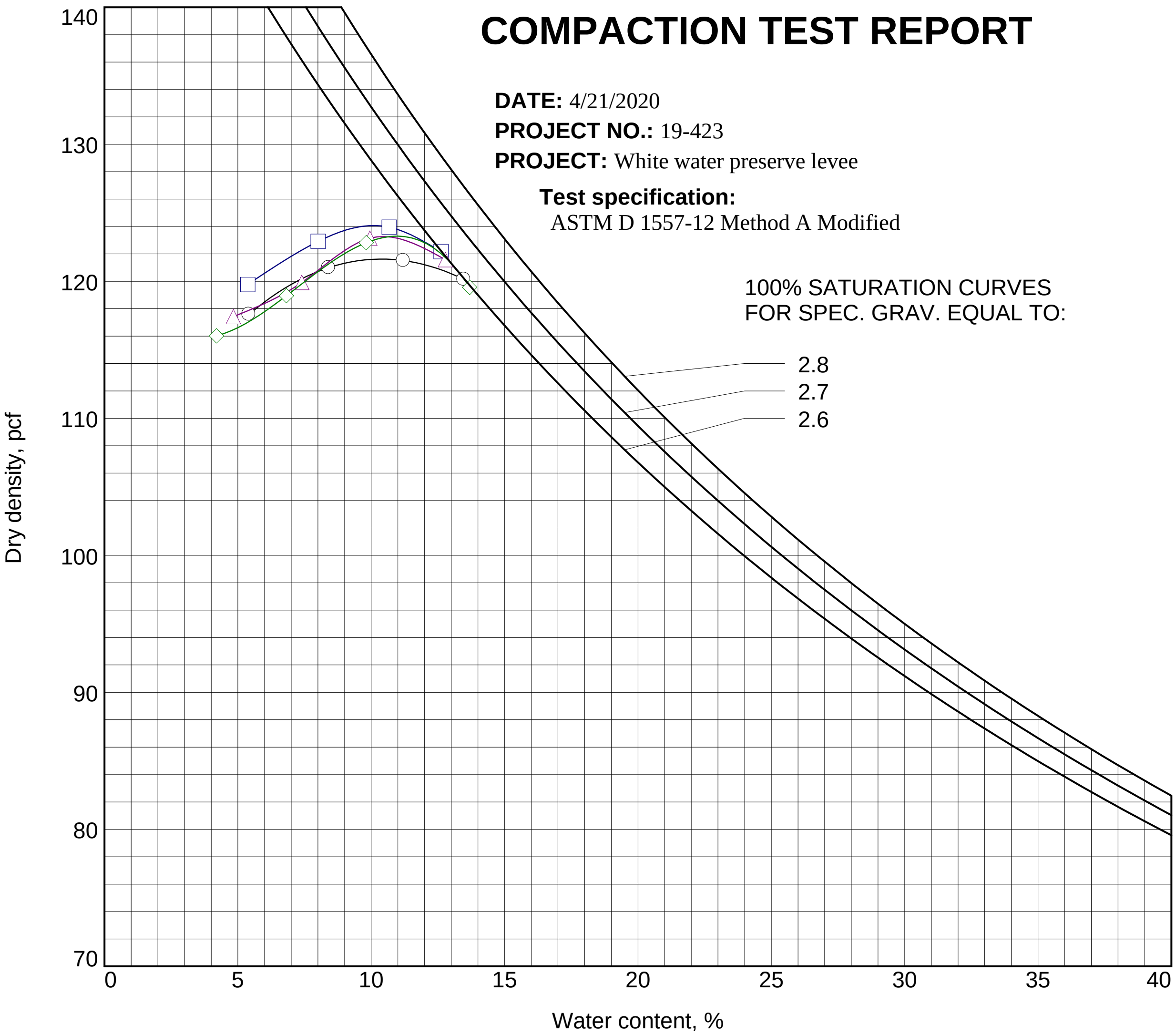
COMPACTION TEST REPORT

DATE: 4/21/2020
PROJECT NO.: 19-423
PROJECT: White water preserve levee

Test specification:
ASTM D 1557-12 Method A Modified



No.	LOCATION AND DESCRIPTION	REMARKS
○	**Source:** Soil-Cement **Sample No.:** combined Sample 2 Gray, Gravelly Sand	Proctor retest
□ 1% PC	**Source of Sample:** Soil-Cement **Sample Number:** 1% PC	
△ 3% PC	**Source of Sample:** Soil-Cement **Sample Number:** 3% PC	
◇ 5% PC	**Source of Sample:** Soil-Cement **Sample Number:** 5% PC	

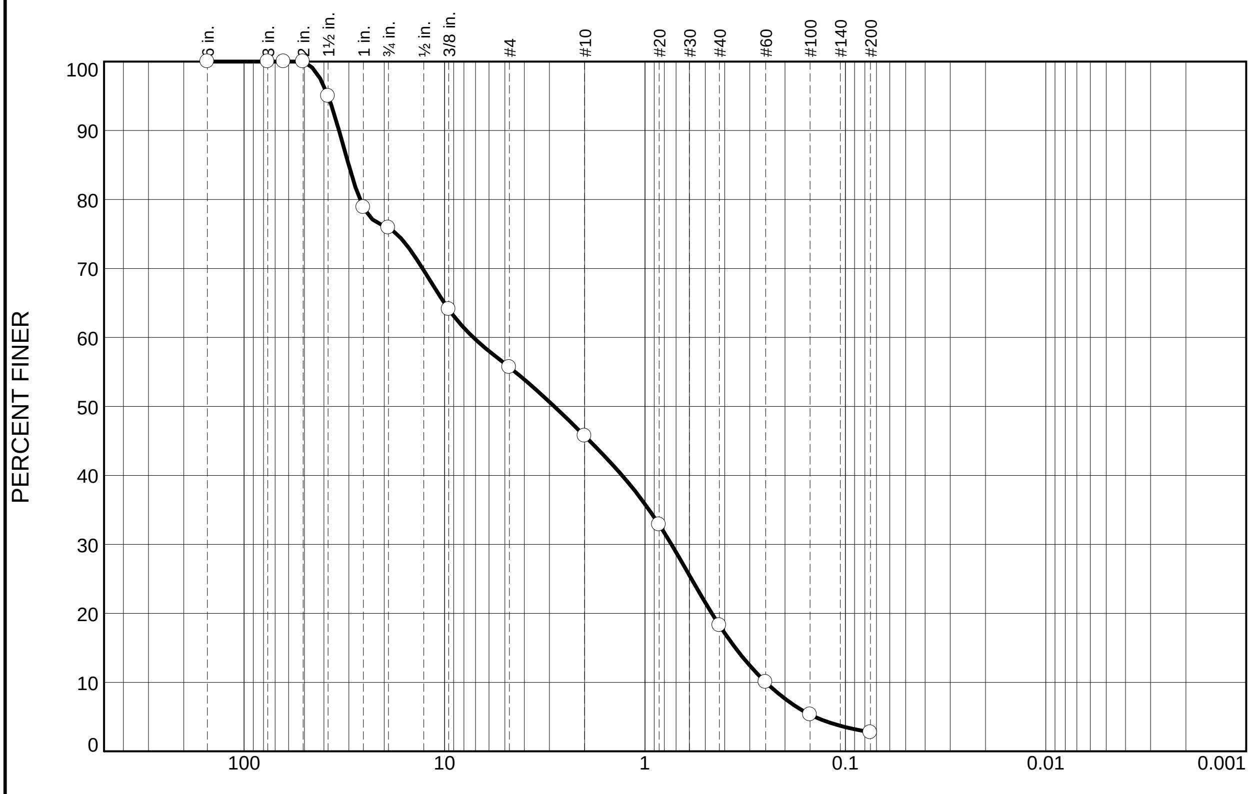
No.	USCS	LL	PI	NAT. MOIST.	OVERSIZE	%< No.200	MAX. DRY DEN.	OPT. MOIST.
○	SP						121.5	10.5 %
□ 1% PC							124.0	10.0 %
△ 3% PC							123.5	10.5 %
◇ 5% PC							123.5	11.0 %

PLATE B-2

Petra Geosciences, Inc.

Laboratory:
1251 West Pomona Road, Unit #103, Corona, Ca 92882 Phone #. 714.549.8921

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	24.1	20.2	10.0	27.4	15.5	2.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
6	100.0		
3	100.0		
2.5	100.0		
2	100.0		
1.5	95.0		
1	78.9		
3/4	75.9		
3/8	64.1		
#4	55.7		
#10	45.7		
#20	32.9		
#40	18.3		
#60	10.0		
#100	5.3		
#200	2.8		

* (no specification provided)

Material Description

Gray, Gravelly Sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 33.6176 D_{85}= 30.0817 D_{60}= 7.1823
D_{50}= 2.8348 D_{30}= 0.7381 D_{15}= 0.3540
D_{10}= 0.2492 C_u= 28.83 C_c= 0.30

Classification

USCS= SP AASHTO=

Remarks

Sieve analysis was performed exclusively on bulk bag sample TP-2 @ 3-7.

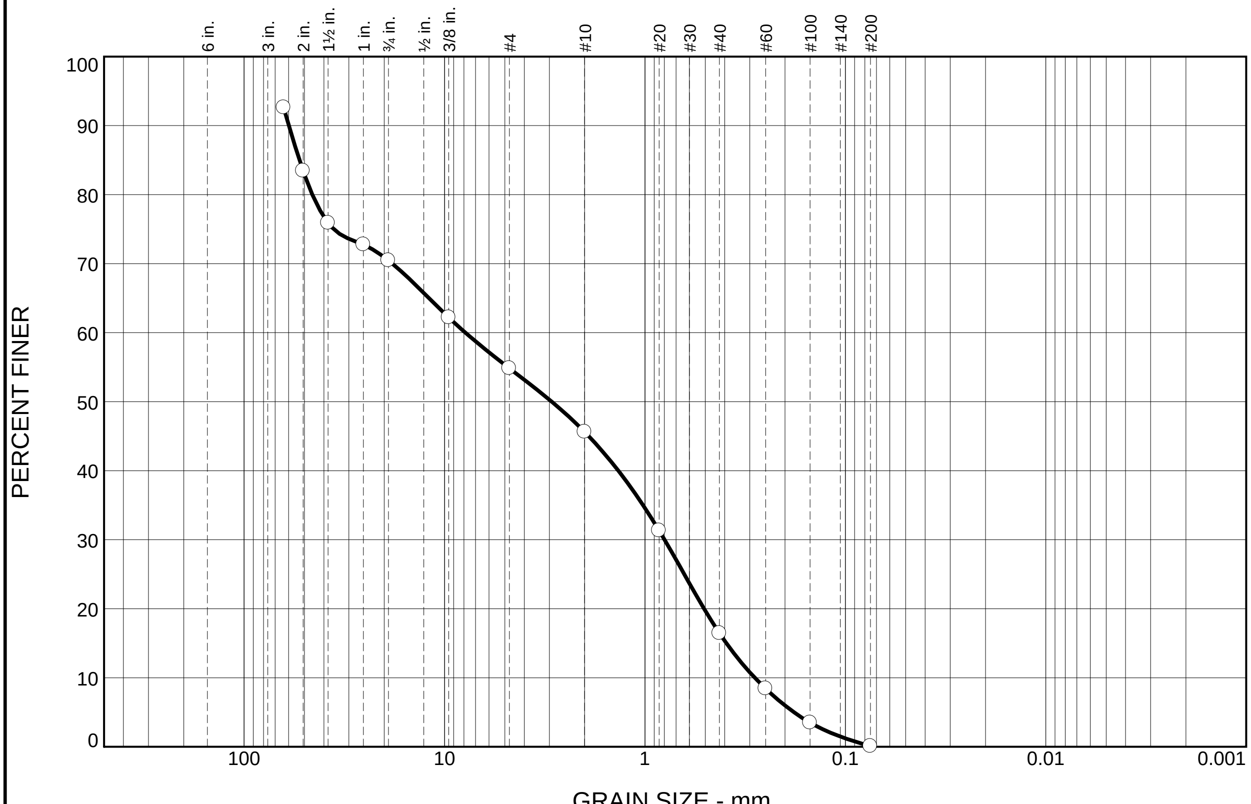
Source of Sample: Soil-Cement
Sample Number: Combined Sample
Date: 4-8-20

PETRA GEOSCIENCES INC.

Client: Q3 Consulting
Project: White water preserve levee
Project No: 19-423
PLATE B-3

Laboratory: 1251 West Pomona Road, Unit #103, Corona, Ca 92882 Phone #. 714.549.8921

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
		15.7	9.2	29.1	16.4	0.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
2.5	92.6		
2	83.5		
1.5	75.9		
1	72.8		
.75	70.5		
.375	62.2		
#4	54.8		
#10	45.6		
#20	31.3		
#40	16.5		
#60	8.4		
#100	3.5		
#200	0.1		

* (no specification provided)

Material Description

Gray, Poorly Graded fine to coarse Sand with Gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 59.7799 D_{85}= 52.9446 D_{60}= 7.8656
D_{50}= 2.9182 D_{30}= 0.7984 D_{15}= 0.3923
D_{10}= 0.2827 C_u= 27.82 C_c= 0.29

Classification

USCS= SP AASHTO=

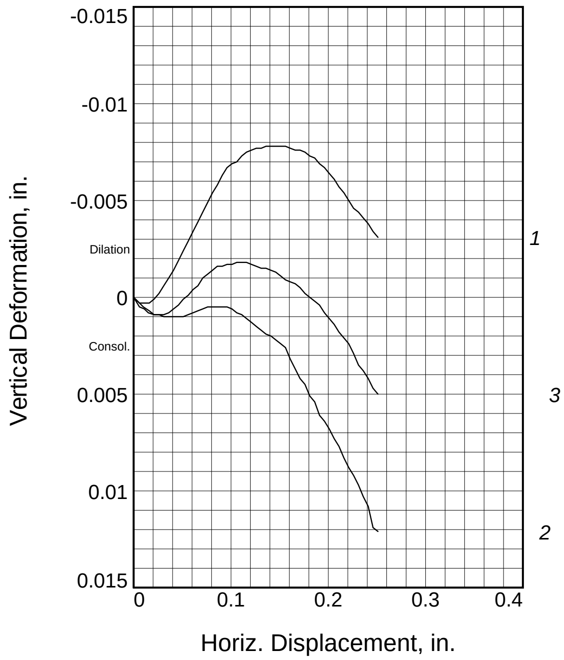
Remarks

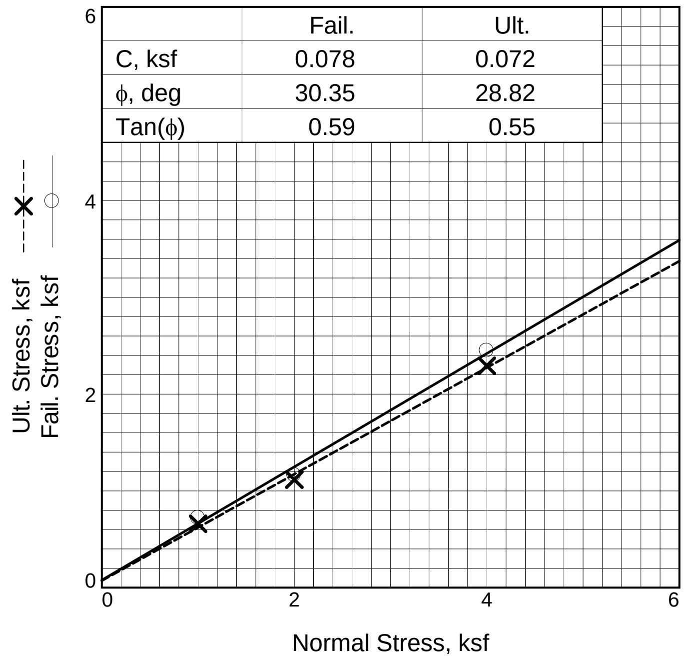
Source of Sample: Sieve data
Sample Number: 4

Date: 5-13-20

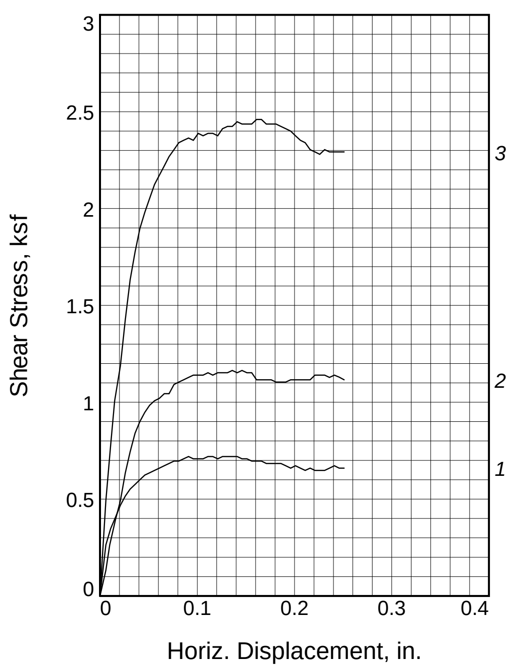
Laboratory:
1251 West Pomona Road, Unit #103, Corona, Ca 92882 Phone #. 714.549.8921

Client: Q3 Consulting
Project: White water preserve levee
Project No: 19-423 **PLATE B-4**





	Fail.	Ult.
C, ksf	0.078	0.072
ϕ, deg	30.35	28.82
Tan(ϕ)	0.59	0.55



	Sample No.	1	2	3
Initial	Water Content, %	10.8	10.8	10.8
	Dry Density, pcf	101.2	101.2	101.2
	Saturation, %	45.1	45.1	45.1
	Void Ratio	0.6353	0.6353	0.6353
	Diameter, in.	2.416	2.416	2.416
	Height, in.	1.000	1.000	1.000
At Test	Water Content, %	23.6	23.5	23.6
	Dry Density, pcf	101.2	101.2	101.2
	Saturation, %	98.5	97.9	98.3
	Void Ratio	0.6353	0.6353	0.6353
	Diameter, in.	2.416	2.416	2.416
	Height, in.	1.000	1.000	1.000
	Normal Stress, ksf	1.000	2.000	4.000
	Fail. Stress, ksf	0.720	1.164	2.448
	Displacement, in.	0.091	0.136	0.141
	Ult. Stress, ksf	0.660	1.116	2.292
	Displacement, in.	0.251	0.251	0.251
	Strain rate, in./min.	0.010	0.010	0.010

Sample Type: Remolded to 85% RC
Description: Gray, Gravelly Sand

Assumed Specific Gravity= 2.65
Remarks: Romold of natural soil

Client: Q3 Consulting

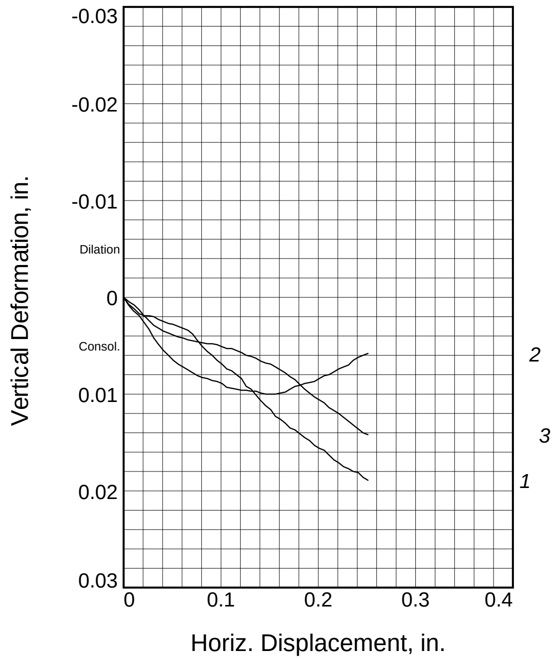
Project: White water preserve levee

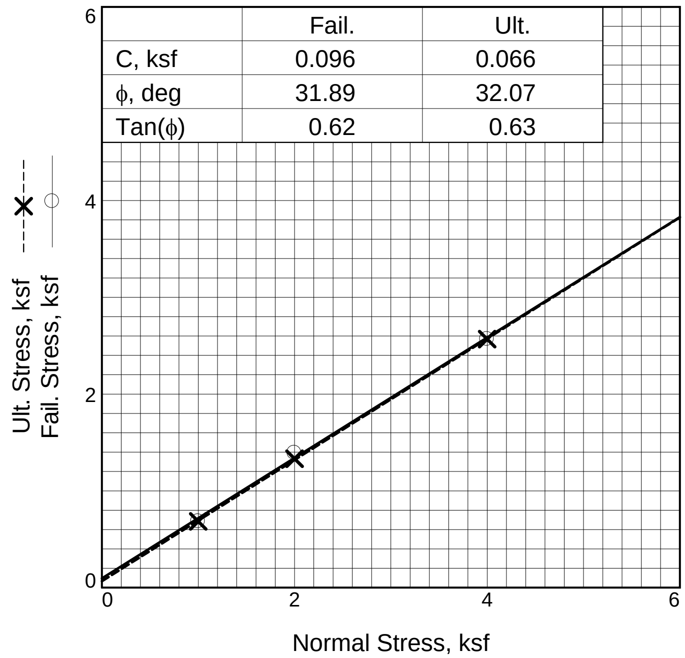
Source of Sample: Combined soil **Depth:** 0-7

Proj. No.: 19-423 **Date Sampled:** 4-1-20

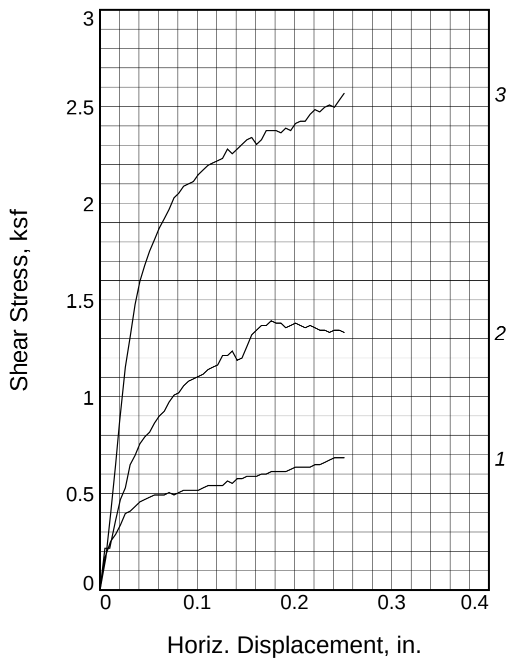
PETRA GEOSCIENCES INC.

PLATE B-5





	Fail.	Ult.
C, ksf	0.096	0.066
ϕ, deg	31.89	32.07
Tan(ϕ)	0.62	0.63



	Sample No.	1	2	3
Initial	Water Content, %	11.1	11.1	11.1
	Dry Density, pcf	100.7	100.7	100.7
	Saturation, %	45.7	45.7	45.7
	Void Ratio	0.6432	0.6432	0.6432
	Diameter, in.	2.416	2.416	2.416
	Height, in.	1.000	1.000	1.000
At Test	Water Content, %	24.1	24.3	24.3
	Dry Density, pcf	100.7	100.7	100.7
	Saturation, %	99.3	100.1	100.3
	Void Ratio	0.6432	0.6432	0.6432
	Diameter, in.	2.416	2.416	2.416
	Height, in.	1.000	1.000	1.000
	Normal Stress, ksf	1.000	2.000	4.000
	Fail. Stress, ksf	0.684	1.392	2.568
	Displacement, in.	0.251	0.176	0.251
	Ult. Stress, ksf	0.684	1.332	2.568
	Displacement, in.	0.251	0.251	0.251
	Strain rate, in./min.	0.010	0.010	0.010

Sample Type: Remolded to 85% relative compaction
Description: Gray, Poorly Graded fine to medium Sand with Gravel

Assumed Specific Gravity= 2.65
Remarks:

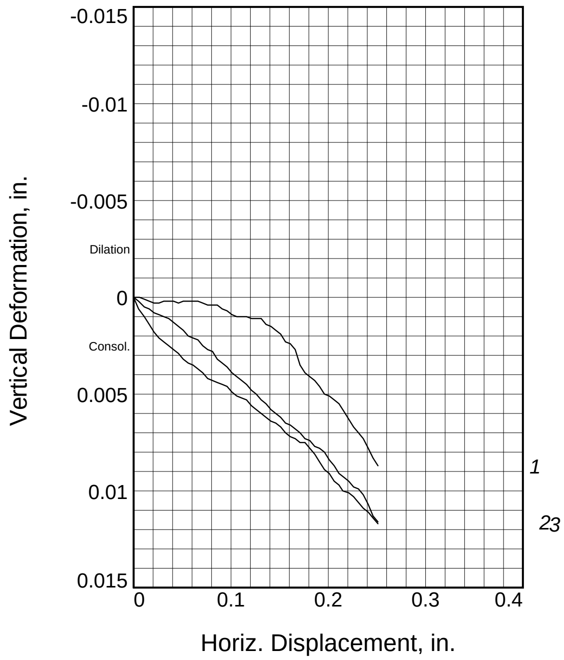
PLATE B-6

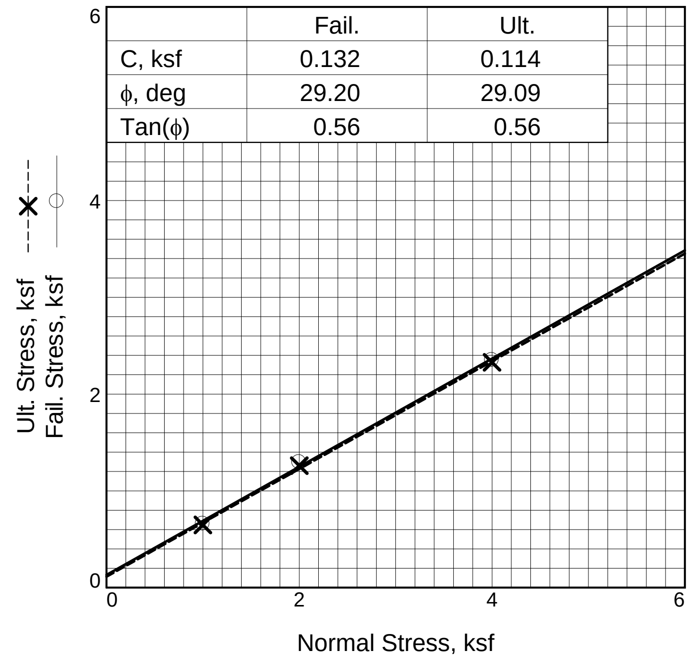
Client: Q3 Consulting

Project: White water preserve levee

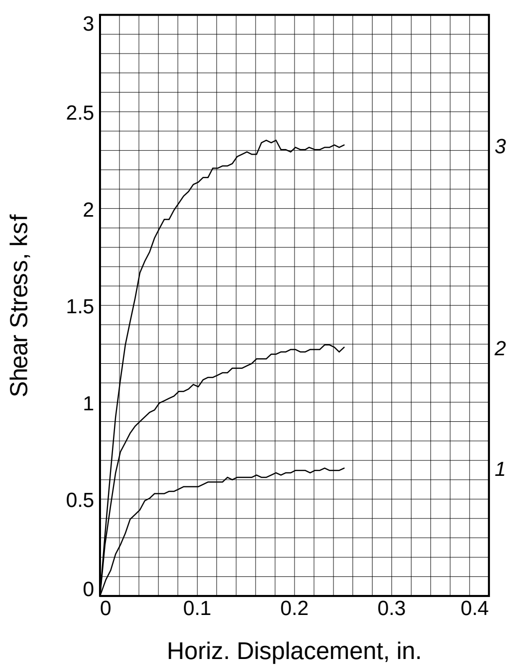
Location: 5-14-20, combined soil
Sample Number: 85% RC
Proj. No.: 19-423 **Date Sampled:** 5-14-20

PETRA GEOSCIENCES INC.





	Fail.	Ult.
C, ksf	0.132	0.114
ϕ, deg	29.20	29.09
Tan(ϕ)	0.56	0.56



	Sample No.	1	2	3
Initial	Water Content, %	10.9	10.9	10.9
	Dry Density, pcf	100.6	99.2	100.5
	Saturation, %	44.7	43.2	44.7
	Void Ratio	0.6450	0.6674	0.6463
	Diameter, in.	2.416	2.416	2.416
	Height, in.	1.000	1.000	1.000
At Test	Water Content, %	24.5	25.3	24.4
	Dry Density, pcf	100.6	99.2	100.5
	Saturation, %	100.5	100.3	100.0
	Void Ratio	0.6450	0.6674	0.6463
	Diameter, in.	2.416	2.416	2.416
	Height, in.	1.000	1.000	1.000
	Normal Stress, ksf	1.000	2.000	4.000
	Fail. Stress, ksf	0.660	1.296	2.352
	Displacement, in.	0.231	0.231	0.171
	Ult. Stress, ksf	0.648	1.260	2.328
	Displacement, in.	0.246	0.246	0.251
	Strain rate, in./min.	0.010	0.010	0.010

Sample Type: Remolded to 85% RC

Description: Gray, Poorly Graded fine to medium Sand with Gravel

Assumed Specific Gravity= 2.65

Remarks:

PLATE B-7

Client: Q3 Consulting

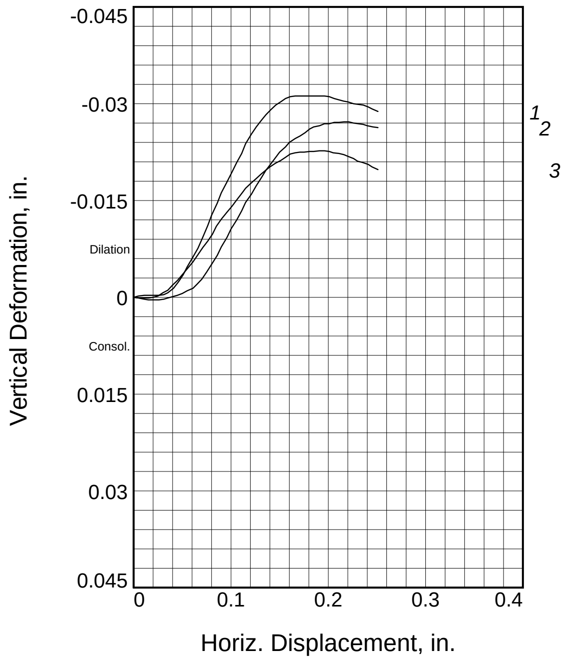
Project: White water preserve levee

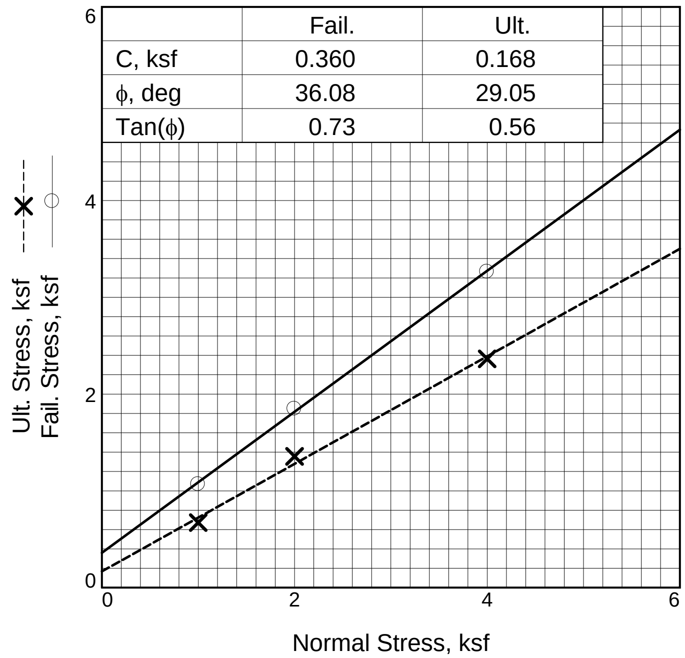
Location: 5-12-20, combined soil

Sample Number: 85% RC (2)

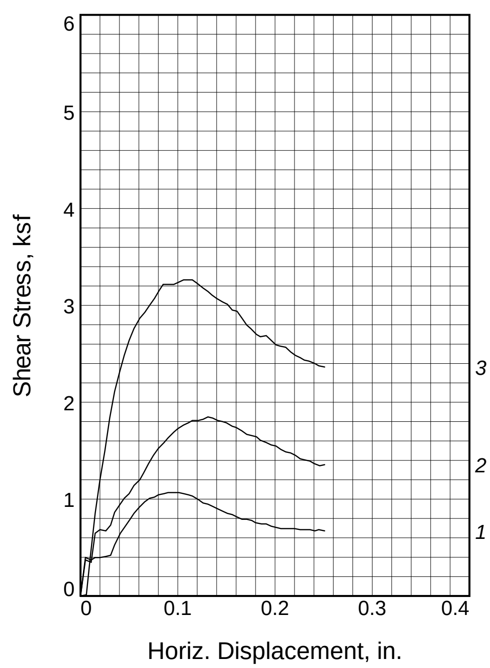
Proj. No.: 19-423 **Date Sampled:** 5-12-20

PETRA GEOSCIENCES INC.





	Fail.	Ult.
C, ksf	0.360	0.168
ϕ, deg	36.08	29.05
Tan(ϕ)	0.73	0.56



	Sample No.	1	2	3
Initial	Water Content, %	10.5	10.5	10.5
	Dry Density, pcf	108.9	108.9	108.9
	Saturation, %	53.5	53.5	53.5
	Void Ratio	0.5191	0.5191	0.5191
	Diameter, in.	2.416	2.416	2.416
	Height, in.	1.000	1.000	1.000
At Test	Water Content, %	19.2	19.5	19.3
	Dry Density, pcf	108.9	108.9	108.9
	Saturation, %	98.2	99.4	98.6
	Void Ratio	0.5191	0.5191	0.5191
	Diameter, in.	2.416	2.416	2.416
	Height, in.	1.000	1.000	1.000
	Normal Stress, ksf	1.000	2.000	4.000
	Fail. Stress, ksf	1.068	1.848	3.264
	Displacement, in.	0.090	0.131	0.106
	Ult. Stress, ksf	0.672	1.356	2.364
	Displacement, in.	0.251	0.251	0.251
	Strain rate, in./min.	0.005	0.005	0.005

Sample Type: Remolded to 90% RC

Description: Gray, Gravelly Sand

Assumed Specific Gravity= 2.65

Remarks: 3 day cure time

PLATE B-8

Client: Q3 Consulting

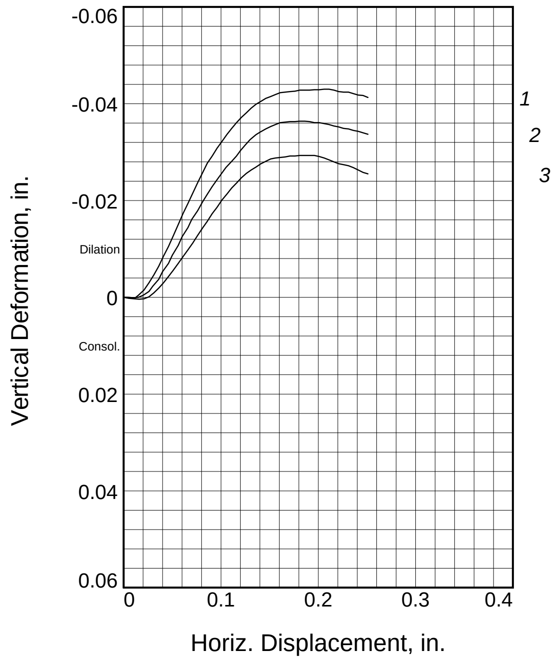
Project: White water preserve levee

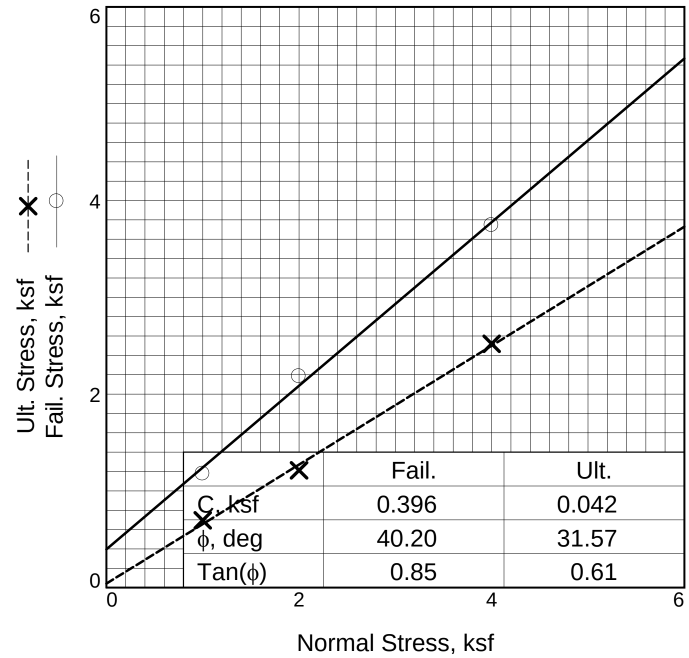
Source of Sample: Combined soil **Depth:** 0-7

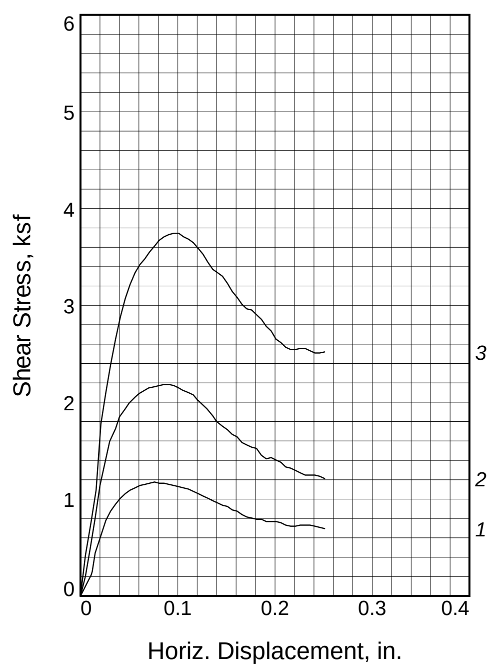
Sample Number: 1% Cement 99% Soil

Proj. No.: 19-423 **Date Sampled:** 4-1-20

PETRA GEOSCIENCES INC.







	Sample No.	1	2	3
Initial	Water Content, %	11.1	11.1	11.1
	Dry Density, pcf	108.4	108.4	108.4
	Saturation, %	55.9	55.9	55.9
	Void Ratio	0.5267	0.5267	0.5267
	Diameter, in.	2.416	2.416	2.416
	Height, in.	1.000	1.000	1.000
At Test	Water Content, %	19.8	20.0	19.9
	Dry Density, pcf	108.4	108.4	108.4
	Saturation, %	99.7	100.4	100.1
	Void Ratio	0.5267	0.5267	0.5267
	Diameter, in.	2.416	2.416	2.416
	Height, in.	1.000	1.000	1.000
	Normal Stress, ksf	1.000	2.000	4.000
	Fail. Stress, ksf	1.176	2.184	3.744
	Displacement, in.	0.076	0.086	0.096
	Ult. Stress, ksf	0.696	1.212	2.520
	Displacement, in.	0.251	0.251	0.251
	Strain rate, in./min.	0.005	0.005	0.005

Sample Type: Remolded to 90% RC
Description: Gray, Gravelly Sand

Assumed Specific Gravity= 2.65
Remarks: 4 day cure time

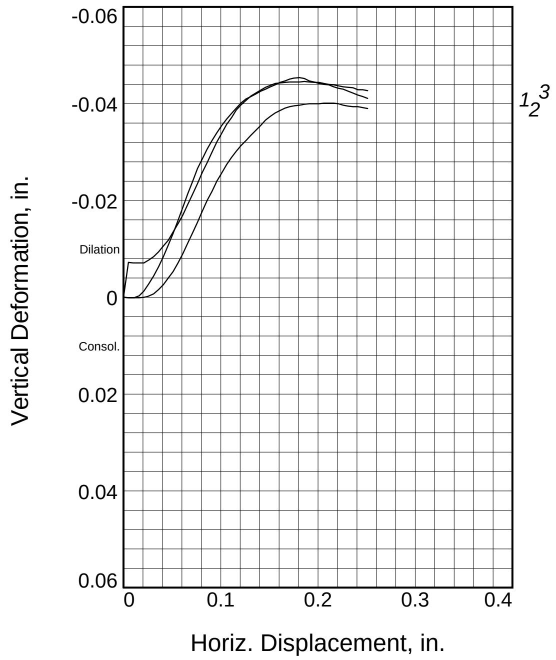
PLATE B-9

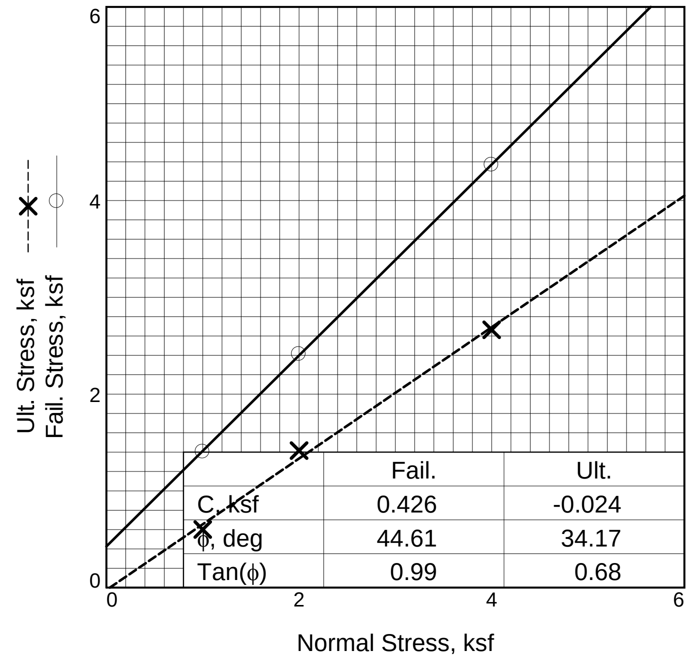
Client: Q3 Consulting

Project: White water preserve levee

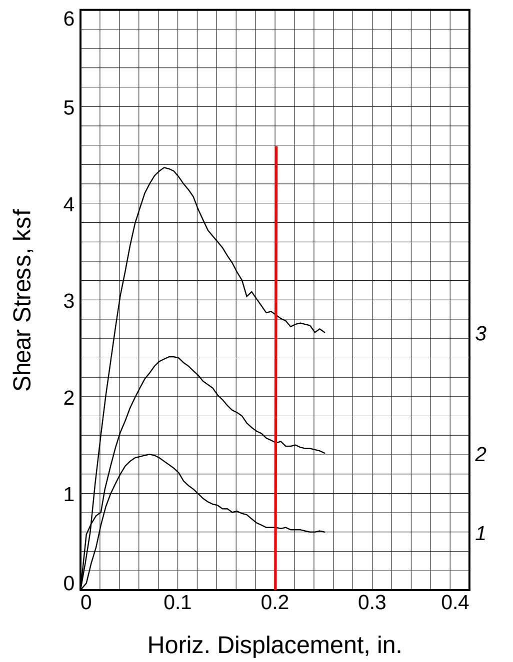
Source of Sample: Combined soil **Depth:** 0-7
Sample Number: 3% Cement 97% Soil
Proj. No.: 19-423 **Date Sampled:** 4-1-20

PETRA GEOSCIENCES INC.





	Fail.	Ult.
C, ksf	0.426	-0.024
φ, deg	44.61	34.17
Tan(φ)	0.99	0.68



	Sample No.	1	2	3
Initial	Water Content, %	10.9	10.9	10.9
	Dry Density, pcf	108.1	108.1	108.1
	Saturation, %	54.5	54.5	54.5
	Void Ratio	0.5300	0.5300	0.5300
	Diameter, in.	2.416	2.416	2.416
	Height, in.	1.000	1.000	1.000
At Test	Water Content, %	19.8	19.7	15.4
	Dry Density, pcf	108.1	108.1	108.1
	Saturation, %	98.9	98.7	77.2
	Void Ratio	0.5300	0.5300	0.5300
	Diameter, in.	2.416	2.416	2.416
	Height, in.	1.000	1.000	1.000
	Normal Stress, ksf	1.000	2.000	4.000
	Fail. Stress, ksf	1.404	2.412	4.368
	Displacement, in.	0.071	0.091	0.086
	Ult. Stress, ksf	0.600	1.416	2.664
	Displacement, in.	0.251	0.251	0.251
	Strain rate, in./min.	0.005	0.005	0.005

Sample Type: Remolded to 90% RC

Description: Gray, Gravelly Sand

Assumed Specific Gravity= 2.65

Remarks: 3 day cure time

PLATE B-10

Client: Q3 Consulting

Project: White water preserve levee

Source of Sample: Combined soil **Depth:** 0-7

Sample Number: 5% Cement 95% Soil

Proj. No.: 19-423 **Date Sampled:** 4-1-20

PETRA GEOSCIENCES INC.

APPENDIX C

SEISMIC DESIGN PARAMETERS

DRAFT

U.S. Geological Survey - Earthquake Hazards Program

Unified Hazard Tool

Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the U.S. Seismic Design Maps web tools (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

Input

Edition

Dynamic: Conterminous U.S. 2014 (u…

Latitude

Decimal degrees

33.988352

Longitude

Decimal degrees, negative values for western longitudes

-116.657013

Site Class

259 m/s (Site class D)

Spectral Period

Peak Ground Acceleration

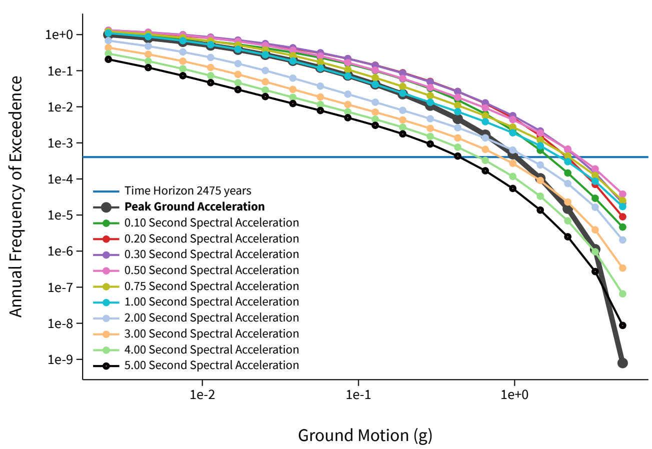
Time Horizon

Return period in years

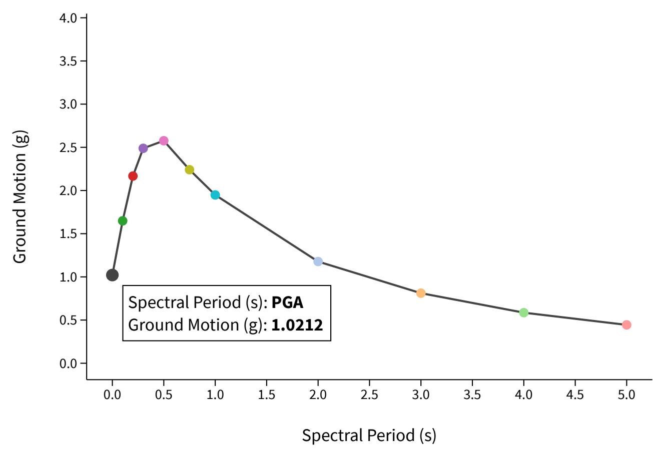
2475

Hazard Curve

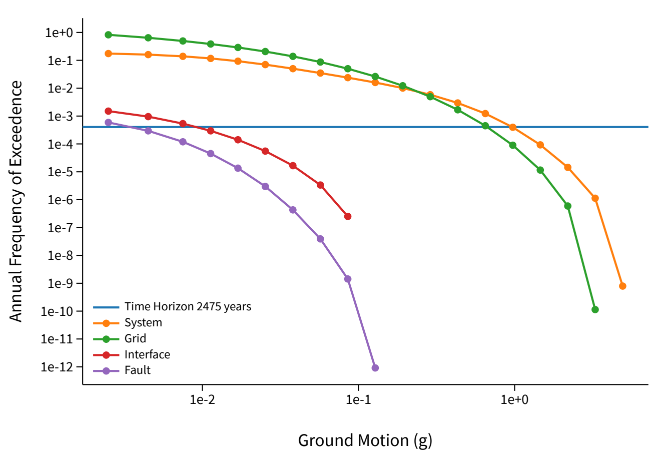
Hazard Curves



Uniform Hazard Response Spectrum



Component Curves for Peak Ground Acceleration

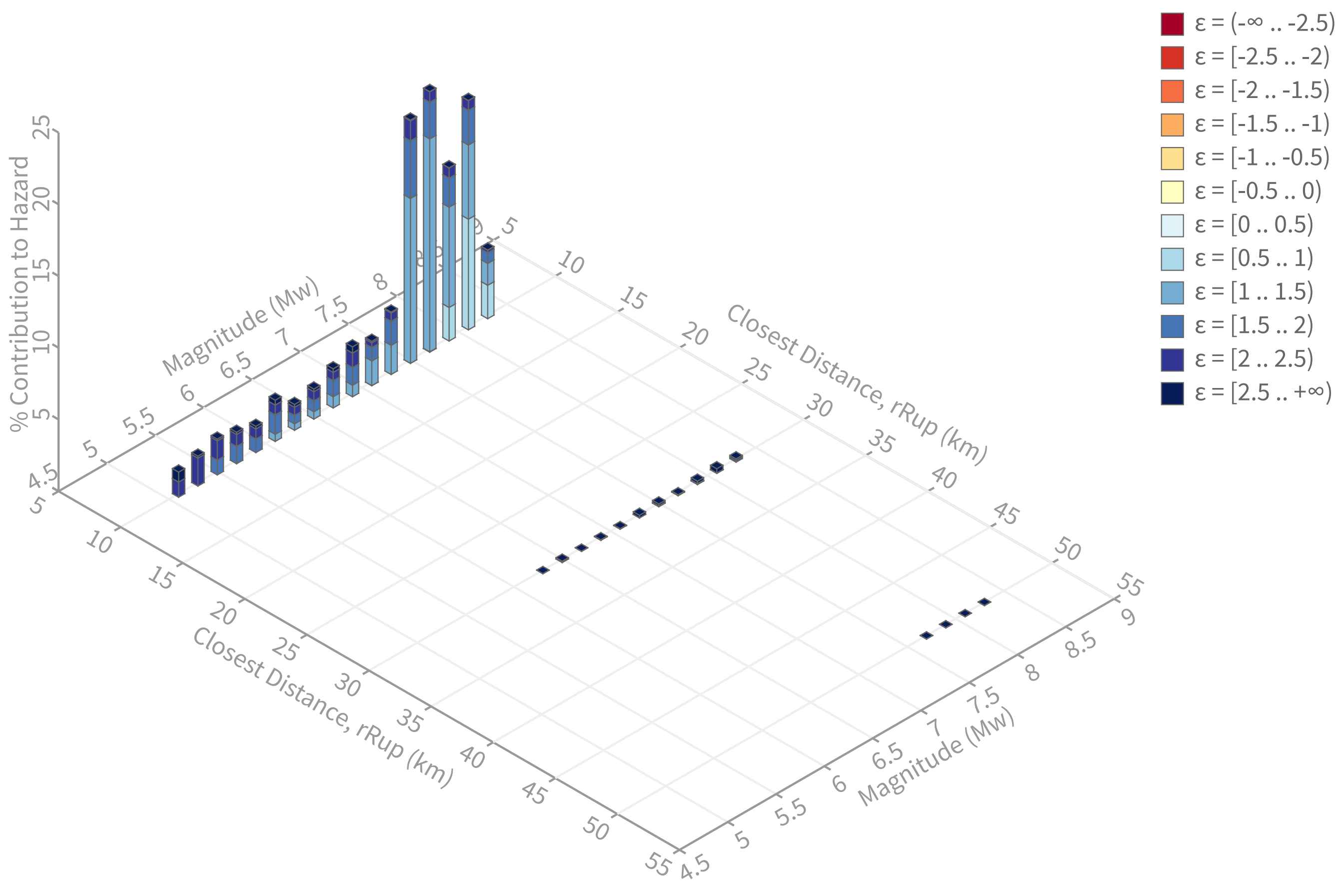


[View Raw Data]

Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 1.0212208 g

Recovered targets

Return period: 3178.1746 yrs
Exceedance rate: 0.00031464602 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.04 %

Mean (over all sources)

m: 7.37
r: 6.18 km
ε_0: 1.48 σ

Mode (largest m-r bin)

m: 7.68
r: 4.91 km
ε_0: 1.25 σ
Contribution: 18.15 %

Mode (largest m-r-ε_0 bin)

m: 7.68
r: 4.9 km
ε_0: 1.14 σ
Contribution: 14.84 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
UC33brAvg_FM32	System							41.94
San Andreas (San Gorgonio Pass-Garnet HIll) [6]		4.78	7.74	1.25	116.677°W	33.948°N	202.07	29.52
San Andreas (North Branch Mill Creek) [7]		5.38	7.89	1.25	116.630°W	34.018°N	36.63	4.33
Mission Creek [0]		5.40	7.26	1.60	116.642°W	34.032°N	16.27	2.09
San Andreas (San Gorgonio Pass-Garnet HIll) [5]		5.30	7.17	1.39	116.657°W	33.940°N	180.38	1.54
Pinto Mtn [0]		9.12	7.13	1.99	116.701°W	34.060°N	333.20	1.13
UC33brAvg_FM31	System							41.92
San Andreas (San Gorgonio Pass-Garnet HIll) [6]		4.78	7.74	1.25	116.677°W	33.948°N	202.07	29.60
San Andreas (North Branch Mill Creek) [7]		5.38	7.91	1.24	116.630°W	34.018°N	36.63	4.28
Mission Creek [0]		5.40	7.25	1.61	116.642°W	34.032°N	16.27	2.45
San Andreas (San Gorgonio Pass-Garnet HIll) [5]		5.30	7.11	1.40	116.657°W	33.940°N	180.38	1.33
UC33brAvg_FM31 (opt)	Grid							8.07
PointSourceFinite: -116.657, 34.029		6.82	5.65	2.04	116.657°W	34.029°N	0.00	2.03
PointSourceFinite: -116.657, 34.029		6.82	5.65	2.04	116.657°W	34.029°N	0.00	2.03
PointSourceFinite: -116.657, 34.038		7.39	5.69	2.11	116.657°W	34.038°N	0.00	1.26
PointSourceFinite: -116.657, 34.038		7.39	5.69	2.11	116.657°W	34.038°N	0.00	1.26
UC33brAvg_FM32 (opt)	Grid							8.07
PointSourceFinite: -116.657, 34.029		6.82	5.65	2.04	116.657°W	34.029°N	0.00	2.03
PointSourceFinite: -116.657, 34.029		6.82	5.65	2.04	116.657°W	34.029°N	0.00	2.03
PointSourceFinite: -116.657, 34.038		7.39	5.69	2.11	116.657°W	34.038°N	0.00	1.26
PointSourceFinite: -116.657, 34.038		7.39	5.69	2.11	116.657°W	34.038°N	0.00	1.26

Whitewater Preserve

Latitude, Longitude: 33.988352, -116.657013

Date	6/3/2020, 12:00:13 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Default (See Section 11.4.3)

Type	Value	Description
S_S	2.403	MCE_R ground motion. (for 0.2 second period)
S_1	0.901	MCE_R ground motion. (for 1.0s period)
S_{MS}	2.884	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.923	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1.2	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.98	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	1.176	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	2.431	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	2.703	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.403	Factored deterministic acceleration value. (0.2 second)
S1RT	0.97	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	1.1	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.901	Factored deterministic acceleration value. (1.0 second)
PGAd	0.98	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.899	Mapped value of the risk coefficient at short periods
C_{R1}	0.883	Mapped value of the risk coefficient at a period of 1 s

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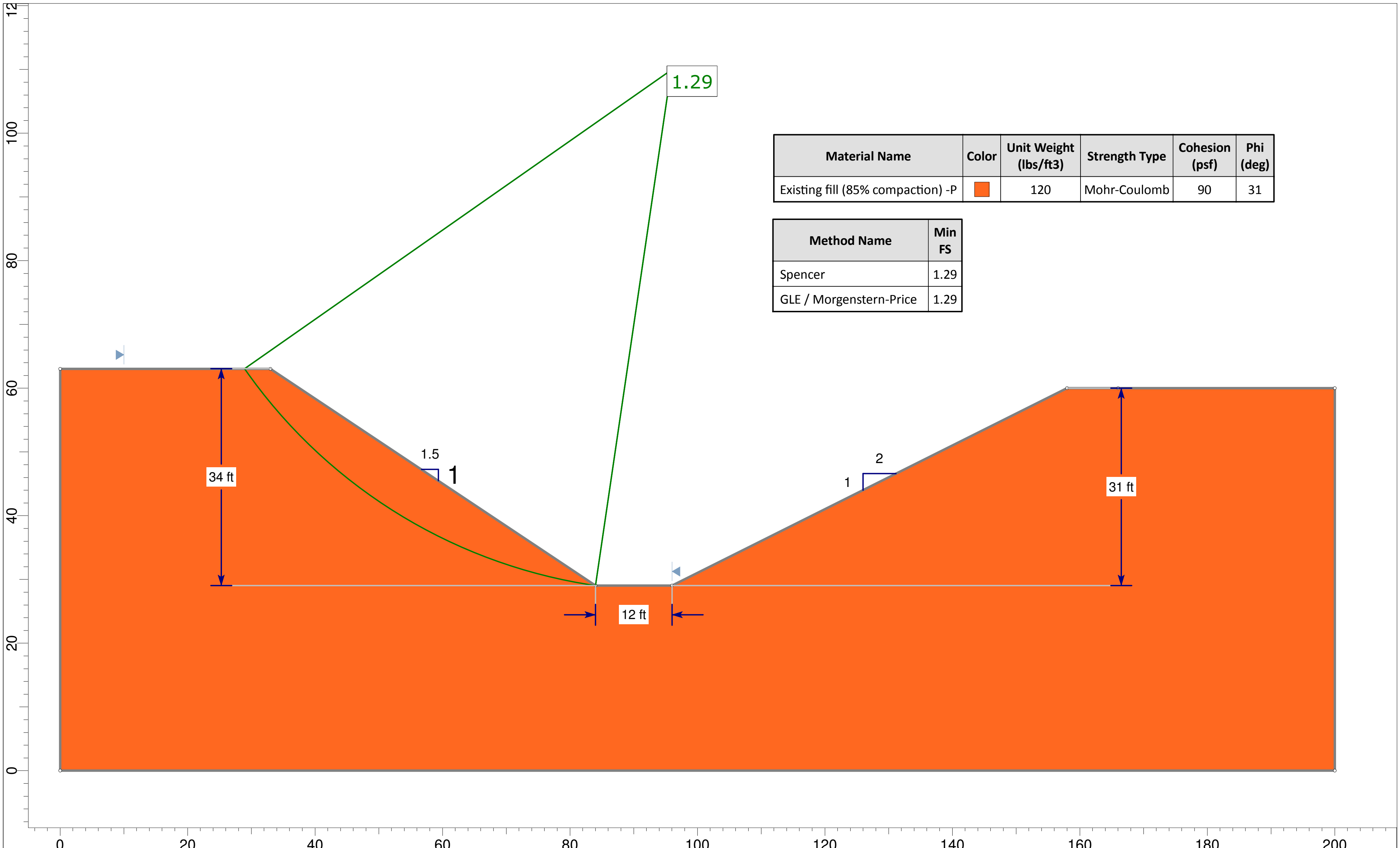
APPENDIX D

SLOPE STABILITY ANALYSIS

DRAFT



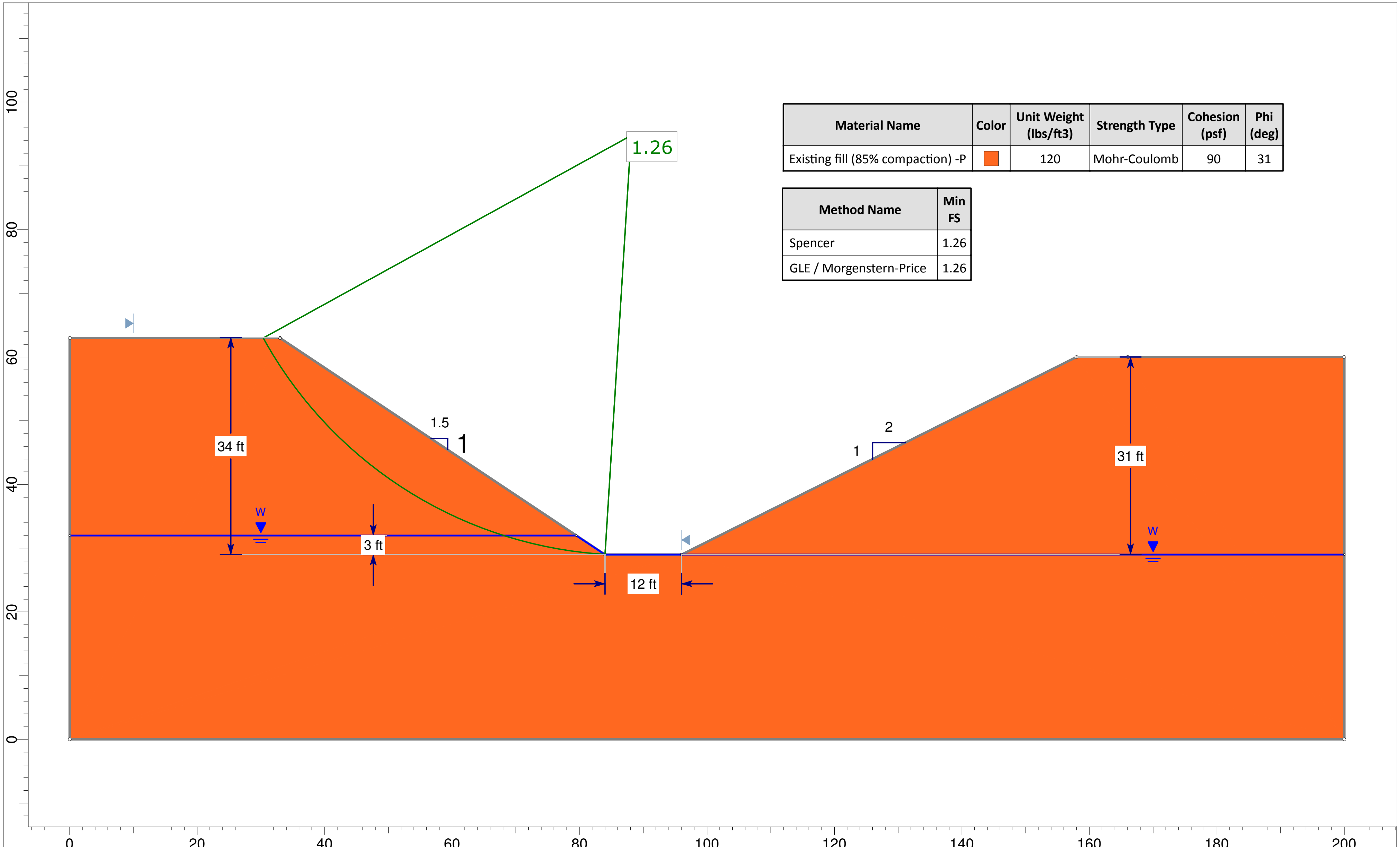
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Existing fill (85% compaction) -P		120	Mohr-Coulomb	90	31

Method Name	Min FS
Spencer	1.29
GLE / Morgenstern-Price	1.29

PETRA GEOSCIENCES INC

Project	Whitewater Preserve Flood Control		
Analysis Description	Stability of False Slope during Construction – Highest Slope (Sta.26+00), Dry Condition, Static		
Drawn By AM	*Scale* 1:250	*Company*	Petra Geosciences, Inc.
Date 6/16/2020		*File Name*	False Slope.slmd

SLIDEINTERPRET 8.032



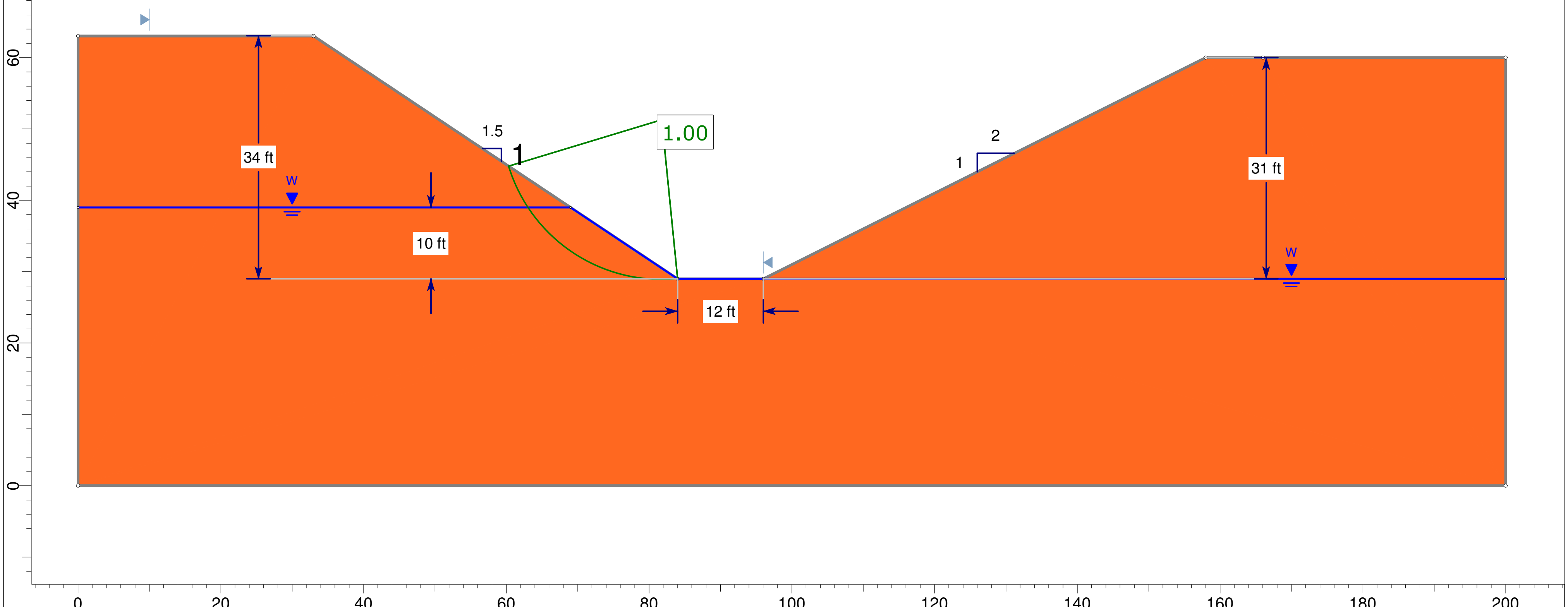
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Existing fill (85% compaction) -P		120	Mohr-Coulomb	90	31

Method Name	Min FS
Spencer	1.26
GLE / Morgenstern-Price	1.26

PETRA GEOSCIENCES INC	*Project* Whitewater Preserve Flood Control		
	Analysis Description Stability of False Slope during Construction Highest Slope (Sta.26+00), Groundwater Seepage @ 3' above the bottom of Valley, Static, Partially Stable Condition		
	Drawn By AM	*Scale* 1:250	*Company* Petra Geosciences, Inc.
SLIDEINTERPRET 8.032	*Date* 6/16/2020		*File Name* False Slope.slmd

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Existing fill (85% compaction) -P		120	Mohr-Coulomb	90	31

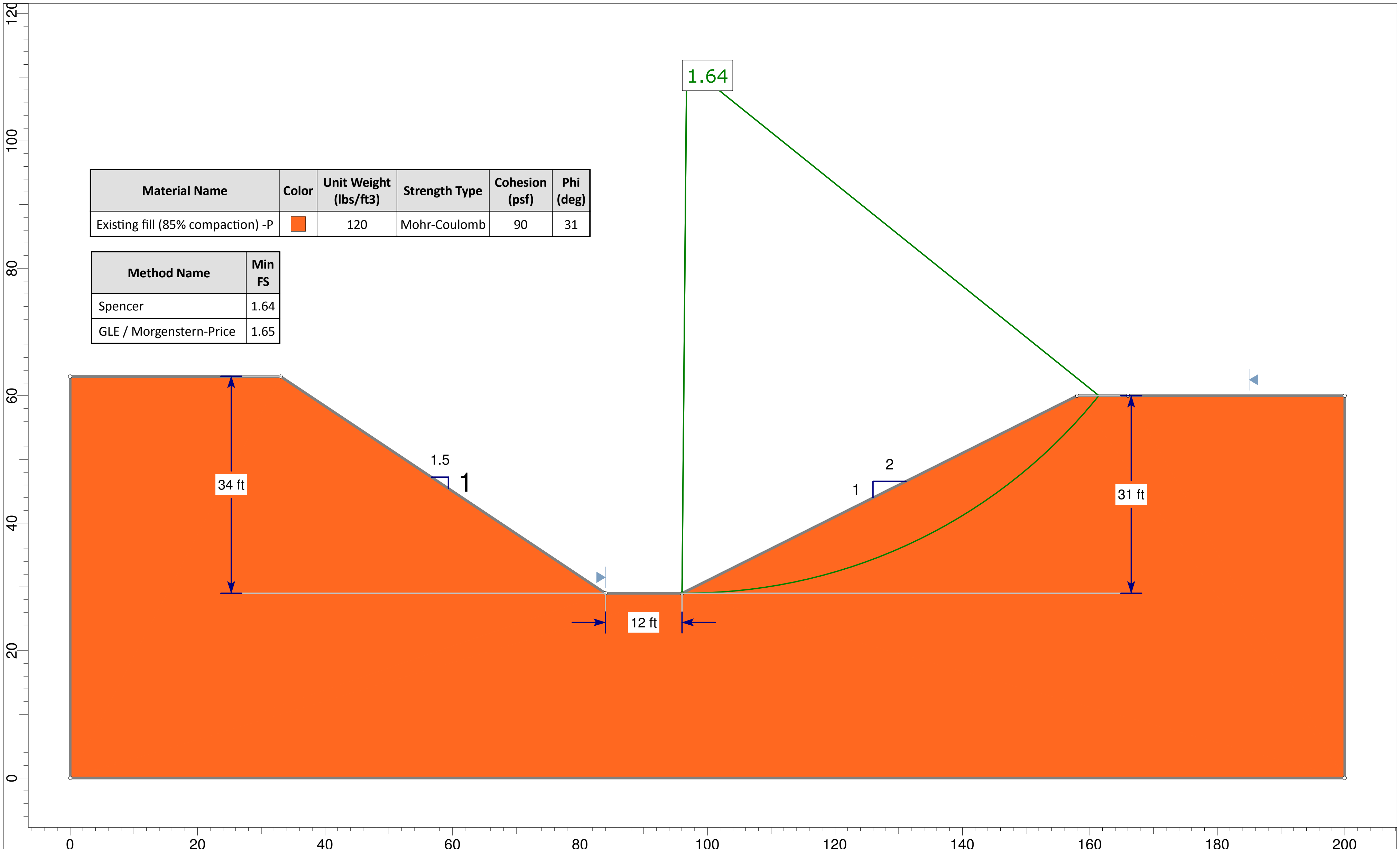
Method Name	Min FS
Spencer	1.00
GLE / Morgenstern-Price	1.00



PETRA GEOSCIENCES INC	Project	Whitewater Preserve Flood Control	
	Analysis Description	Stability of False Slope during Construction Highest Slope (Sta.26+00), Groundwater Seepage @ 10' above the bottom of Valley, Static, Unstable Condition	
	Drawn By: AM	Scale: 1:250	Company: Petra Geosciences, Inc.
SLIDEINTERPRET 8.032	Date: 6/16/2020		File Name: False Slope.slmd

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Existing fill (85% compaction) -P		120	Mohr-Coulomb	90	31

Method Name	Min FS
Spencer	1.64
GLE / Morgenstern-Price	1.65



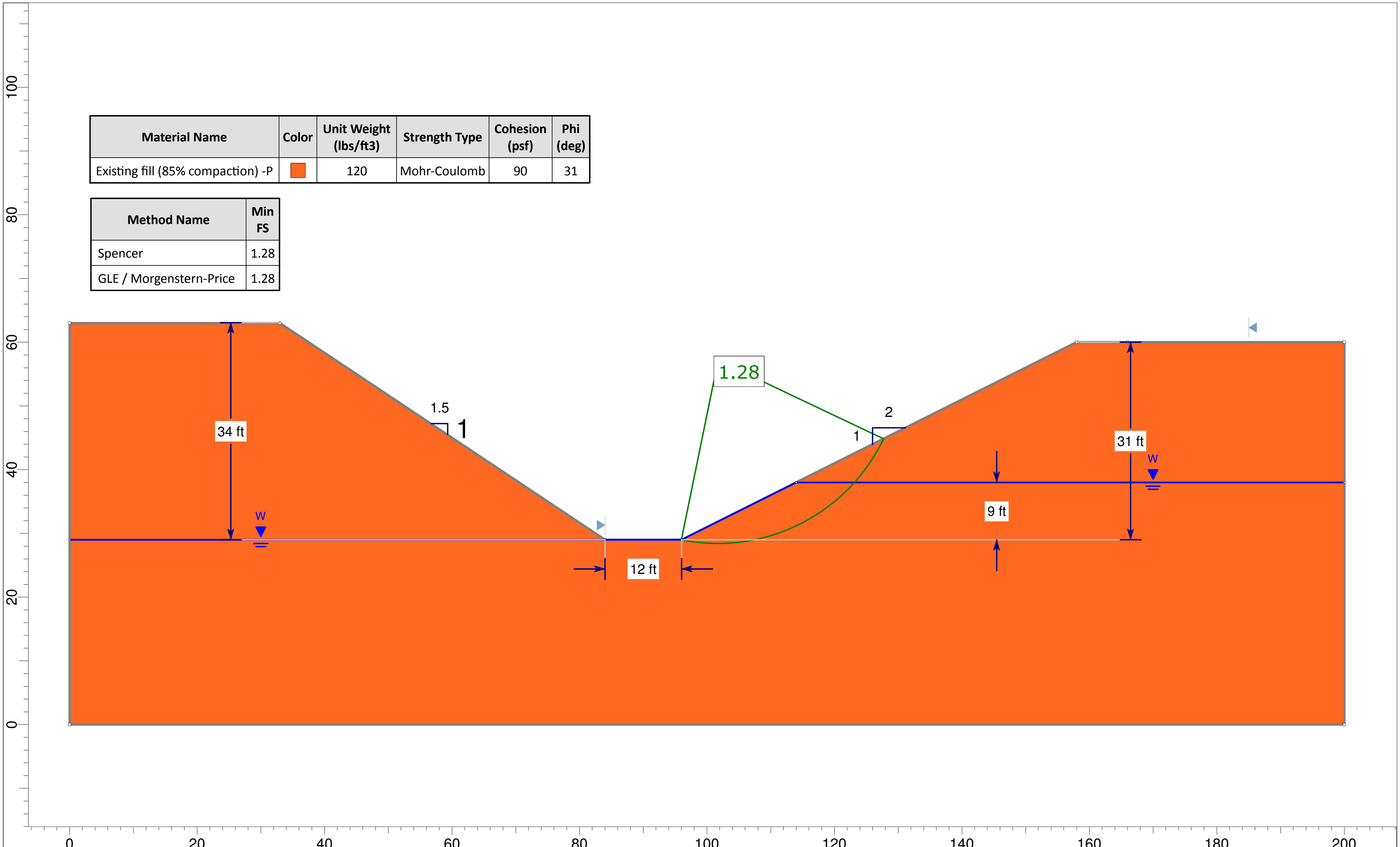
Project	Whitewater Preserve Flood Control		
Analysis Description	Stability of Levee Slope during Construction – Highest Slope (Sta.26+00), Dry Condition, Static		
Drawn By AM	*Scale* 1:250	*Company*	Petra Geosciences, Inc.
Date 6/16/2020		*File Name*	False Slope.slmd

PETRA GEOSCIENCES INC

SLIDEINTERPRET 8.032

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Existing fill (85% compaction) -P		120	Mohr-Coulomb	90	31

Method Name	Min FS
Spencer	1.28
GLE / Morgenstern-Price	1.28



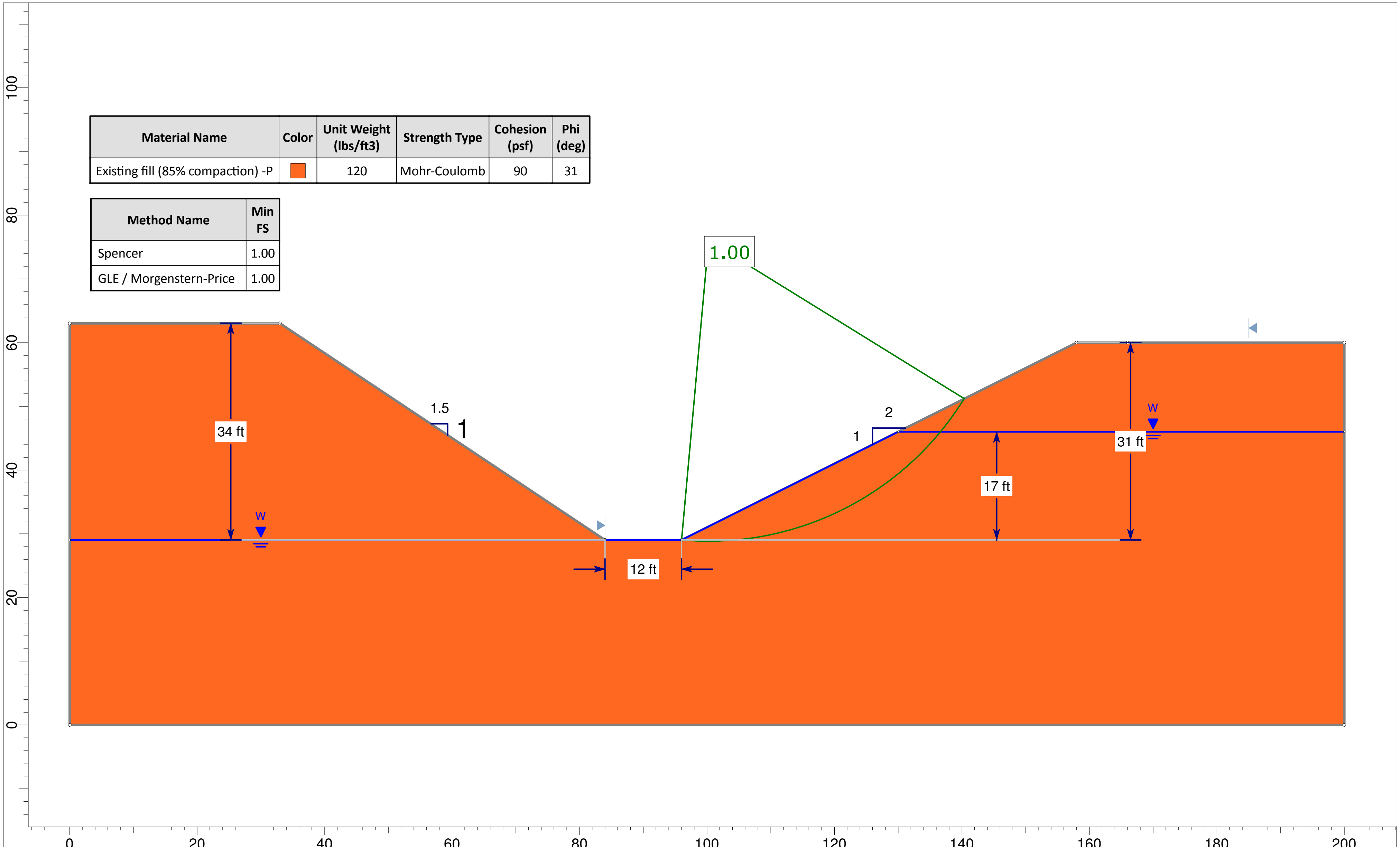
PETRA GEOSCIENCES INC.

SLIDEINTERPRET 8.032

Project	Whitewater Preserve Flood Control
Analysis Description	Stability of Levee Slope during Construction Highest Slope (Sta.26+00), Groundwater Seepage @ 9' above the bottom of Valley, Static, Partially Stable Condition
Drawn By	AM
Scale	1:250
Company	Petra Geosciences, Inc.
Date	6/16/2020
File Name	Slope Stability.slmd

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Existing fill (85% compaction) -P		120	Mohr-Coulomb	90	31

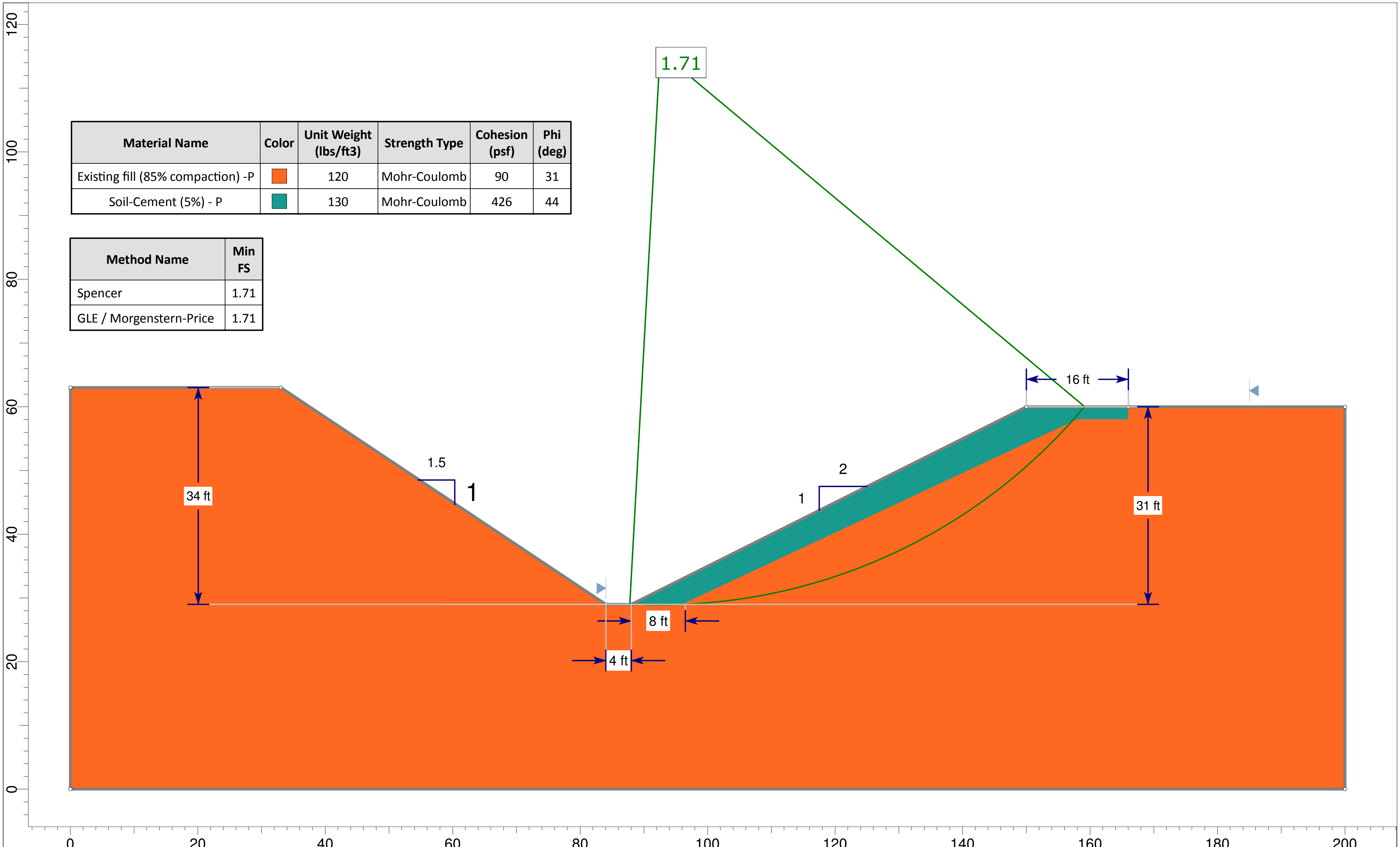
Method Name	Min FS
Spencer	1.00
GLE / Morgenstern-Price	1.00



Project	Whitewater Preserve Flood Control
Analysis Description	Stability of Levee Slope during Construction Highest Slope (Sta.26+00), Groundwater Seepage @ 17’ above the bottom of Valley, Static, Unstable Condition
Drawn By	AM
Scale	1:250
Company	Petra Geosciences, Inc.
Date	6/16/2020
File Name	Slope Stability.slmd

PETRA GEOSCIENCES INC.

SLIDEINTERPRET 8.032



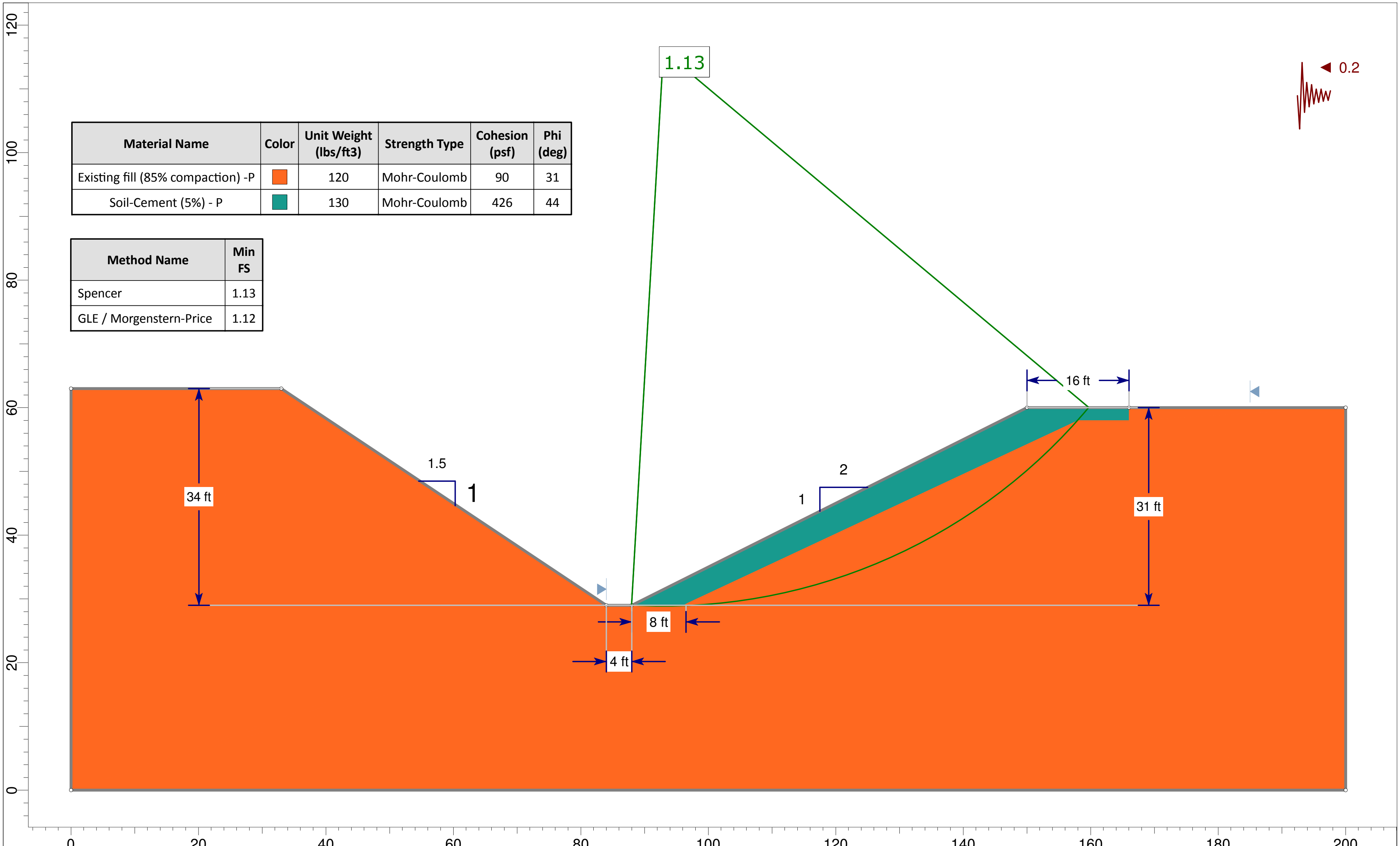
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Existing fill (85% compaction) -P		120	Mohr-Coulomb	90	31
Soil-Cement (5%) - P		130	Mohr-Coulomb	426	44

Method Name	Min FS
Spencer	1.71
GLE / Morgenstern-Price	1.71

PETRA GEOSCIENCES INC

SLIDEINTERPRET 8.032

Project	Whitewater Preserve Flood Control		
Analysis Description	Stability of Levee Slope Immediately after Construction – Highest Slope (Sta.26+00), Dry Condition, Static		
Drawn By AM	*Scale* 1:250	*Company*	Petra Geosciences, Inc.
Date	6/15/2020	*File Name*	Slope Stability.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Existing fill (85% compaction) -P		120	Mohr-Coulomb	90	31
Soil-Cement (5%) - P		130	Mohr-Coulomb	426	44

Method Name	Min FS
Spencer	1.13
GLE / Morgenstern-Price	1.12

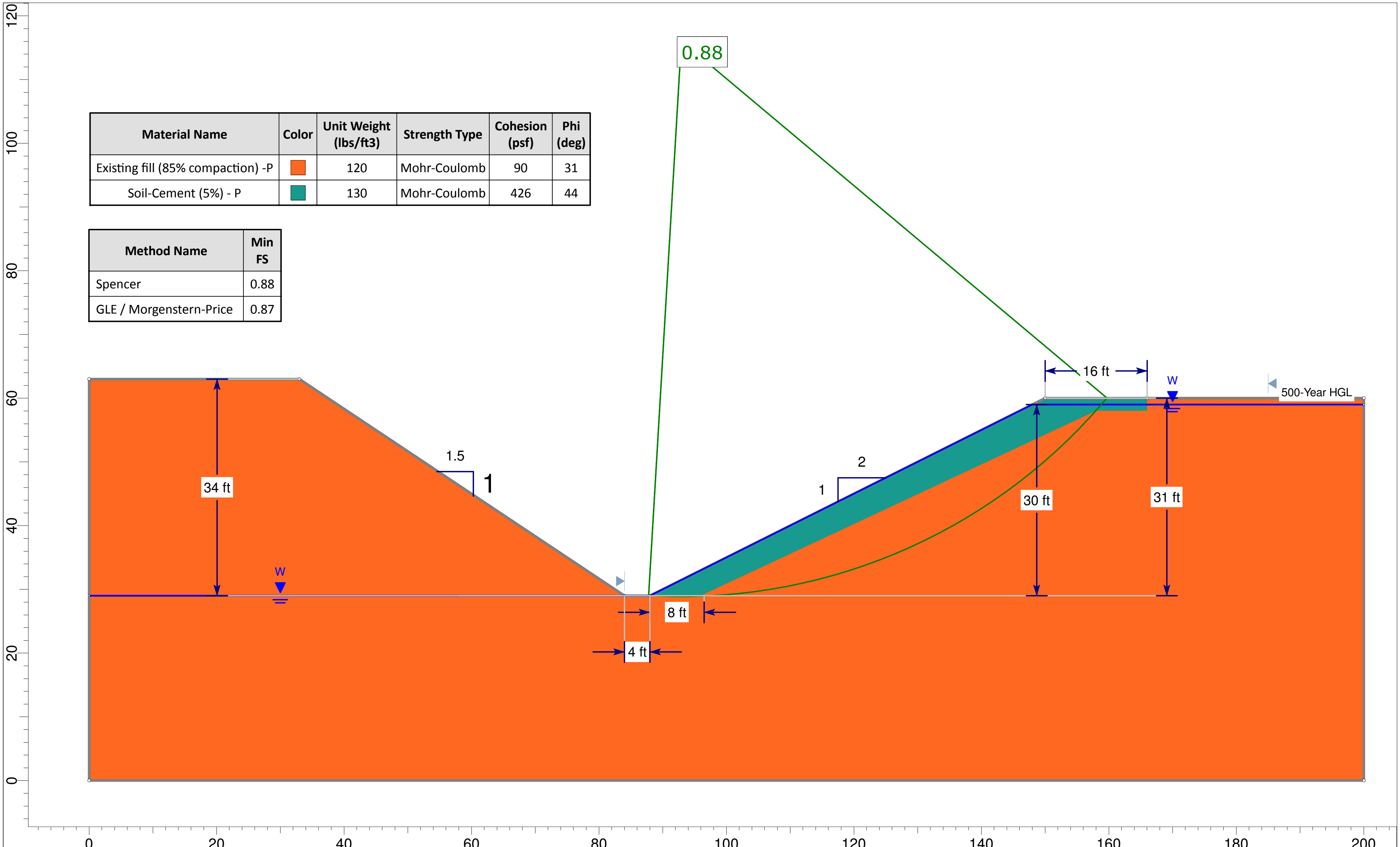
PETRA GEOSCIENCES INC

Project	Whitewater Preserve Flood Control
Analysis Description	Stability of Levee Slope Immediately after Construction – Highest Slope (Sta.26+00), Dry Condition, Pseudo-Static
Drawn By	AM
Scale	1:250
Company	Petra Geosciences, Inc.
Date	6/15/2020
File Name	Slope Stability.slmd

SLIDEINTERPRET 8.032

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Existing fill (85% compaction) -P		120	Mohr-Coulomb	90	31
Soil-Cement (5%) - P		130	Mohr-Coulomb	426	44

Method Name	Min FS
Spencer	0.88
GLE / Morgenstern-Price	0.87

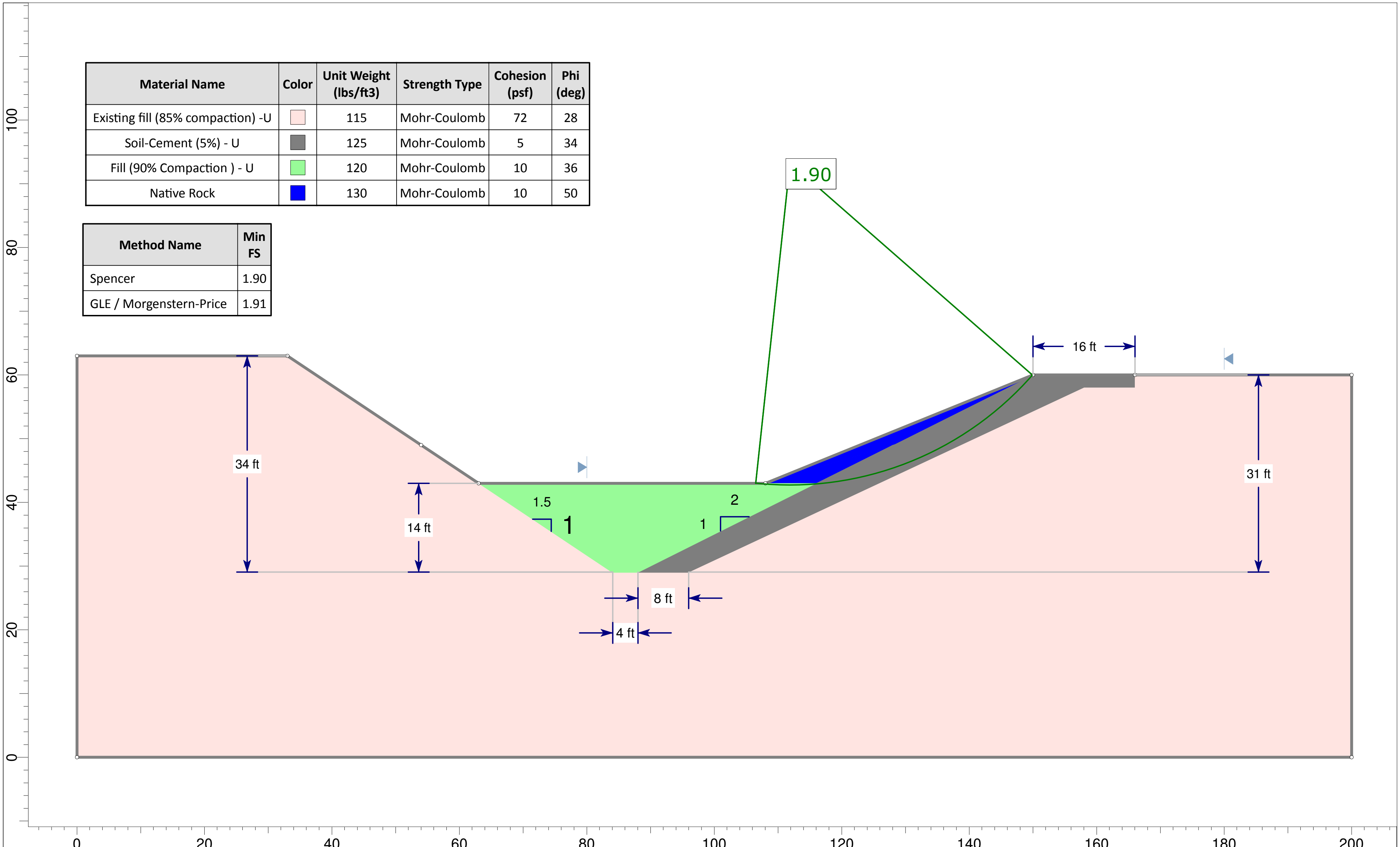


SLIDEINTERPRET 8.032

Project	Whitewater Preserve Flood Control
Analysis Description	Stability of Levee Slope Immediately after Construction Highest Slope (Sta.26+00), Seepage @ 28' above the bottom of Valley, Rapid Drawdown
Drawn By	AM
Scale	1:250
Company	Petra Geosciences, Inc.
Date	6/16/2020
File Name	Slope Stability.slmd

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Existing fill (85% compaction) -U		115	Mohr-Coulomb	72	28
Soil-Cement (5%) - U		125	Mohr-Coulomb	5	34
Fill (90% Compaction) - U		120	Mohr-Coulomb	10	36
Native Rock		130	Mohr-Coulomb	10	50

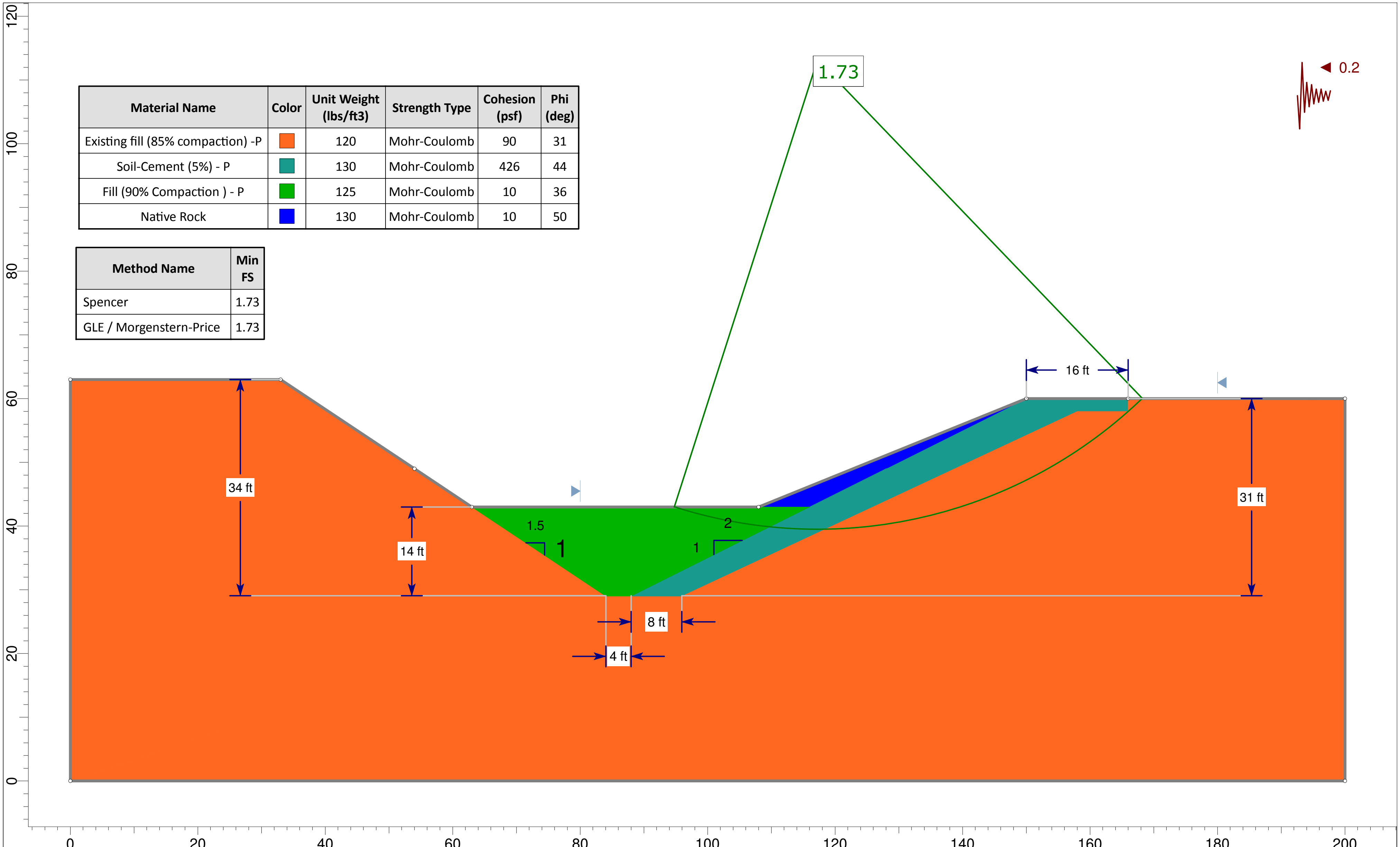
Method Name	Min FS
Spencer	1.90
GLE / Morgenstern-Price	1.91



Project	Whitewater Preserve Flood Control		
Analysis Description	Stability of Levee Slope during Operation – Highest Slope (Sta.18+00), Dry Condition, Static		
Drawn By AM	*Scale* 1:250	*Company*	Petra Geosciences, Inc.
Date 6/16/2020		*File Name*	Slope Stability.slmd

PETRA GEOSCIENCES INC

SLIDEINTERPRET 8.032



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Existing fill (85% compaction) -P		120	Mohr-Coulomb	90	31
Soil-Cement (5%) - P		130	Mohr-Coulomb	426	44
Fill (90% Compaction) - P		125	Mohr-Coulomb	10	36
Native Rock		130	Mohr-Coulomb	10	50

Method Name	Min FS
Spencer	1.73
GLE / Morgenstern-Price	1.73

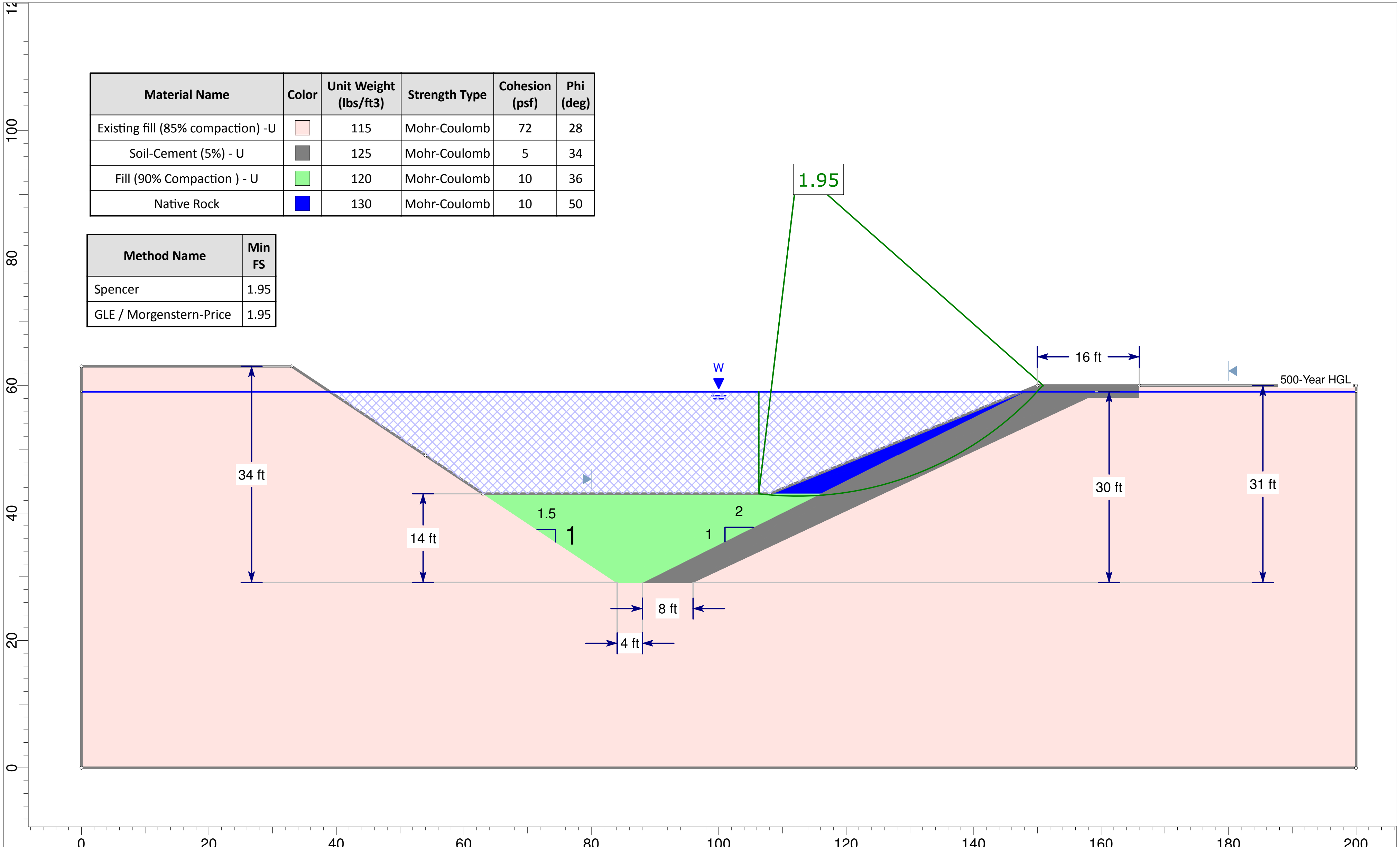
PETRA GEOSCIENCES INC

SLIDEINTERPRET 8.032

Project	Whitewater Preserve Flood Control
Analysis Description	Stability of Levee Slope during Operation – Highest Slope (Sta.18+00), Dry Condition, Pseudo-Static
Drawn By	AM
Scale	1:250
Company	Petra Geosciences, Inc.
Date	6/16/2020
File Name	Slope Stability.slmd

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Existing fill (85% compaction) -U		115	Mohr-Coulomb	72	28
Soil-Cement (5%) - U		125	Mohr-Coulomb	5	34
Fill (90% Compaction) - U		120	Mohr-Coulomb	10	36
Native Rock		130	Mohr-Coulomb	10	50

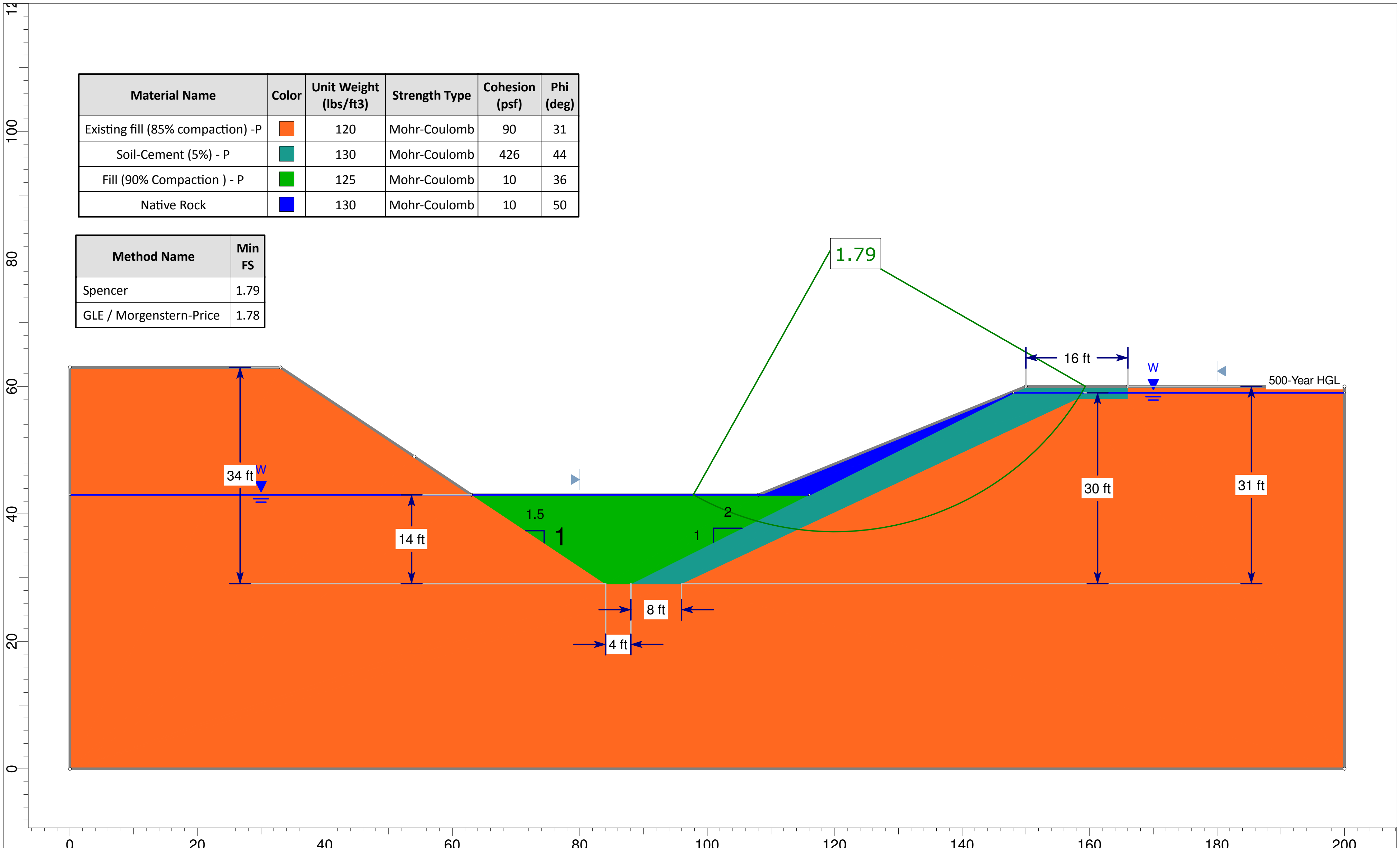
Method Name	Min FS
Spencer	1.95
GLE / Morgenstern-Price	1.95



PETRA GEOSCIENCES INC

SLIDEINTERPRET 8.032

Project	Whitewater Preserve Flood Control
Analysis Description	Stability of Levee Slope during Operation – Highest Slope (Sta.18+00), Water @ 28’ above the bottom of Valley, Static
Drawn By	AM
Scale	1:250
Company	Petra Geosciences, Inc.
Date	6/15/2020
File Name	Slope Stability.slmd



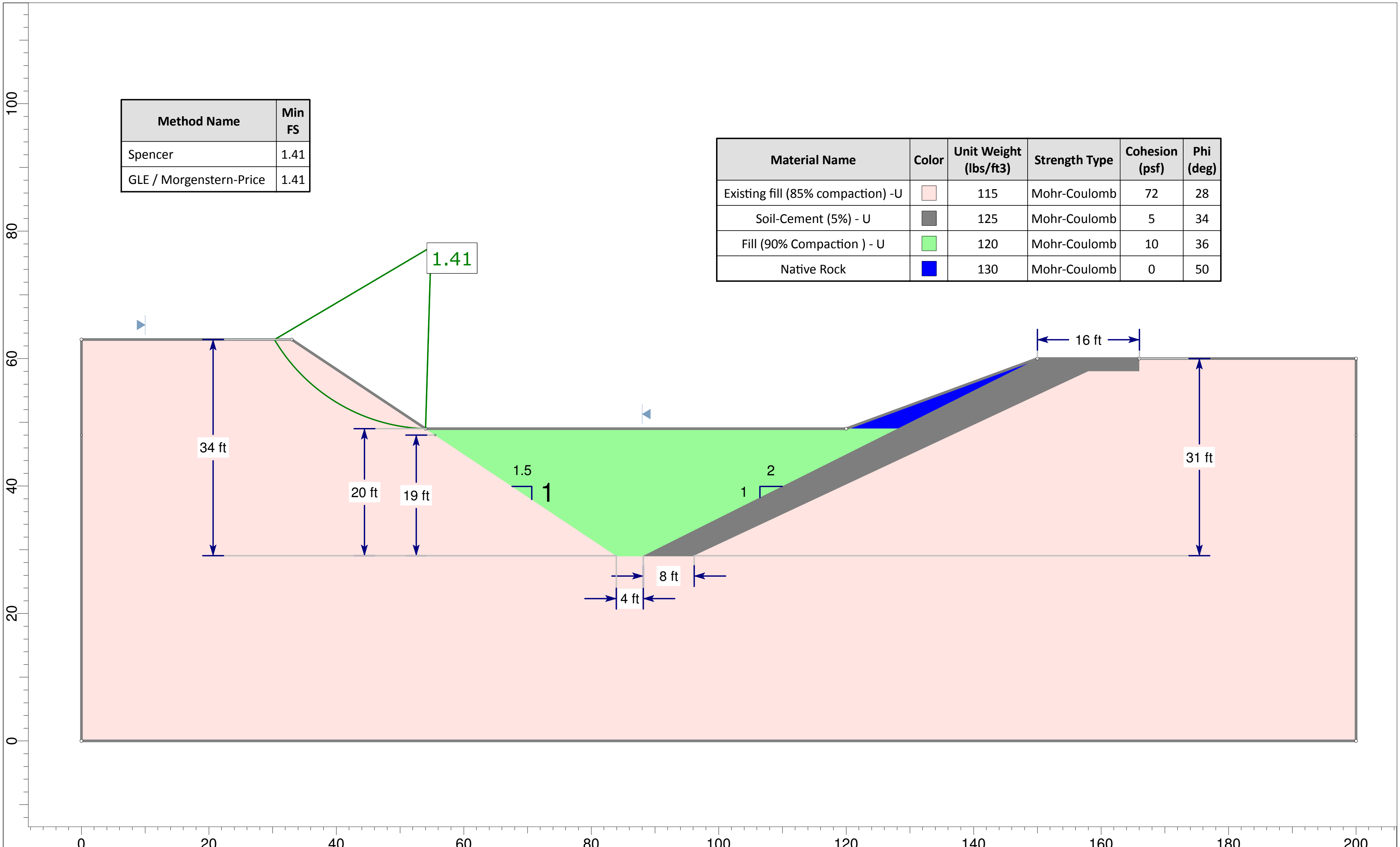
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Existing fill (85% compaction) -P		120	Mohr-Coulomb	90	31
Soil-Cement (5%) - P		130	Mohr-Coulomb	426	44
Fill (90% Compaction) - P		125	Mohr-Coulomb	10	36
Native Rock		130	Mohr-Coulomb	10	50

Method Name	Min FS
Spencer	1.79
GLE / Morgenstern-Price	1.78

PETRA GEOSCIENCES INC.

SLIDEINTERPRET 8.032

Project	Whitewater Preserve Flood Control
Analysis Description	Analysis Description Stability of Levee Slope during Operation Highest Slope (Sta.18+00), Seepage @ 28' above the bottom of Valley, Rapid Drawdown
Drawn By	AM
Scale	1:250
Company	Petra Geosciences, Inc.
Date	6/16/2020
File Name	Slope Stability.slmd



Method Name	Min FS
Spencer	1.41
GLE / Morgenstern-Price	1.41

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Existing fill (85% compaction) -U		115	Mohr-Coulomb	72	28
Soil-Cement (5%) - U		125	Mohr-Coulomb	5	34
Fill (90% Compaction) - U		120	Mohr-Coulomb	10	36
Native Rock		130	Mohr-Coulomb	0	50

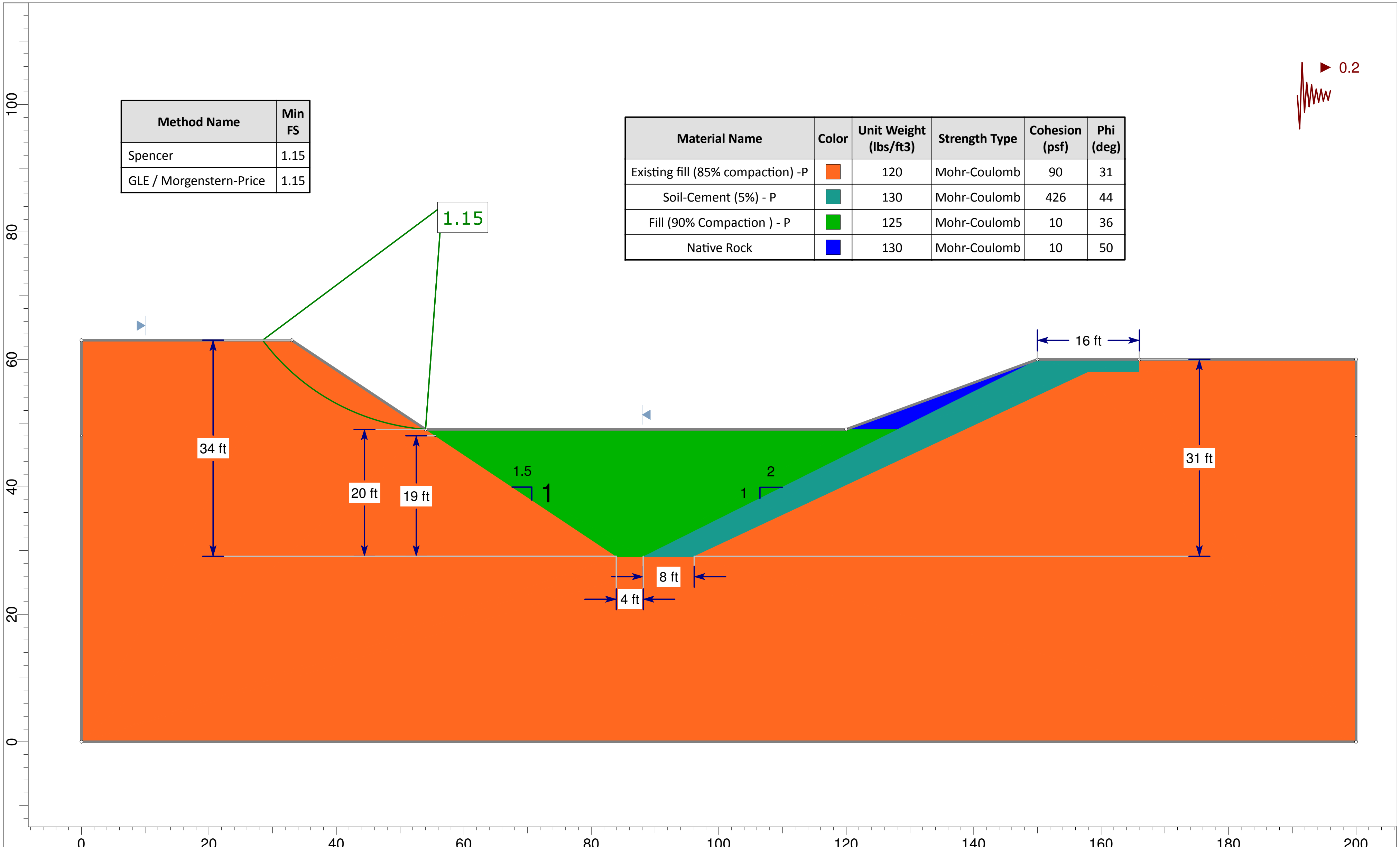
PETRA GEOSCIENCES INC

SLIDEINTERPRET 8.032

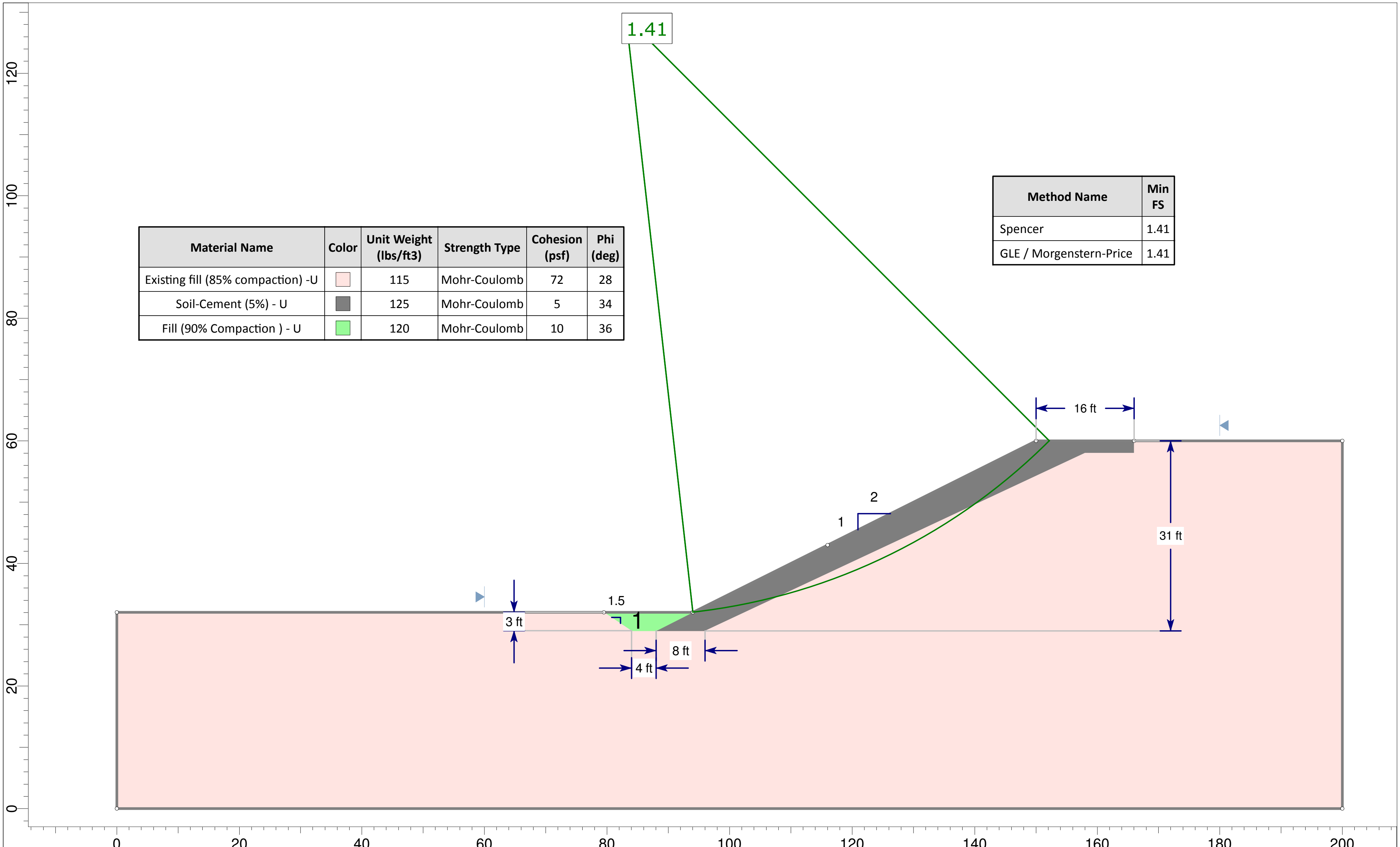
Project	Whitewater Preserve Flood Control
Analysis Description	Stability of False Slope during Operation – Highest Slope (Sta.26+00), Dry Condition, Static
Drawn By	AM
Scale	1:250
Company	Petra Geosciences, Inc.
Date	6/16/2020
File Name	False Slope.slmd

Method Name	Min FS
Spencer	1.15
GLE / Morgenstern-Price	1.15

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Existing fill (85% compaction) -P		120	Mohr-Coulomb	90	31
Soil-Cement (5%) - P		130	Mohr-Coulomb	426	44
Fill (90% Compaction) - P		125	Mohr-Coulomb	10	36
Native Rock		130	Mohr-Coulomb	10	50



PETRA GEOSCIENCES INC	*Project* Whitewater Preserve Flood Control		
	Analysis Description Stability of False Slope during Operation – Highest Slope (Sta.26+00), Dry Condition, Pseudo-Static		
	Drawn By AM	*Scale* 1:250	*Company* Petra Geosciences, Inc.
SLIDEINTERPRET 8.032	*Date* 6/16/2020		*File Name* False Slope.slmd



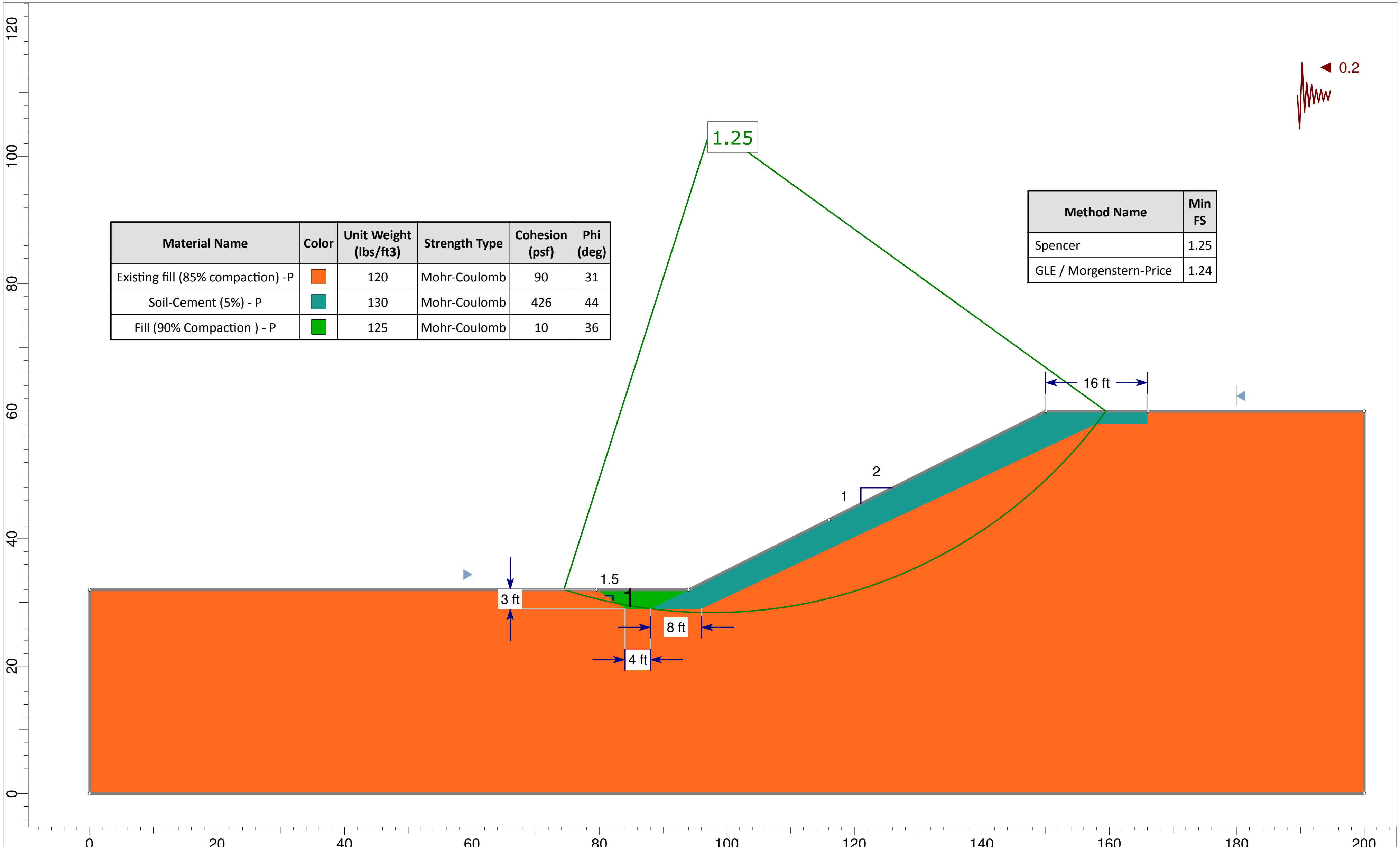
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Existing fill (85% compaction) -U		115	Mohr-Coulomb	72	28
Soil-Cement (5%) - U		125	Mohr-Coulomb	5	34
Fill (90% Compaction) - U		120	Mohr-Coulomb	10	36

Method Name	Min FS
Spencer	1.41
GLE / Morgenstern-Price	1.41

PETRA GEOSCIENCES INC.

SLIDEINTERPRET 8.032

Project	Whitewater Preserve Flood Control
Analysis Description	Stability of Levee Slope After 500-Year Flood (Post Scour Condition) – Highest Slope (Sta.26+00), Dry Condition, Static
Drawn By	AM
Scale	1:260
Company	Petra Geosciences, Inc.
Date	6/16/2020
File Name	Soil-Cement.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Existing fill (85% compaction) -P		120	Mohr-Coulomb	90	31
Soil-Cement (5%) - P		130	Mohr-Coulomb	426	44
Fill (90% Compaction) - P		125	Mohr-Coulomb	10	36

Method Name	Min FS
Spencer	1.25
GLE / Morgenstern-Price	1.24

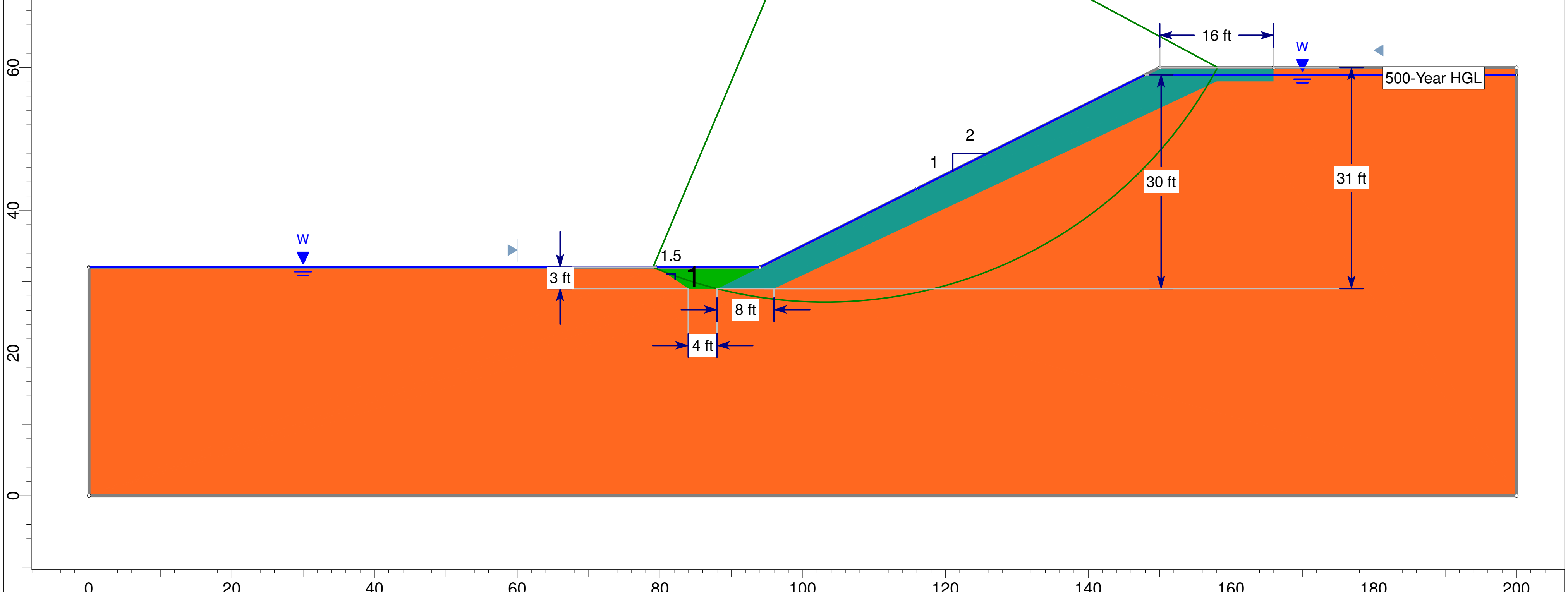
PETRA GEOSCIENCES INC

SLIDEINTERPRET 8.032

Project	Whitewater Preserve Flood Control		
Analysis Description	Stability of Levee Slope After 500-Year Flood (Post Scour Condition) – Highest Slope (Sta.26+00), Dry Condition, Pseudo-Static		
Drawn By AM	*Scale* 1:250	*Company*	Petra Geosciences, Inc.
Date	6/16/2020	*File Name*	Soil-Cement.slmd

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Existing fill (85% compaction) -P		120	Mohr-Coulomb	90	31
Soil-Cement (5%) - P		130	Mohr-Coulomb	426	44
Fill (90% Compaction) - P		125	Mohr-Coulomb	10	36

Method Name	Min FS
Spencer	1.00
GLE / Morgenstern-Price	1.00



Project	Whitewater Preserve Flood Control		
Analysis Description	Stability of Levee Slope After 500-Year Flood (Post Scour Condition) Highest Slope (St.26+00), Seepage @ 28' above the bottom of Valley, Rapid Drawdow		
Drawn By AM	*Scale* 1:250	*Company*	Petra Geosciences, Inc.
Date	6/16/2020	*File Name*	Post Scour.slmd

PETRA GEOSCIENCES INC

SLIDEINTERPRET 8.032

APPENDIX E

STANDARD GRADING SPECIFICATIONS

DRAFT

STANDARD GRADING SPECIFICATIONS

These specifications present the usual and minimum requirements for projects on which Petra Geosciences, Inc. (Petra) is the geotechnical consultant. No deviation from these specifications will be allowed, except where specifically superseded in the preliminary geology and soils report, or in other written communication signed by the Soils Engineer and Engineering Geologist of record (Geotechnical Consultant).

I. GENERAL

A. The Geotechnical Consultant is the Owner's or Builder's representative on the project. For the purpose of these specifications, participation by the Geotechnical Consultant includes that observation performed by any person or persons employed by, and responsible to, the licensed Soils Engineer and Engineering Geologist signing the soils report.

B. The contractor should prepare and submit to the Owner and Geotechnical Consultant a work plan that indicates the sequence of earthwork grading, the number of "spreads" and the estimated quantities of daily earthwork to be performed prior to the commencement of grading. This work plan should be reviewed by the Geotechnical Consultant to schedule personnel to perform the appropriate level of observation, mapping, and compaction testing as necessary.

C. All clearing, site preparation, or earthwork performed on the project shall be conducted by the Contractor in accordance with the recommendations presented in the geotechnical report and under the observation of the Geotechnical Consultant.

D. It is the Contractor's responsibility to prepare the ground surface to receive the fills to the satisfaction of the Geotechnical Consultant and to place, spread, mix, water, and compact the fill in accordance with the specifications of the Geotechnical Consultant. The Contractor shall also remove all material considered unsatisfactory by the Geotechnical Consultant.

E. It is the Contractor's responsibility to have suitable and sufficient compaction equipment on the job site to handle the amount of fill being placed. If necessary, excavation equipment will be shut down to permit completion of compaction to project specifications. Sufficient watering apparatus will also be provided by the Contractor, with due consideration for the fill material, rate of placement, and time of year.

F. After completion of grading a report will be submitted by the Geotechnical Consultant.

II. SITE PREPARATION

A. Clearing and Grubbing

1. All vegetation such as trees, brush, grass, roots, and deleterious material shall be disposed of offsite. This removal shall be concluded prior to placing fill.

2. Any underground structures such as cesspools, cisterns, mining shafts, tunnels, septic tanks, wells, pipe lines, etc., are to be removed or treated in a manner prescribed by the Geotechnical Consultant.

III. FILL AREA PREPARATION

A. Remedial Removals/Overexcavations

1. Remedial removals, as well as overexcavation for remedial purposes, shall be evaluated by the Geotechnical Consultant. Remedial removal depths presented in the geotechnical report and shown on the geotechnical plans are estimates only. The actual extent of removal should be determined by the Geotechnical Consultant based on the conditions exposed during grading. All soft, loose, dry, saturated, spongy, organic-rich, highly fractured or otherwise unsuitable ground shall be overexcavated to competent ground as determined by the Geotechnical Consultant.

2. Soil, alluvium, or bedrock materials determined by the Soils Engineer as being unsuitable for placement in compacted fills shall be removed from the site. Any material incorporated as a part of a compacted fill must be approved by the Geotechnical Consultant.

3. Should potentially hazardous materials be encountered, the Contractor should stop work in the affected area. An environmental consultant specializing in hazardous materials should be notified immediately for evaluation and handling of these materials prior to continuing work in the affected area.

B. Evaluation/Acceptance of Fill Areas

All areas to receive fill, including removal and processed areas, key bottoms, and benches, shall be observed, mapped, elevations recorded, and/or tested prior to being accepted by the Geotechnical Consultant as suitable to receive fill. The contractor shall obtain a written acceptance from the Geotechnical Consultant prior to fill placement. A licensed surveyor shall provide sufficient survey control for determining locations and elevations of processed areas, keys, and benches.

C. Processing

After the ground surface to receive fill has been declared satisfactory for support of fill by the Geotechnical Consultant, it shall be scarified to a minimum depth of 6 inches and until the ground surface is uniform and free from ruts, hollows, hummocks, or other uneven features which may prevent uniform compaction.

The scarified ground surface shall then be brought to optimum moisture, mixed as required, and compacted to a minimum relative compaction of 90 percent.

D. Subdrains

Subdrainage devices shall be constructed in compliance with the ordinances of the controlling governmental agency, and/or with the recommendations of the Geotechnical Consultant. (Typical Canyon Subdrain details are given on Plate SG-1).

E. Cut/Fill & Deep Fill/Shallow Fill Transitions

In order to provide uniform bearing conditions in cut/fill and deep fill/shallow fill transition lots, the cut and shallow fill portions of the lot should be overexcavated to the depths and the horizontal limits discussed in the approved geotechnical report and replaced with compacted fill. (Typical details are given on Plate SG-7.)

IV. COMPACTED FILL MATERIAL

A. General

Materials excavated on the property may be utilized in the fill, provided each material has been determined to be suitable by the Geotechnical Consultant. Material to be used for fill shall be essentially free of organic material and other deleterious substances. Roots, tree branches, and other matter missed during clearing shall be removed from the fill as recommended by the Geotechnical Consultant. Material that is spongy, subject to decay, or otherwise considered unsuitable shall not be used in the compacted fill.

Soils of poor quality, such as those with unacceptable gradation, high expansion potential, or low strength shall be placed in areas acceptable to the Geotechnical Consultant or mixed with other soils to achieve satisfactory fill material.

B. Oversize Materials

Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 12 inches in diameter, shall be taken offsite or placed in accordance with the recommendations of the Geotechnical Consultant in areas designated as suitable for rock disposal (Typical details for Rock Disposal are given on Plate SG-4).

Rock fragments less than 12 inches in diameter may be utilized in the fill provided, they are not nested or placed in concentrated pockets; they are surrounded by compacted fine grained soil material and the distribution of rocks is approved by the Geotechnical Consultant.

C. Laboratory Testing

Representative samples of materials to be utilized as compacted fill shall be analyzed by the laboratory of the Geotechnical Consultant to determine their physical properties. If any material other than that previously tested is encountered during grading, the appropriate analysis of this material shall be conducted by the Geotechnical Consultant as soon as possible.

D. Import

If importing of fill material is required for grading, proposed import material should meet the requirements of the previous section. The import source shall be given to the Geotechnical Consultant at least 2 working days prior to importing so that appropriate tests can be performed and its suitability determined.

V. FILL PLACEMENT AND COMPACTION

A. Fill Layers

Material used in the compacting process shall be evenly spread, watered, processed, and compacted in thin lifts not to exceed 6 inches in thickness to obtain a uniformly dense layer. The fill shall be placed and compacted on a horizontal plane, unless otherwise approved by the Geotechnical Consultant.

B. Moisture Conditioning

Fill soils shall be watered, dried back, blended, and/or mixed, as necessary to attain a relatively uniform moisture content at or slightly above optimum moisture content.

C. Compaction

Each layer shall be compacted to 90 percent of the maximum density in compliance with the testing method specified by the controlling governmental agency. (In general, ASTM D 1557-02, will be used.)

If compaction to a lesser percentage is authorized by the controlling governmental agency because of a specific land use or expansive soils condition, the area to received fill compacted to less than 90 percent shall either be delineated on the grading plan or appropriate reference made to the area in the soils report.

D. Failing Areas

If the moisture content or relative density varies from that required by the Geotechnical Consultant, the Contractor shall rework the fill until it is approved by the Geotechnical Consultant.

E. Benching

All fills shall be keyed and benched through all topsoil, colluvium, alluvium or creep material, into sound bedrock or firm material where the slope receiving fill exceeds a ratio of 5 horizontal to 1 vertical, in accordance with the recommendations of the Geotechnical Consultant.

VI. SLOPES

A. Fill Slopes

The contractor will be required to obtain a minimum relative compaction of 90 percent out to the finish slope face of fill slopes, buttresses, and stabilization fills. This may be achieved by either overbuilding the slope and cutting back to the compacted core, or by direct compaction of the slope face with suitable equipment, or by any other procedure that produces the required compaction.

B. Side Hill Fills

The key for side hill fills shall be a minimum of 15 feet within bedrock or firm materials, unless otherwise specified in the soils report. (See detail on Plate SG-5.)

C. Fill-Over-Cut Slopes

Fill-over-cut slopes shall be properly keyed through topsoil, colluvium or creep material into rock or firm materials, and the transition shall be stripped of all soils prior to placing fill. (see detail on Plate SG-6).

D. Landscaping

All fill slopes should be planted or protected from erosion by other methods specified in the soils report.

E. Cut Slopes

1. The Geotechnical Consultant should observe all cut slopes at vertical intervals not exceeding 10 feet.

2. If any conditions not anticipated in the preliminary report such as perched water, seepage, lenticular or confined strata of a potentially adverse nature, unfavorably inclined bedding, joints or fault planes are encountered during grading, these conditions shall be evaluated by the Geotechnical Consultant, and recommendations shall be made to treat these problems (Typical details for stabilization of a portion of a cut slope are given in Plates SG-2 and SG-3.).

3. Cut slopes that face in the same direction as the prevailing drainage shall be protected from slope wash by a non-erodible interceptor swale placed at the top of the slope.

4. Unless otherwise specified in the soils and geological report, no cut slopes shall be excavated higher or steeper than that allowed by the ordinances of controlling governmental agencies.

5. Drainage terraces shall be constructed in compliance with the ordinances of controlling governmental agencies, or with the recommendations of the Geotechnical Consultant.

VII. GRADING OBSERVATION

A. General

All cleanouts, processed ground to receive fill, key excavations, subdrains, and rock disposals must be observed and approved by the Geotechnical Consultant prior to placing any fill. It shall be the Contractor's responsibility to notify the Geotechnical Consultant when such areas are ready.

B. Compaction Testing
Observation of the fill placement shall be provided by the Geotechnical Consultant during the progress of grading. Location and frequency of tests shall be at the Consultants discretion based on field conditions encountered. Compaction test locations will not necessarily be selected on a random basis. Test locations may be selected to verify adequacy of compaction levels in areas that are judged to be susceptible to inadequate compaction.

C. Frequency of Compaction Testing

In general, density tests should be made at intervals not exceeding 2 feet of fill height or every 1000 cubic yards of fill placed. This criteria will vary depending on soil conditions and the size of the job. In any event, an adequate number of field density tests shall be made to verify that the required compaction is being achieved.

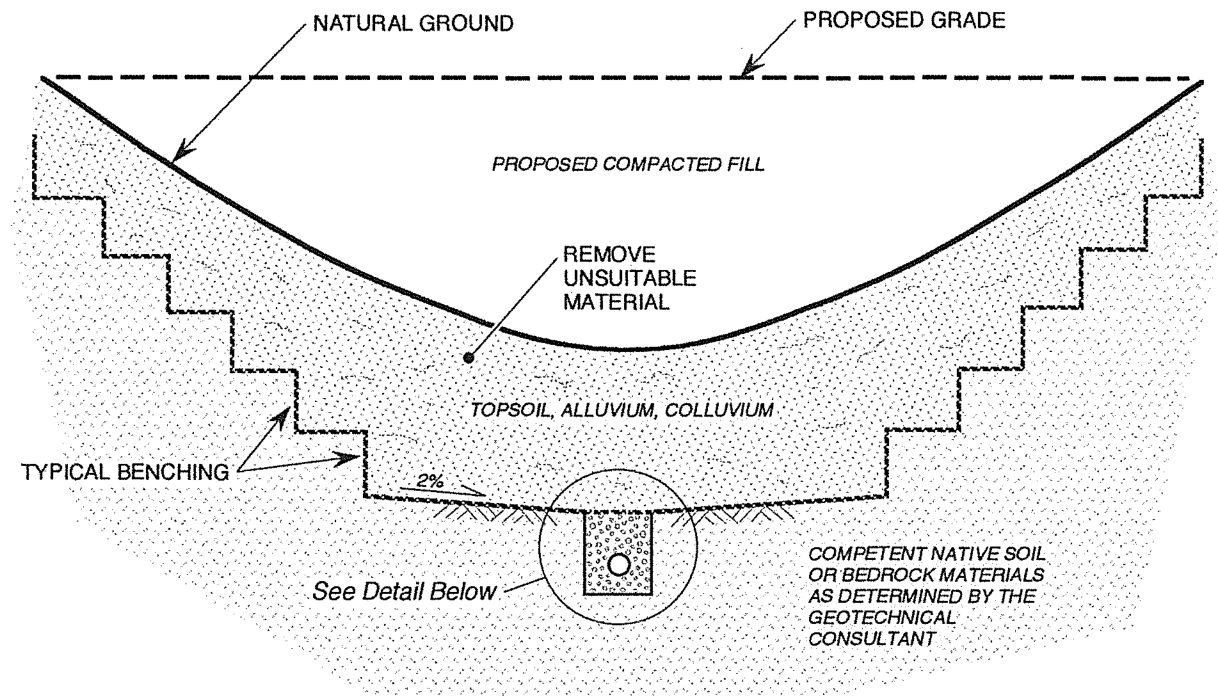
VIII. CONSTRUCTION CONSIDERATIONS

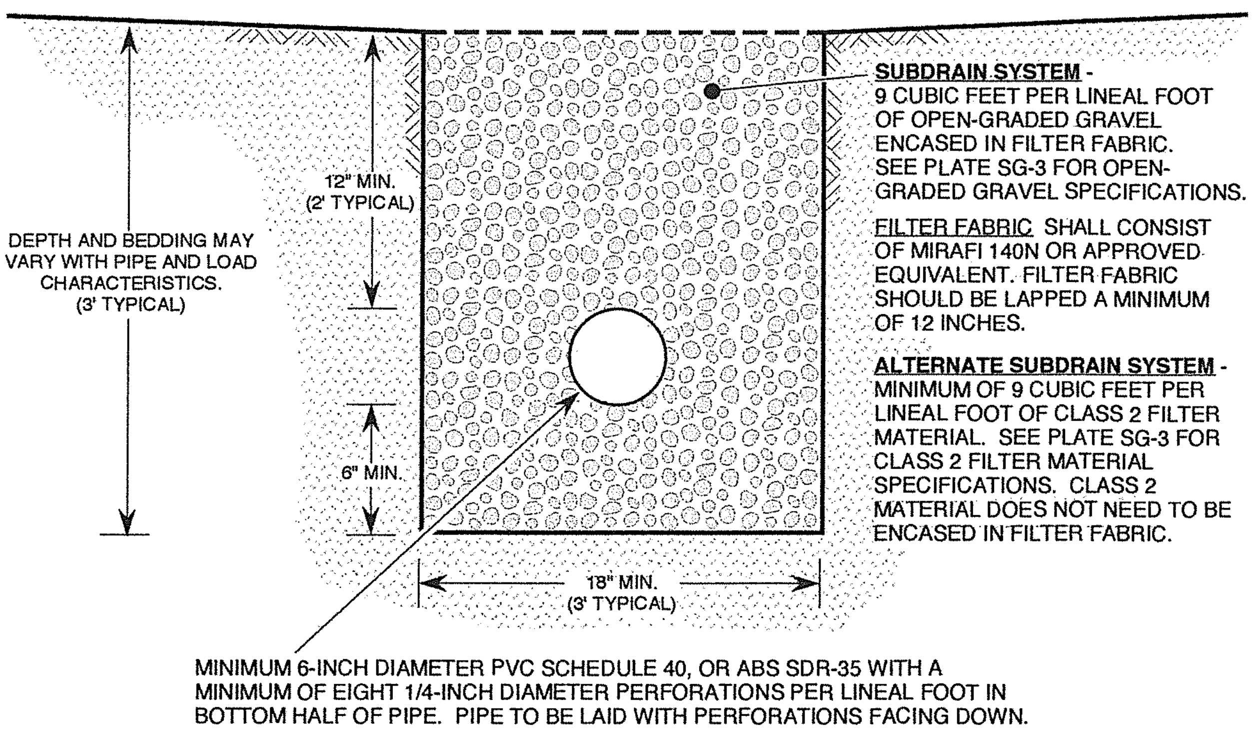
A. Erosion control measures, when necessary, shall be provided by the Contractor during grading and prior to the completion and construction of permanent drainage controls.

B. Upon completion of grading and termination of observations by the Geotechnical Consultant, no further filling or excavating, including that necessary for footings, foundations, large tree wells, retaining walls, or other features shall be performed without the approval of the Geotechnical Consultant.

C. Care shall be taken by the Contractor during final grading to preserve any berms, drainage terraces, interceptor swales, or other devices of permanent nature on or adjacent to the property.

S:\!BOILERS-WORK\REPORT INSERTS\STANDARD GRADING SPECS

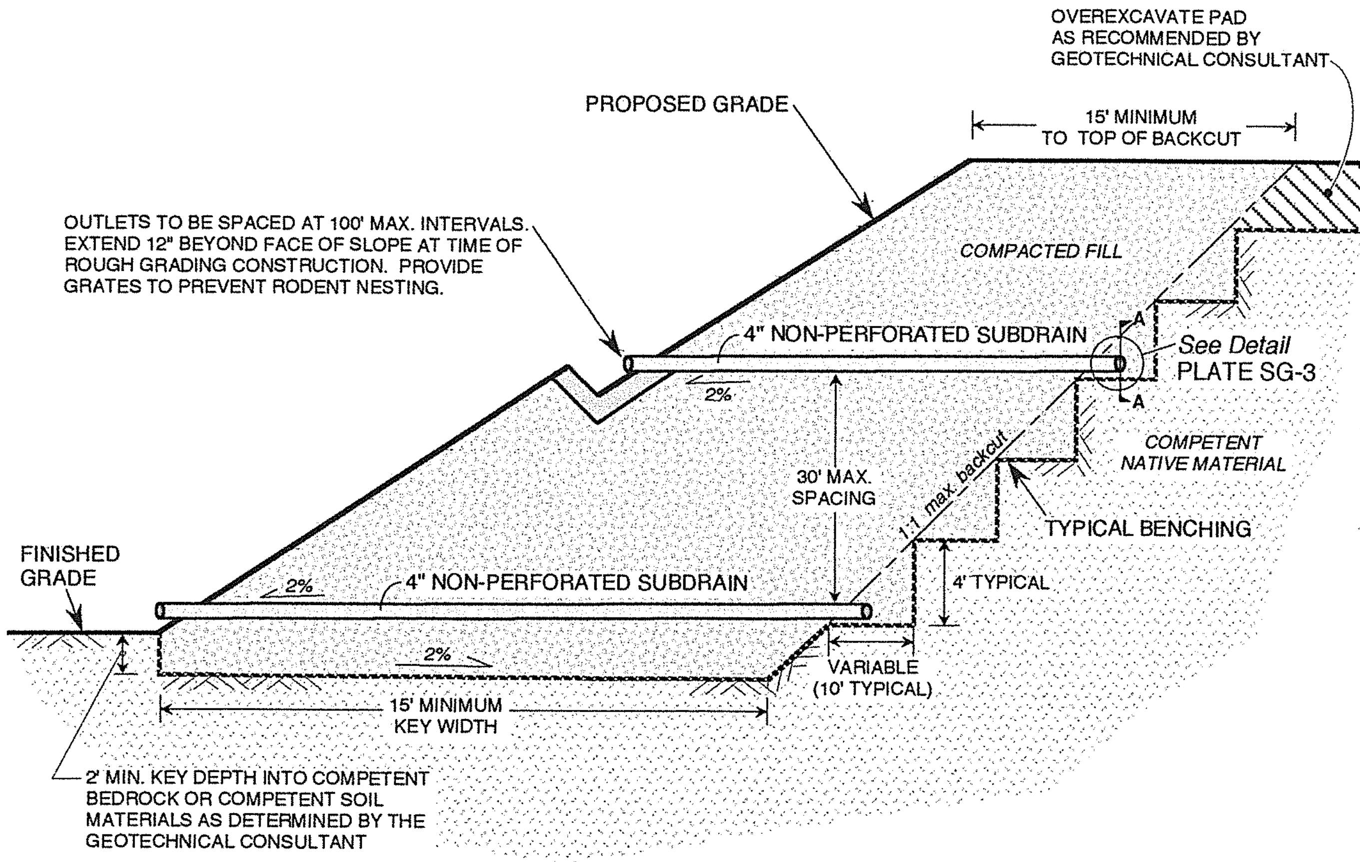




NOTES:

1. FOR CONTINUOUS RUNS IN EXCESS OF 500 FEET USE 8-INCH DIAMETER PIPE.
2. FINAL 20 FEET OF PIPE AT OUTLET SHALL BE NON-PERFORATED AND BACKFILLED WITH FINE-GRAINED MATERIAL.

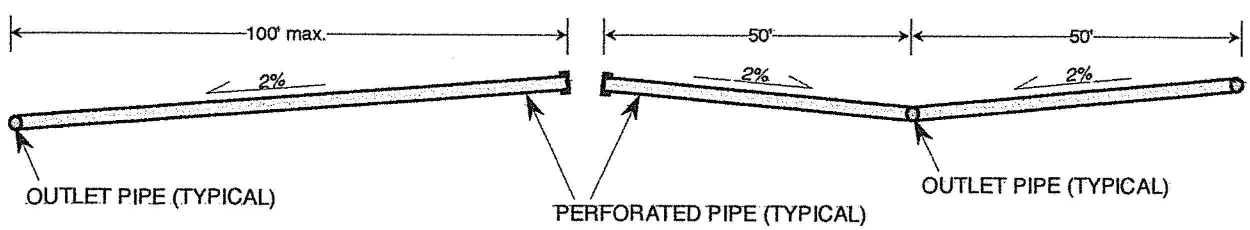
PETRA	CANYON SUBDRAIN DETAIL	PLATE SG-1



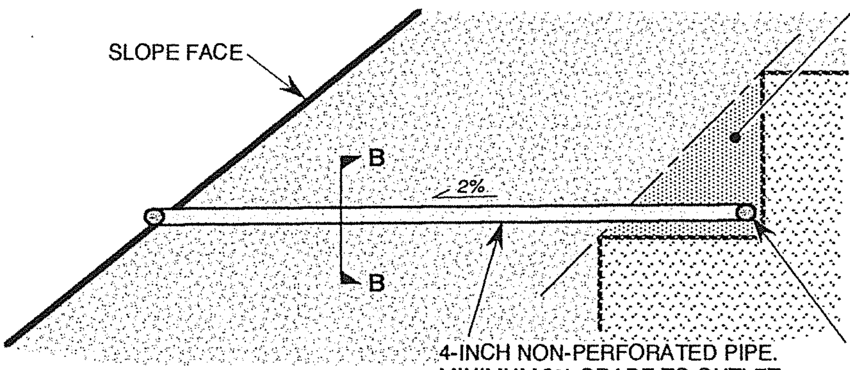
NOTES:

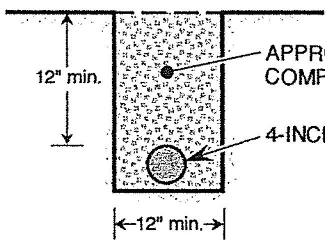
1. 30' MAXIMUM VERTICAL SPACING BETWEEN SUBDRAIN SYSTEMS.
2. 100' MAXIMUM HORIZONTAL DISTANCE BETWEEN NON-PERFORATED OUTLET PIPES. *(See Below)*
3. MINIMUM GRADIENT OF 2% FOR ALL PERFORATED AND NON-PERFORATED PIPE.

SECTION A-A (PERFORATED PIPE PROFILE)



PETRA	**BUTTRESS OR STABILIZATION FILL DETAIL**	**PLATE SG-2**





SECTION B-B (OUTLET PIPE)

PIPE SPECIFICATIONS:

1. 4-INCH MINIMUM DIAMETER, PVC SCHEDULE 40 OR ABS SDR-35.
2. FOR PERFORATED PIPE, MINIMUM 8 PERFORATIONS PER FOOT ON BOTTOM HALF OF PIPE.

FILTER MATERIAL/FABRIC SPECIFICATIONS:

OPEN-GRADED GRAVEL ENCASED IN FILTER FABRIC.
(MIRAFI 140N OR EQUIVALENT)

OPEN-GRADED GRAVEL

SIEVE SIZE	PERCENT PASSING
1 1/2-INCH	88 - 100
1-INCH	5 - 40
3/4-INCH	0 - 17
3/8-INCH	0 - 7
No. 200	0 - 3

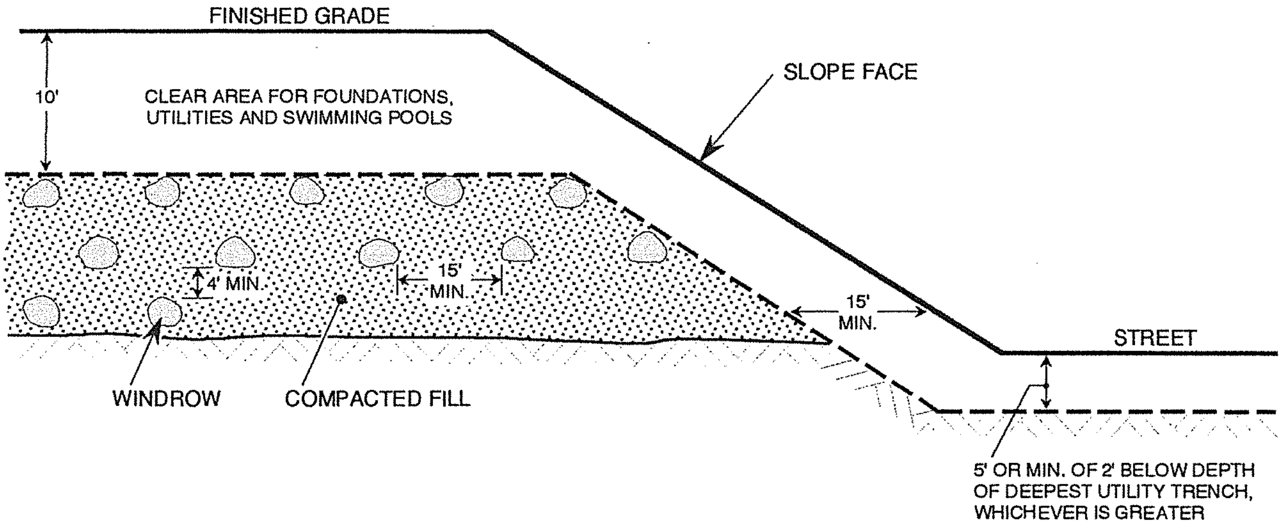
ALTERNATE:

CLASS 2 PERMEABLE FILTER MATERIAL PER CALTRANS STANDARD SPECIFICATION 68-1.025.

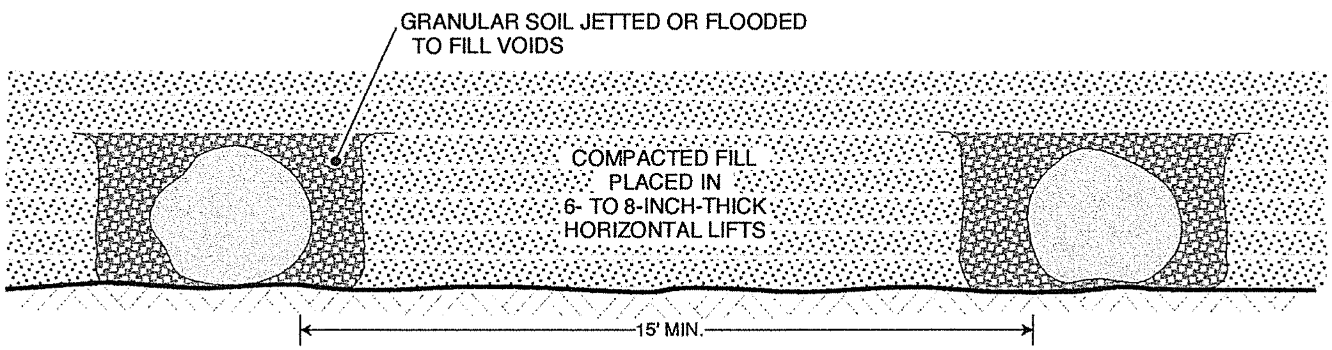
CLASS 2 FILTER MATERIAL

SIEVE SIZE	PERCENT PASSING
1-INCH	100
3/4-INCH	90 - 100
3/8-INCH	40 - 100
No. 4	25 - 40
No. 8	18 - 33
No. -30	5 - 15
No. -50	0 - 7
No. 200	0 - 3

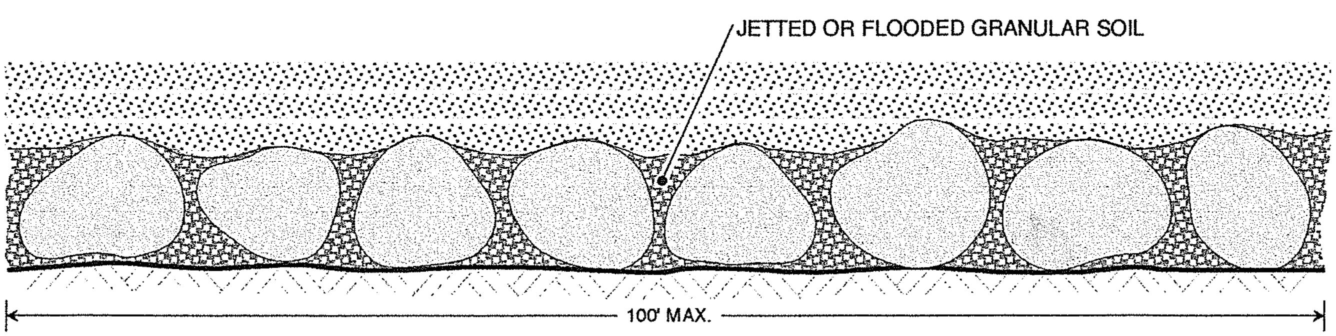
PETRA	BUTTRESS OR STABILIZATION FILL SUBDRAIN	PLATE SG-3



TYPICAL WINDROW DETAIL (END VIEW)

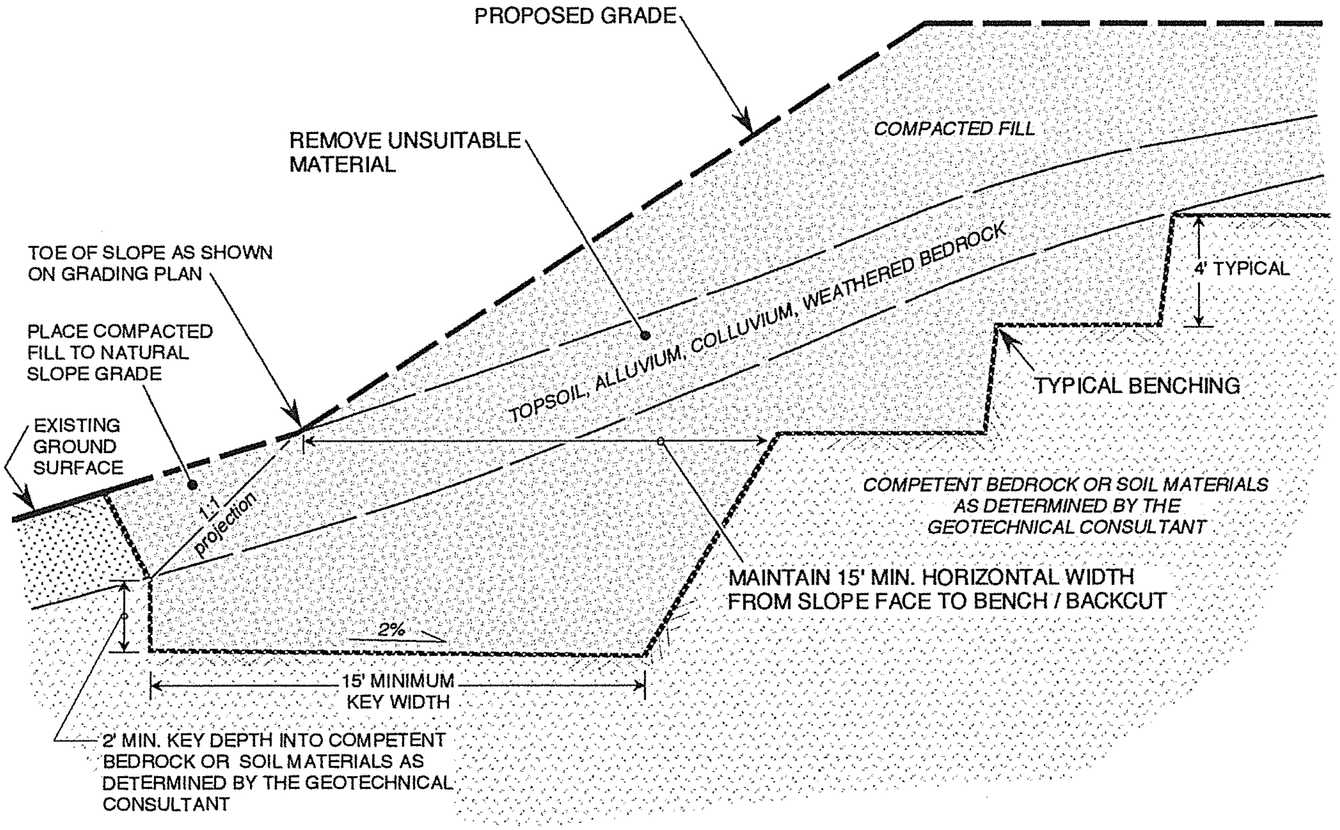


TYPICAL WINDROW DETAIL (PROFILE VIEW)



NOTE: OVERSIZE ROCK IS DEFINED AS CLASTS HAVING A MAXIMUM DIMENSION OF 12" OR LARGER

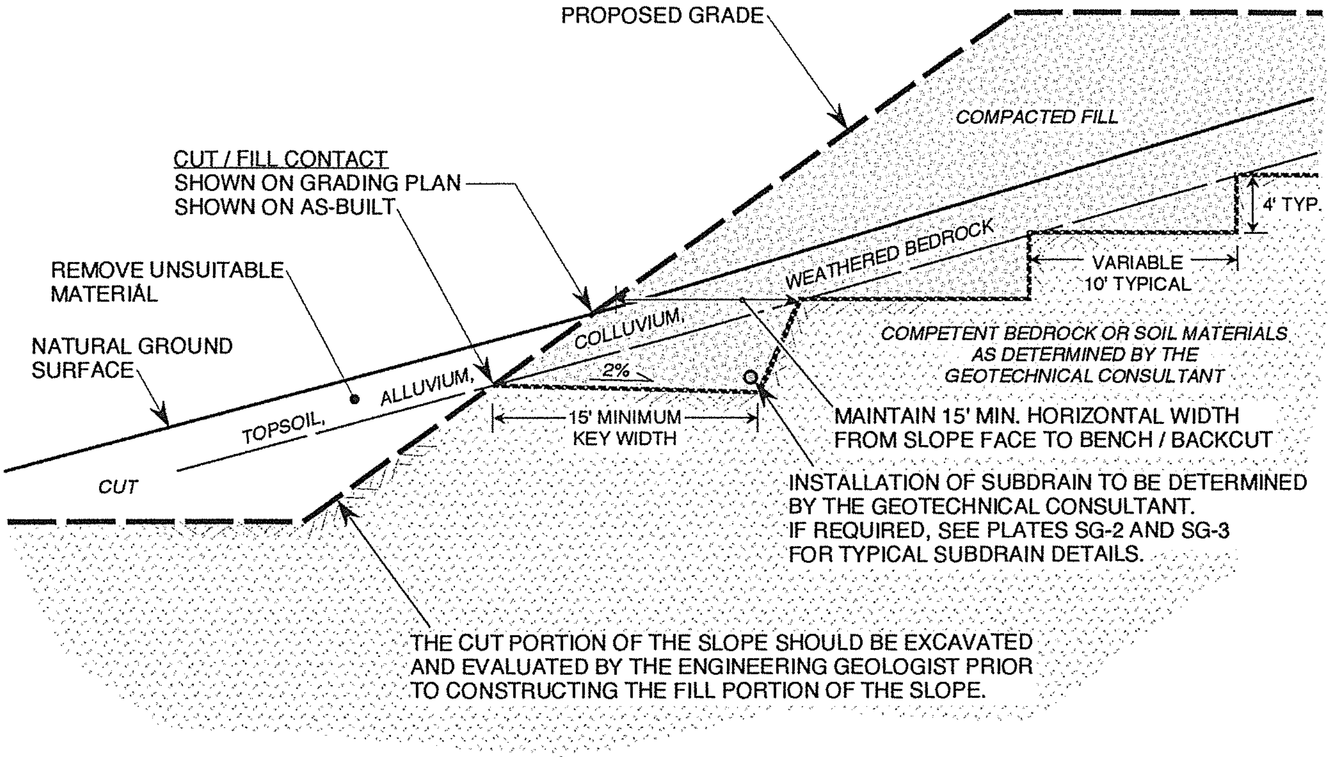
PETRA	TYPICAL ROCK DISPOSAL DETAIL	PLATE SG-4



NOTES:

1. WHERE NATURAL SLOPE GRADIENT IS 5:1 OR LESS, BENCHING IS NOT NECESSARY; HOWEVER, FILL IS NOT TO BE PLACED ON COMPRESSIBLE OR UNSUITABLE MATERIAL.
2. SOILS ENGINEER TO DETERMINE IF SUBDRAIN IS REQUIRED.

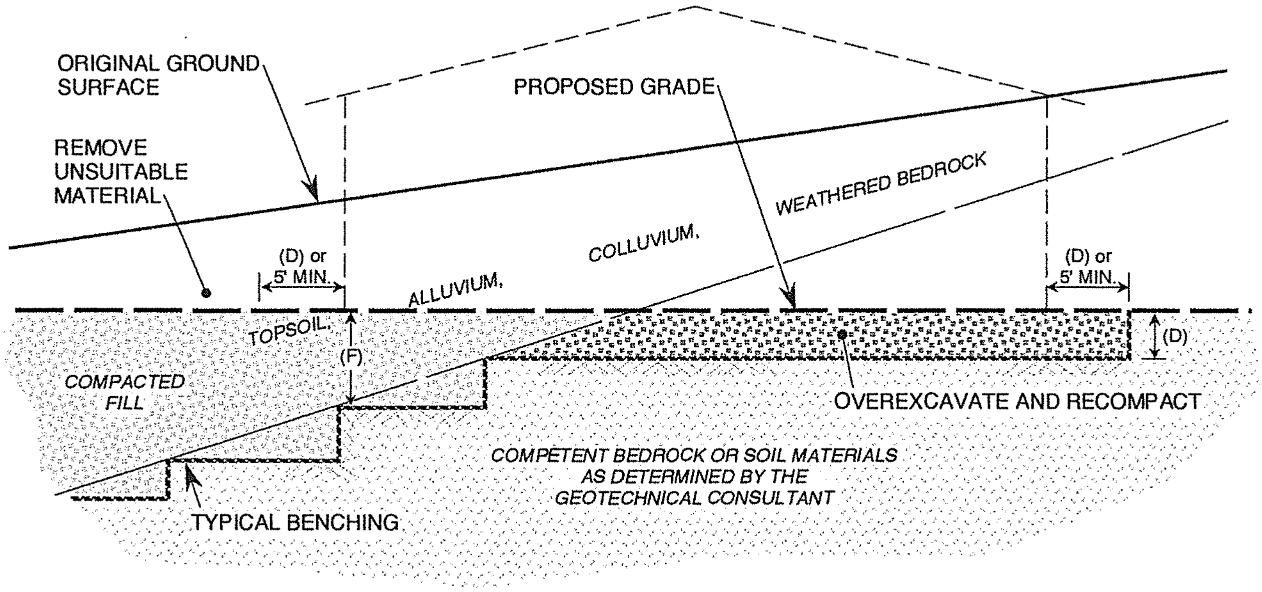
PETRA	FILL SLOPE ABOVE NATURAL SLOPE	PLATE SG-5



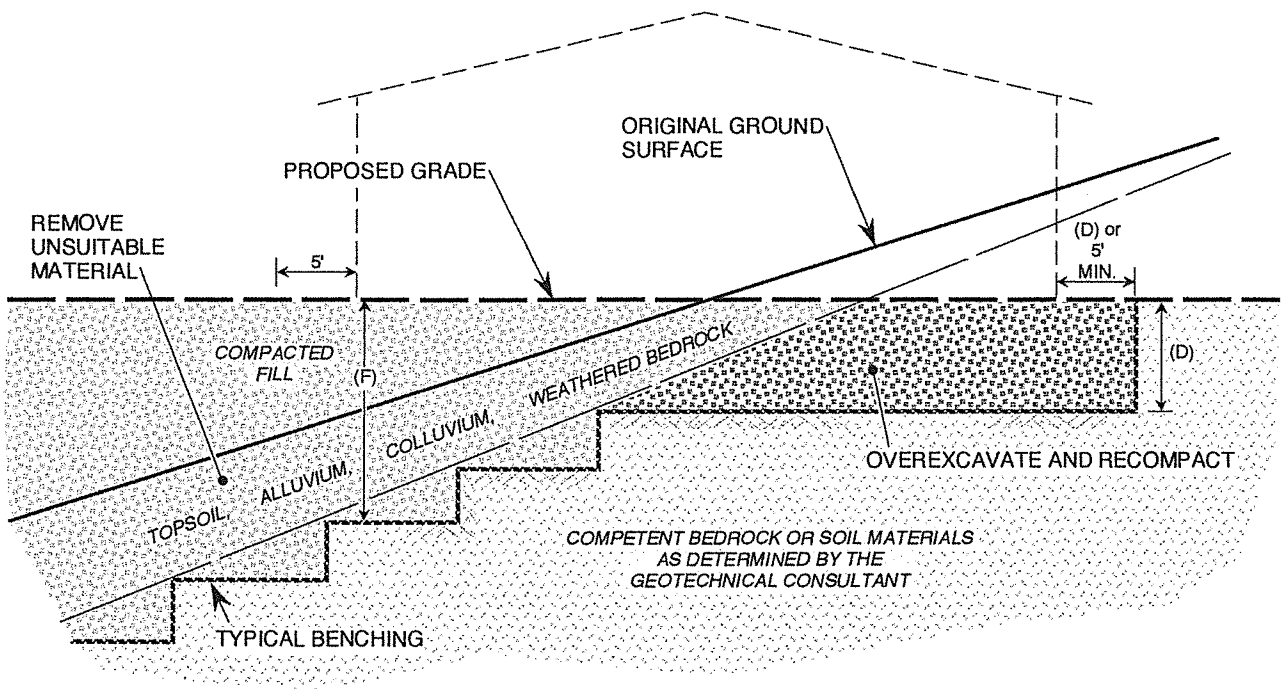
PETRA	FILL SLOPE ABOVE CUT SLOPE	PLATE SG-6

CUT LOT

UNSUITABLE MATERIAL EXPOSED IN PORTION OF CUT PAD



CUT-FILL TRANSITION LOT

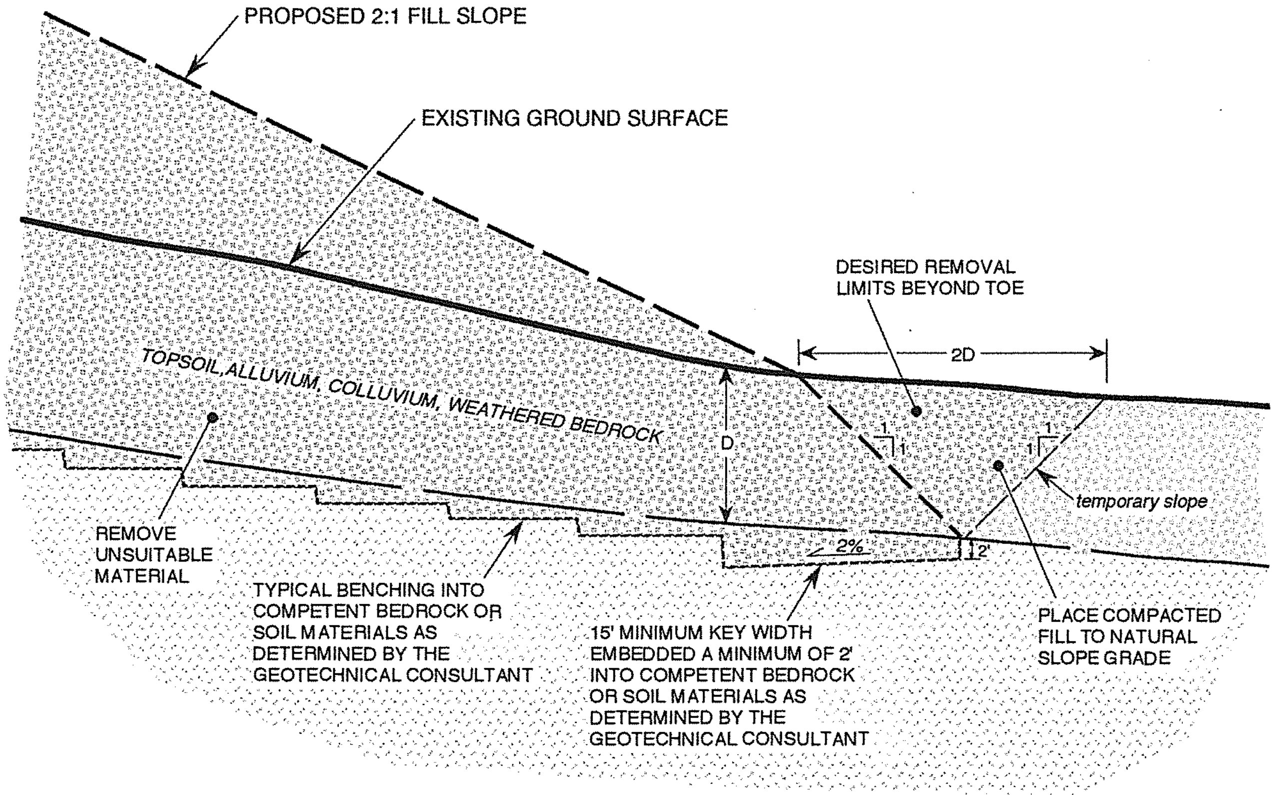


MAXIMUM FILL THICKNESS (F)	DEPTH OF OVEREXCAVATION (D)
FOOTING DEPTH TO 3 FEET	EQUAL DEPTH
3 TO 6 FEET	3 FEET
GREATER THAN 6 FEET	1/2 THE THICKNESS OF DEEPEST FILL PLACED WITHIN THE "FILL" PORTION (F) TO 15 FEET MAXIMUM

PETRA

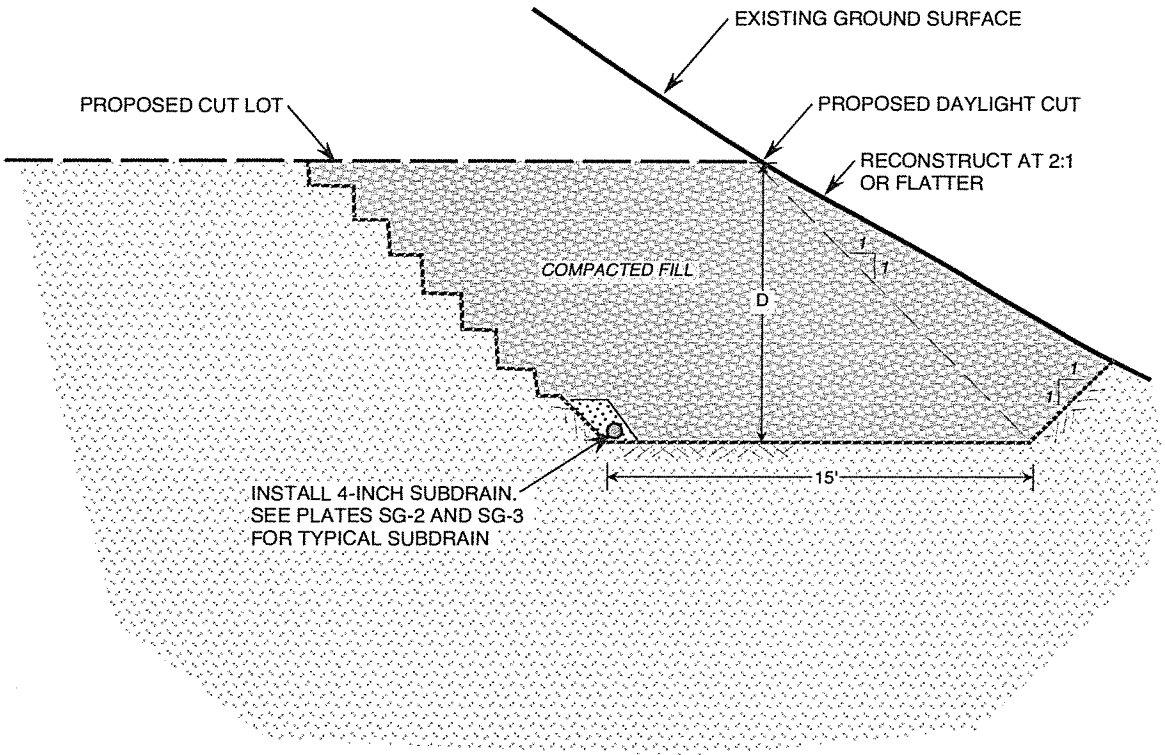
CUT LOTS AND CUT-FILL TRANSITION LOTS

PLATE SG-7



D = RECOMMENDED DEPTH OF REMOVAL
PER GEOTECHNICAL REPORT

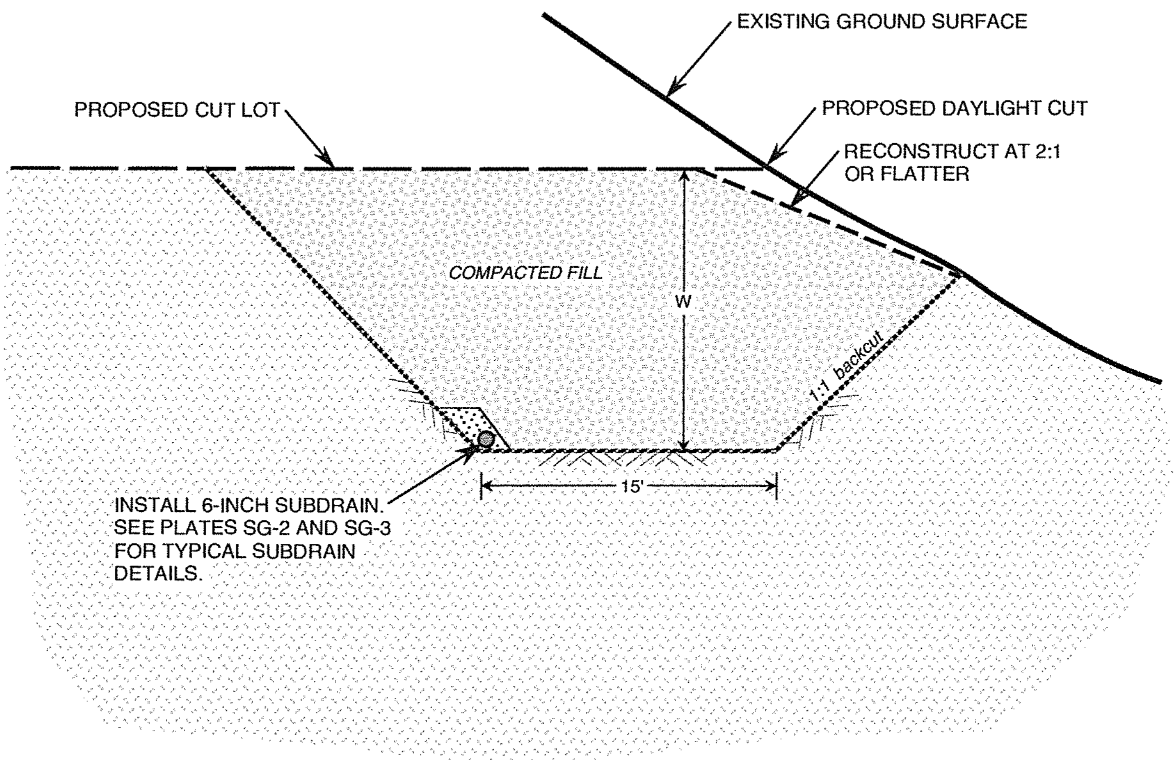
PETRA	TYPICAL REMOVALS BEYOND TOE OF PROPOSED FILL SLOPE	PLATE SG-8



NOTE:

1. "D" SHALL BE 10 FEET MINIMUM OR AS DETERMINED BY SOILS ENGINEER.

PETRA	SHEAR KEY ON DAYLIGHT CUT LOTS	PLATE SG-9



NOTE:

1. "W" SHALL BE 10 FEET MINIMUM OR AS DETERMINED BY SOILS ENGINEER.

PETRA	SHEAR KEY ON DAYLIGHT CUT LOTS	PLATE SG-9

APPENDIX F

PHOTOS TAKEN DURING THE FIELD EXPLORATION

DRAFT


JOHN DEERE

APPENDIX G

SOIL-CEMENT BANK PROTECTION GUIDELINES

DRAFT

SOIL-CEMENT BANK PROTECTION GUIDELINES

The following guidelines present the usual and minimum requirements for projects on which **Petra Geosciences, Inc. (Petra)** is the original geotechnical consultant. No deviation from these minimum guidelines will be allowed, except where specifically superseded in the main text of this report, or in other written communication signed by the Geotechnical Engineer and Engineering Geologist of record (Geotechnical Consultant).

I. Construction

A. Required Contractor Submittals

Prior to the start of construction, the contractor shall submit in writing to the civil and geotechnical engineers of project the following items for approval:

a. The Sequencing of construction activities (i.e., excavation, soil cement placement, compaction, backfill, etc.)

b. The number and type of equipment to be used for placement.

c. The number and type of equipment to be used for spreading.

d. The number and type of equipment to be used for compaction.

e. The number and type of equipment to be used for watering.

f. The number of laborers needed to control placement of soil-cement and removal of contaminated soil cement.

g. The method to be used to keep surfaces continually moist until subsequent layers of soil cement are placed.

h. The method used to be to cure exposed surfaces permanently.

i. The proposed source of soil, if other than native soils.

B. Soil-Cement Test Section

Prior to placement of any soil-cement, the Contractor shall construct a soil-cement test section. The purpose of the test section is to demonstrate the suitability of the Contractor's equipment, methods, and personnel skill. The test section shall be constructed at a location similar to the subject site conditions, approved by project design engineer and geotechnical engineer, shall be the minimum of 60 feet long, 4 feet high, and consist of compacted soil-cement lifts that are 6 to 9 inches thick and 8 feet wide. The foundation upon which the first lift of soil cement is to be constructed shall be cleared, leveled, and the compacted to a minimum of 90 percent as determined by ASTM D 1557. Each soil-cement lift shall be constructed and cured as specified herein. The project design engineer and geotechnical engineer, with the assistance and cooperation of the contractor, shall perform all tests and inspections required for the permanent work on the test section. The project design and/or geotechnical engineer will observe the operation and may perform additional tests upon the materials and completed soil-cement. The contractor shall furnish all the materials and equipment and provide such assistance as necessary for sampling and testing by the geotechnical engineer. After evaluation and assessment of the test section by the Engineer, test section shall be disposed in an approved manner. Under no circumstances shall the test section be incorporated into or become a part of the permanent soil-cement structure. The test section shall demonstrate sustained soil-cement plant production rates, and batching, mixing, transporting, spreading, and compaction procedures. Soil-cement operations for the levee structure shall not begin until testing and evaluations have been completed, and it has been demonstrated to the satisfaction of the Engineer that all specification requirements were met. Following completion of test section construction, 10 calendar days shall be allowed for testing and evaluations. If the test section does not meet requirements as specified, the contractor shall make such adjustments in equipment, methods, and mix designs as are necessary to produce soil cement in accordance with the requirements of these guidelines and an additional test section or sections shall be constructed.

C. Subgrade/Foundation Preparation

A firm subgrade is necessary to compact the overlying layers of soil-cement to the required density. The subgrade is prepared by removing and replacing, or stabilizing, soft or wet areas, removing deleterious materials, and grading and compaction to construction plans and specifications. Most overly wet subgrade areas can be corrected by aerating and recompacting. Dry subgrades are surface moistened immediately prior to soil-cement placement. The subgrade shall be compacted to a minimum of 90 percent as determined by ASTM D 1557.

D. Mix Plant

The most common method of soil-cement construction for bank protection is a central mix plant. Central plant shall be located at the site of the work, subject to the approval of the engineer. There are two basic types of central mix plants, pugmill mixers either continuous or batch type, and rotary drum mixers (also a batch type of mixer). For the purpose of this project, a weight-batch type or continuous type mixing plant shall be used. Production rates between 100 and 200 cubic yard per hour are common for stair-step soil cement construction. Central mixing plants with rated capacities of 250 to 1,000 tons per hour (about 125 to 500 cubic yard) are used commonly. Special blending requirements may require several stockpiles and separate storage feeder bins.

Prior to mixing and placing, it is necessary to measure the quantities and proportions of material supplied by the plant. The plant shall be equipped with screening, feeding, and metering devices that will add the soil, cement, and water into the mixer in the specified quantities. The plant shall be accurately calibrated. The mixing shall be adequate to secure a homogeneous, intimate, uniform mixture of the soil, cement and water. Additionally, discharge from mixer shall be without segregation. Mixers shall not be charged in excess of the capacity recommended by the manufacturer. Excessive overmixing requiring additions of water will not be permitted. The mixers shall be maintained in satisfactory operating condition. Should any mixer at any time produce unsatisfactory results, its use shall be promptly discontinued until it is repaired or replaced.

Access to Mixing Facility

Free and safe access to the plant must always be provided to the engineer and geotechnical consultant for inspection of the plant's operation and for sampling the soil cement mixture and its components.

E. Required Moisture

At the time of compaction, the moisture content shall not be more than one percentage point below optimum and shall not be more than one percentage point above optimum when the mean air temperature during construction hours does not exceed 90 degrees Fahrenheit (F). When the mean air temperature does exceed 90 degrees Fahrenheit (F), or there is a breeze or a wind which promotes the rapid drying out of the soil cement mixture, the moisture content of said mix shall be increased as needed at the direction of the geotechnical engineer, but shall be less than that quantity that will cause the soil cement to become unstable during compaction and finishing operations. The optimum moisture content of soil-cement mixture shall be determined by ASTM D 1557.

F. Handling

The soil cement mixture shall be transported from the mixing plant to the embankment in clean equipment provided with suitable protective devices in unfavorable weather. The total elapsed time between the addition of water to the mixture and the start of compaction shall be the minimum possible. In no case shall the total elapsed time exceed 60 minutes. Haul time shall not exceed 30 minutes, and compaction should start as soon as possible after spreading. The soil-cement mixture is not to be left undisturbed for longer than 30 minutes at any time during compaction operations. This time may be reduced by the civil or geotechnical engineer when the air temperature exceeds 90 degrees Fahrenheit (F) or when there is a breeze or wind which promotes rapid drying of the soil-cement mixture.

The Contractor shall take all necessary precautions to avoid damage to completed soil cement by the equipment and to avoid the deposition of raw earth or foreign materials between layers of soil cement. In stair step construction, temporary ramps are constructed at intervals along the bank to enable trucks to reach the layer to be placed. These temporary ramps should have a minimum 2 feet thickness of material to protect the edge of the previous lift from truck traffic. There is also a requirement, where streambeds are dry, for ramps to be spaced to allow egress from the channel in case of a flood. These are constructed at 45-degree angles and spaced about 300 to 400 feet apart. Where ramps are constructed over soil cement that is not to grade, all foreign materials and the uppermost one inch of the previously placed soil cement mixture must be removed prior to continuation of the soil cement construction.

G. Placing

The mixture shall be placed on the moistened subgrade embankment, or previously completed soil-cement with spreading equipment that will produce layers of such widths and thicknesses as are necessary for compaction to the required dimensions of the completed soil-cement layers. The compacted layers of soil-cement shall not exceed 9 inches in thickness and shall not be less than 6 inches. Each successive layer shall be placed as soon as practicable after the preceding layer is completed and approved by the geotechnical engineer.

All soil-cement surfaces that will be in contact with succeeding layers of soil cement shall be kept continuously moist by fog spraying until placement of the subsequent layer, provided that the Contractor will not be required to keep such surfaces continuously moist for a period longer than 7 days.

When the time between completion of compaction on a layer and start of placement of the next layer is greater than two hours, the Contractor shall scarify the surface to a depth of 1 inch at a maximum spacing of 12 inches. The Contractor shall clean off the scarified surface thoroughly by power brooming or other approved methods prior to proceeding. The broomed surface shall then be thoroughly moistened over its entire surface before the next layer of soil cement is placed.

Placement of stair-step sections may need to be limited to a maximum of 4 feet height in a single shift to avoid instability producing bulging in the outer face from the surcharge weight of material and equipment above. Soil cement shall not be mixed or placed when the air temperature is below 45º F, unless the air temperature is at least 40º F and rising. Soil cement shall not be placed on a frozen foundation, or if the soil to be processed is frozen, or if weather conditions are such that the material being processed cannot be completely compacted and protected before the onset of damaging weather (such as overnight lows below 40º F, cold fronts, rainstorms, etc.). The use of accelerators or antifreeze compounds will not be allowed, unless otherwise specified. The temperature of fresh soil-cement shall not be allowed to drop below 32º F for a period of seven 7 days after placement. If temperatures are expected to be below 45º F, the Contractor's method for protection shall be approved by the Engineer prior to placement of any soil cement.

H. Compaction

At the start of compaction, the mixture shall be in a uniform, loose condition, throughout its full depth. Its moisture content shall be as specified by the geotechnical engineer at start-up. Soil cement shall be uniformly compacted to obtain a relative compaction of at least 95 percent of the maximum density as determined by ASTM D1557. The in-place density shall be determined by using Nuclear Test (ASTM D 6938) or Sand Cone (ASTM D 1556) methods. Wheel rolling, only using hauling equipment, shall not be an acceptable method of compaction.

In stair-stepped soil-cement application, compaction of the outer edge of the layer is usually not necessary from the standpoint of structural integrity. However, uniform edges provide a better appearance and allow for easier emergency egress from streambeds. Sharp edges reduce wave runup but increase roughness. Edge compaction can be accomplished by hand tampers or through the use of some type of edge support during compaction.

No section shall be left undisturbed for longer than 30 minutes during compaction operation. Compaction of each layer shall be done in such a manner to produce a dense surface, free of compaction planes, in not longer than 1 hour from the time water is added to the mixture. Whenever the contractor's operation is interrupted for more than 2 hours, or prior to commencement of the day's construction, one of the following procedures may be chosen to provide binding of layers:

1. Sprinkle the top of a completed layer with dry cement, about 1 to 1½ pounds per square yard. The cement should be applied to a clean surface, then moistened to form a slurry no more than 15 minutes (or less, at field/soils engineer's discretion and depending upon such factors as ambient temperature and humidity) prior to placement of the next layer.

2. Apply a neat cement slurry at an approximate 1:1 ratio over a clean surface no more than 15 minutes (or less, at field/soils engineer's discretion and depending upon such factors as ambient temperature and humidity) prior to placement of the next layer.

Prior to commencement of the day's construction, the surface of the soil cement must be cleaned to remove all debris and waste soil cement, and a smooth surface as described herein provided to ensure proper bonding with succeeding soil cement layers. The final two (vertical) feet of soil cement shall be placed without interruption to prevent lamination.

If the surface of a layer of soil-cement has been rutted or compacted unduly by hauling or other equipment, the Contractor shall scarify and re-compact such surfaces within 2 hours of the addition of water to the cement. When required to maintain uniformity of the layer surface, blading in connection with compaction operations shall be employed. If blading is required, raw unmixed soil shall not be bladed onto the mixed soil cement. When greater than 2 hours has occurred from the time water was added to the cement, the damaged soil-cement shall be removed in a manner and to the extent approved by the Engineer.

The soil cement layer placed, if considered defective, shall be removed and replaced in accordance with these guidelines, when any one of the following condition occurs:

a. Compaction operations are interrupted for any reason prior to the completion of compaction and the soil cement mixture is left undisturbed for more than 30 minutes.

b. The soil cement mixture becomes excessively wet prior to completion of compaction, so that the moisture content exceeds the specified limits.

c. The compacted soil cement does not meet the density and moisture requirements; except that when the moisture is lower than required, the soil cement mixture may be reworked, thoroughly mixed, and compacted within the time limits stated previously.

d. The finished surface is rough or below grade such that a thin "scab" section would be required to smooth the surface or bring the surface to grade.

I. Finishing

After compaction, the soil cement shall be further shaped, if necessary, to the required lines, grades and cross sections and rolled to a reasonable smooth surface. The face of soil cement above the riverbed shall be trimmed within 48 hours of placement. In no case shall trimming occur any more frequently than at the end of each day’s placement.

The following requirements shall apply regarding the finishing of soil-cement surface.

During the compaction operations for the uppermost lift of soil-cement, shaping will be required to obtain the required surface and cross section. During shaping operations, it may be necessary to lightly scarify and broom-drag the surface in order to remove ridges or depressions in excess of the permitted tolerance specified herein. The resulting surface shall then be rolled with a smooth steel-wheel roller in nonvibratory mode, weighing not less than 10 tons, or pneumatic tire rollers, or both. The final rolling shall be done by a smooth steel-wheel roller. Several applications of water may be required to keep the surface at the proper moisture content, as directed by the engineer, during the finishing operation. Water shall be applied by the pressure spray bar method. Compaction and finishing shall be done in such a manner to produce a smooth dense surface, free of surface compaction planes, cracks, ridges, or loose material no longer than 2 hours after completion of mixing. Immediately after rolling, the surface of the course shall be tested for trueness, transversely and longitudinally. The uppermost layer including access roads and all pertinent soil cement structures shall be constructed within a vertical tolerance of 0.1 foot per elevations shown on the development plan. Surface finishing shall be completed in daylight hours.

The edges on stair-stepped soil cement applications shall be finished by cutting back the uncompacted edges, by using special rounded attachments on compaction equipment, and by leaving sacrificial uncompacted edge material in place to be eroded later.

Any portion of this course which has a density less than the specified shall be corrected or removed and replaced to its full depth to meet the specifications.

J. Curing

Temporarily exposed surfaces shall be kept moist as set forth herein. Care must be exercised to ensure that no curing material, other than water, is applied to the surfaces that will be in contact with succeeding layers. Permanently exposed surfaces shall be kept in a moist condition for 7 days, or they may be covered with some suitable curing material, subject to the engineer's approval. Any damage to the protective covering within 7 days shall be repaired to the satisfaction of the engineer.

The finish exposed surfaces shall be cured based on one of the following methods:

Method 1: Concrete curing compound conforming to ASTM C 309 of the type specified shall be applied at a rate of not less than 1 gallon per 150 square feet of surface using constantly agitating, pressure spray equipment. It shall form a uniform, continuous, adherent film that shall not check, crack, or peel.

The surfaces of each section of soil cement to be treated with curing compound shall be moistened with a light spray of water immediately after the section has been compacted. As soon as the surface film of moisture disappears, but while the surface still has a damp appearance, the curing compound shall be applied. Special care shall be implemented to insure ample coverage with the compound at edges, corners, and around rough spots. After application of the curing compound has been completed and the coating is dry to the touch, any required repair of the soil cement surfaces shall be performed. All curing compound or other foreign substances shall be removed from the area prior to receiving additional soil cement to ensure a clean bonding surface. Each repair, after being finished, shall be moistened and coated with curing compound in accordance with the foregoing requirements.

Method 2: Curing moisture shall be maintained by sprinkling, flooding, fog spraying, at least three times each shift and three times per day on non-workdays. Water and/or covering shall be applied in such a manner that the soil cement surface is not eroded or otherwise damaged.

Method 3: Waterproof paper or plastic sheeting shall be used to completely cover the soil-cement and prevent moisture loss. Adjoining sheeting shall be overlapped at least 1 foot and weighted or taped to prevent moisture loss at joints. Sheeting shall be anchored sufficiently to prevent displacement due to wind.

The curing process for all methods shall be maintained for 7 days. Any curing compound that is removed from the surface or damaged within 7 days after application shall be repaired immediately. The Contractor shall have all equipment and materials required for curing at the site ready for use before starting soil-cement placement activities.

K. Construction Joints

At the end of each day's work, or whenever construction operations are interrupted for more than 2 hours, a transverse construction joint shall be formed by cutting back into the compacted soil-cement to form a full-depth vertical face in the last lift placed.

L. Maintenance

The Contractor shall be required to maintain the soil cement in good condition until all work is completed and accepted. Maintenance shall include immediate repairs of any defects that may occur. Faulty work shall be replaced for a full depth of the layer.

II. Inspection and Testing

The geotechnical engineer and the project design engineer, with the assistance and cooperation of the contractor, will make such inspections and tests as deemed necessary to ensure the conformance of the work to the required specification. These inspections may include, but will not be limited to: (1) field survey certification of top and toe of soil cement bank protection as to ensure the contractor meets the vertical tolerance specified in the plans, (2) the taking of the test samples of the soil cement and its individual components at all stages of processing and after completion, and (3) the close observation of the operation of all equipment used on the work. Only those materials, machines, and methods meeting the project requirements shall be approved by the geotechnical engineer.

All testing of soil cement or its individual components, unless otherwise provided specifically in the contract documents, shall be in accordance with the latest applicable California, ASTM, or AASHTO specifications in effect as of the date of construction.

Testing for proper compaction shall be done on at least every other layer of compacted soil cement at any location chosen by the inspector. If the layer being tested does not pass the minimum 95 percent of the maximum density requirement as determined by ASTM D1557, it must be reworked until it passes or be removed and replaced by compacted materials. The contractor shall not continue placing layers of soil-cement on any layer that has failed the compaction tests until such time as that lift has been reworked, retested, and passed as to meeting density requirements.

The initial acceptance of material shall in no way preclude further examination and testing at any time during the course of construction or subsequent warranty period or if the geotechnical engineer suspects that the material is no longer properly represented by the acceptance sample. The acceptance at any time of any material incorporated into the work shall not bar its future rejection if it is subsequently found to be defective in quality or uniformity.

A. Mix Design

The estimated mix design for this project shall be XXX percent by dry weight. The percent of cement to be used in the mix shall be calculated to be the weight of cement divided by the total weight of the dry compacted soil cement. The actual mix design used on this project shall be determined by laboratory tests, on stockpiled materials excavated from designated areas of the site, in accordance with the requirements provided herein, per approval of the geotechnical engineer.

B. Stockpiling of Aggregate

Soil stockpile shall be constructed on level, firm ground free of brush, trees, stumps, roots, rubbish, debris, and other objectionable or deleterious materials and shall be located so as to provide a distance of not less than 50 feet from the outside bottom edge of the conical stockpile build up under the processing plant conveyor or any other existing stockpile. The stockpile shall be constructed in layers, each layer not exceeding 2 feet in thickness. Ramps formed for stockpile construction shall be of the same material as that being stockpiled and will be considered a part of the stockpile. Before steepening a ramp, any contaminated surface material shall be removed. The total height of the stockpile shall not exceed 15 feet, or the reach of the equipment employed to remove material for sampling and utilization, whichever is less.

Stockpiled material should be thoroughly mixed throughout its depth, width, and length before utilization. The material shall be homogenous and uniform in color, gradation, and moisture throughout.

During construction of stockpiles to be utilized in the production of soil-cement, the contractor will be solely responsible for monitoring the quality and uniformity of the material being placed therein. To assure conformance with the gradation requirements specified for ideal soil material, the testing laboratory shall sample and test at frequent enough intervals to demonstrate such compliance to the geotechnical engineer.

Stockpile sampling will be done by the geotechnical engineer after the required amount of soil for the entire soil-cement job has been excavated and stockpiled. After the stockpile has been sampled and approved, no material will be added to it without the approval of the geotechnical engineer.

Upon completion of the stockpile, the Contractor shall notify the geotechnical engineer in order to allow for verification of the soil-cement mix design determined during design from random site sampling. The contractor shall provide the manpower and equipment necessary to sample the stockpile in accordance with the following procedure:

Under the direction of the geotechnical engineer, the contractor shall use a front-end loader to excavate a face for the full height of the stockpile, extending into the stockpile a distance specified by the geotechnical engineer, at four different locations around the perimeter of the stockpile. The front-end loader shall then be used to channel the total excavated face at each location from the bottom to the top in one operation, and the material obtained shall be dumped on the ground in piles. The geotechnical engineer or its representative will then sample each of the four piles by channeling it with a hand shovel at four locations equally spaced around the perimeter.

Approval of a stockpile shall not relieve, in any degree, the full responsibility of the contractor to furnish in its final position, a material conforming to all the specification requirements.

C. Utilization of Stockpiles

Stockpiles of material may be used for any item (i.e. soil-cement or compacted fill) for which it is acceptable. Material removal from accepted stockpiles for project utilization shall be by side excavation for the full height of the stockpile unless otherwise approved, in writing, by the geotechnical engineer.

Unless otherwise stipulated, the contractor shall provide and pay for all supplies, materials, labor, water, tools, equipment, light, power, transportation, and other facilities necessary for execution and completion of the project. Unless otherwise specified, all materials and supplies shall be new and of the best quality. The contractor, if required, shall furnish satisfactory evidence as to the kind and quality of supplies and materials. It shall be the responsibility of the contractor and/or materials supplier to maintain in-house quality control of processed materials.

The contractor shall submit a request for materials testing by geotechnical engineer 48 hours in advance. Stockpile(s) shall be completed and approved at least 10 days prior to start of soil cement production, mix design shall then be performed by the geotechnical engineer to determine job mix proportions.

D. Test Procedure for Determination of Cement Required for Soil-Cement Mixtures and Cylinder Specimens Prepared During Construction

The compressive strength of molded specimens at varying cement contents shall be determined by ASTM D1633 and shall be used to determine the percentage of Portland cement required in developing soil cement mixtures. (ASTM D1633 shall be modified to use ASTM D1557 to prepare the molded specimens.)

E. Soil-Cement Compressive Strength

The design requirements for the soil-cement bank protection shall be such that the running average of 7-day compressive strengths for field specimens over any 3 consecutive days of construction shall be

750 psi. The minimum 7-day compressive strength for any single field specimen shall not be less than 450 psi. if the running average of the 7-day compressive strengths over any 3 consecutive days of construction is below 750 psi, the engineer shall analyze the effect on the service life of the lining. Should it be determined that the service life of the lining is inadequate, the affected area of the lining shall be removed and replaced with acceptable soil cement. Testing of in-place soil cement by boring or other methods shall be performed at the expense of the contractor and as directed by the engineer for test samples that fall outside of the established parameters.

Furthermore, if the result of 7-day compressive strength samples for 3-day period are fallen below the set criteria several times, the geotechnical engineer will perform its investigation and may adjust the cement content accordingly to obtain the desired compressive strength result, if required. In this case, the amount of cement thus determined by laboratory testing shall continue to be monitored throughout the life of the project with modification as required to meet existing field conditions.

F. Schedule of Geotechnical Testing

Geotechnical engineer shall oversee construction of the stockpile, including any engineering necessary to bring the oversize material and fines content to acceptable levels. Geotechnical engineer shall ensure that the stockpile contains no material retained on a 3-inch sieve, nor any organic or deleterious material, before approving it for use in construction of the soil cement bank protection. Additionally, at job start up, the sand equivalent of the stockpiled material shall be determined, using California test method 217 or ASTM D2419 (i.e. SE $\geq$15).

Geotechnical engineer shall determine the following information for the stockpile at the beginning of every day that construction of the soil-cement bank protection is to occur prior to commencement of construction for that day:

a) Perform sieve analysis of stockpiled material (passed through mixing apparatus) prior to addition of cement, in accordance with the methods laid out in ASTM D421 and ASTM D422.

b) Perform moisture content determination for stockpiled material prior to addition of cement, in accordance to one of the following methods: AASHTO T 217, ASTM D3017, AASHTO T 239, or ASTM D4643.

c) Perform modified proctor test (ASTM 1557) to determine required in-place compaction. Soil-cement must be compacted to 95 percent of maximum dry density as determined by this test.

Geotechnical engineer shall perform the following at reasonable intervals or at least every 500 cubic yard of soil cement placed, such that a minimum of four samples are obtained twice a day per every day of soil cement construction:

a) Take in-place samples after soil cement mixture is placed, but prior to blading and compaction.

b) Record station and elevation from which samples were taken.

c) Perform moisture content determination on samples, in accordance with the test method selected to determine moisture content for stockpiled material.

d) Determine maximum dry density and optimum moisture of soil cement mixture in accordance with ASTM D1557.

e) Prepare four cylinders for compressive strength testing (1, 3 and 7 days and stand-by specimen). These cylinders shall be prepared in accordance with ASTM D1557.

f) Cap and seal samples and send to lab for compressive strength testing at (1, 3 and 7 days and stand-by specimen), in accordance with ASTM D1633. Samples should be kept moist in an airtight container prior to transport to lab to ensure proper curing.

g) Standby specimen should be tested for 28-day compressive strength for every 5000 cubic yard of soil-cement placed in accordance with ASTM D1633.

Geotechnical engineer shall verify field compaction to 95 percent of maximum density and moisture content utilizing either ASTM D1556 or ASTM 6938 on every other layer of soil cement placed.

In addition, the following inspections shall be performed by geotechnical engineer to verify the cement content utilized for soil cement construction:

a) perform numerical calculations using scale values from the pugmill to determine cement content and then compare to target cement and coordinate with soil-cement pugmill operator to adjust as necessary.

b) if there is inconsistency in the cement content being used for soil cement construction, at the direction of the geotechnical engineer, the following test shall be performed:

Heat of neutralization test may be used to determine cement content of freshly mixed soil cement used in preparing samples, in percent by dry weight, in accordance with ASTM D5982-96. This test, which can be conducted in the field, provides a means for reliably determining the cement content of soil cement in approximately 15 to 20 minutes. The measured result should be compared to target cement content and coordinated with soil cement pugmill operator to adjust, as necessary.