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Geotechnical Investigation Report

Flint Canyon Wash Trail
South of Westbound Foothill 210 Freeway Berkshire Place On-Ramp
La Cañada Flintridge, California

Prepared for:

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December 15, 2020
Project No.: 200376.1

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December 15, 2020
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Mr. Mark Krebs
President
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Subject: Geotechnical Investigation Report
Flint Canyon Wash Trail
South of Westbound Foothill 210 Freeway Berkshire Place On-Ramp
La Cañada Flintridge, California

Dear Mr. Krebs,

In accordance with your request and authorization, we are presenting the results of our geotechnical investigation for the Flint Canyon Wash Trail project located south of the westbound Foothill 210 Freeway (I-210) Berkshire Place on-ramp in La Cañada Flintridge, California. The purpose of our investigation has been to evaluate the subsurface conditions at the site and to provide geotechnical engineering recommendations for restoration of the Flint Canyon Wash Trail.

We note that the recommendations presented in this report are based on assumptions stated herein. Should conditions encountered during development differ from those assumed, or should the proposed development change, our recommendations may need to be modified accordingly. This report was prepared in accordance with the requirements of the 2020 County of Los Angeles Building Code (2020 CLABC), the 2019 California Building Code (2019 CBC) and ASCE 7-16 (ASCE 2017).

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

Respectfully submitted,
TWINING, INC.

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1. INTRODUCTION

This report presents the results of the geotechnical investigation performed by Twining, Inc. (Twining) for the Flint Canyon Wash Trail project located south of the westbound Foothill 210 Freeway (I-210) Berkshire Place on-ramp in La Cañada Flintridge, California. The purpose of this investigation is to characterize the existing subsurface conditions and to provide geotechnical engineering recommendations for the Flint Canyon Wash Trail restoration project.

2. SITE DESCRIPTION AND PROPOSED IMPROVMENTS

The Flint Canyon Wash is on the west of the I-210. The Flint Canyon Wash Trail is between the canyon wash and the I-210. The project limits are approximately 2,000 feet of the trail approximately between Berkshire Place on the north and the trail entrance at Oak Grove Drive on the south in La Cañada Flintridge, California, as shown in Figure 1, Site Location Map. The approximate site coordinates are latitude 34.189972°N longitude 118.18177°W at the north end and latitude 34.187198°N longitude 118.18024°W at the south end. The site is located on the Pasadena, California 7½-Minute Quadrangle, based on the United States Geological Survey (USGS) topographic map (USGS 2018).

The site plan is presented on Figure 2, Site Plan and Boring Location Map. The project will consist of improvements to mitigate continued erosion and undercutting of the slope supporting the Flint Canyon Trail. There is an approximately 1000-foot-long section of the trail that has suffered from significant erosion and will be the focus of the project. Based on our communications with Pacific Advanced Civil Engineering, Inc. (PACE), proposed improvements will consist of construction of retaining walls consisting of gabion units on the east bank of the canyon wash to protect the trail within the project limits. The gabions units will consist of 3-foot by 3-foot by 6-foot gabion cages containing angular rock fill.

3. SCOPE OF WORK

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

3.1. Literature Review

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

3.2. Pre-Field Activities and Field Exploration

Before starting our exploration program, we performed site geologic mapping and a site reconnaissance to observe the general surficial conditions at the site, to select field exploration locations, and to plan field logistics including health and safety. After exploration locations were delineated, Underground Service Alert was notified of the planned locations a minimum of 72 hours prior to excavation. We also obtained a permit from the City of Pasadena and a permit from the County of Los Angeles Department of Public Works to perform the field exploration. Copies of the permits are attached to the end of Appendix A of this report.

The field exploration was conducted on July 27 and 28, 2020 and consisted of drilling, testing, sampling, and logging five exploratory borings (B-1 through B-5). The borings were advanced to approximately 16.5 feet to 31.5 feet below the existing ground surface (bgs), which was at least 1.5 feet into older alluvium. Drilling was performed using a limit access track-mounted drill rig (Mini Mole) equipped with 6-inch-diameter solid-stem-augers. The approximate locations of the borings are shown on Figure 2 – Site Plan and Boring Location Map.

Drive samples of the subsurface materials were obtained from the borings using a Standard Penetration Test (SPT) sampler without liners and a modified California split spoon sampler. The samplers were driven using a 140-pound automatic hammer falling approximately 30 inches. The blow-counts to drive the samplers were recorded, and subsurface conditions encountered in the borings were logged by a Twining field engineer. Samples obtained from the borings were transported to Twining's geotechnical engineering laboratory for examination and testing.

Upon completion of drilling, sampling and testing, the borings were backfilled with cement-bentonite grout by the drilling subcontractor. The surface of the exploratory borings was repaired to match existing conditions.

Detailed descriptions of the borings and soils encountered during drilling are presented in Appendix A.

3.3. Geotechnical Laboratory Testing

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

- In-situ moisture and density;
- #200 Wash;
- Sieve analysis;
- Atterberg limits;
- Expansion index;
- Direct shear; and
- Maximum dry density and optimum moisture content;
- Unconsolidated undrained (UU) shear strength; and
- Corrosivity.

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

3.4. Engineering Analyses and Report Preparation

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

- Evaluation of site geology and geologic conditions as it pertains to stability of the slopes adjacent to the wash;
- Evaluation of general subsurface conditions including soil formation, description of cobble sizes (if encountered), type, distribution, and engineering characteristics of subsurface materials at the site;

- Evaluation of current groundwater conditions at the site and potential dewatering or other impacts on design and construction;
- Evaluation of slope stability of existing and proposed new slopes created as part of the improvements;
- Recommendations for gabion retaining walls, including bearing capacity, lateral earth pressures, and lateral resistance; and
- Evaluation of the excavation conditions and the potential for difficult excavation;
- Evaluation of allowable temporary excavation side slopes;
- Suitability of on-site materials for use engineered fill material;
- Recommendations for shoring and retaining wall pressures, including lateral loads and seismic pressures for walls taller than 6 feet;
- Evaluation of the potential for on-site soil to corrode buried steel and concrete objects; and
- Development of general recommendations for earthwork, including site preparation and excavation, and requirements for placement of trench backfill.

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

4. GEOLOGY AND SUBSURFACE CONDITIONS

The geology and subsurface conditions at the site are based on the results of our field investigation (Appendix A) and our review of published geologic maps (Figure 3 – Regional Geologic Map).

4.1. Site Geology, Subsurface Conditions and Geologic Cross Sections

According to the geologic map of Dibblee (1989) and our field investigation, the site is underlain by alluvium (map symbol: Qa) underlain by older alluvium. The alluvium in boring B-2 is overlain by approximately 5 feet of artificial fill. A portion of the geologic map is reproduced as Figure 3 – Regional Geologic Map.

A generalized description of the subsurface conditions encountered in the borings drilled at the site is provided below. Detailed descriptions of the earth materials encountered in the exploratory borings are presented in Appendix A. Cross sections illustrating the geologic conditions and results of the geologic mapping are presented on Figure 4A through Figure 4G. Locations of the cross sections are shown on Figure 2.

4.1.1. Artificial Fill

Boring B-2 encountered approximately 5 feet of artificial fill consisting of silty sand with approximately 10% gravel. No fill was encountered in other borings (B-1 and B-3 through B-5). No documentation for the placement and compaction of the fill is available for our review, and the fill is considered undocumented.

4.1.2. Alluvium

Alluvium underlies the site to approximately 5 feet to 15 feet bgs, depending on locations. The alluvium consisted primarily of dense to very dense silty sand and occasionally of medium dense silty sand and hard sandy silt.

4.1.3. Older Alluvium

Older alluvium underlies the alluvium. The older alluvium consisted primarily of hard silt/clay with varying amounts of sand and occasionally into dense silty sand.

4.2. Groundwater Conditions

The historic high groundwater level at the project site is not well defined on the map of the historical high groundwater in the CGS Seismic Hazard Zone Report 14 for the Pasadena quadrangle (CGS, 1998, revised 2006). During drilling, groundwater was encountered at 24 feet bgs corresponding to approximately 1,049 feet above mean sea level (msl) in boring B-2. Other borings extended to 16.5 feet bgs did not encounter groundwater. At the time of drilling, a few inches of water were observed in the Flint Canyon Wash, and the elevation of the bottom of the wash adjacent to boring B-2 is approximately 1,053 feet msl. It suggests that groundwater flows from the wash toward the trail slope.

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

5. GEOLOGIC HAZARDS AND SEISMIC DESIGN CONSIDERATIONS

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

5.1. Active Faulting and Surface Fault Rupture

The site is not located within or adjacent to an Alquist-Priolo Earthquake Fault Zone (EFZ) (CGS 2016). The boundary of the closest Alquist-Priolo EFZ is located approximately 4.5 miles south of the site associated with the Los Angeles fault zone. The boundary of the next closest Alquist-Priolo EFZ is located approximately 5.7 miles northwest of the site associated with the Burbank fault zone. Known active faults closest to the site are the Sierra Madre fault approximately 1.3 miles to the north and the Verdugo fault approximately 2.8 miles to the southwest. Based on our review of geologic and seismologic literature and our site evaluation, it is our opinion that the likelihood of surface fault rupture at the site during the life of the project is low.

5.2. Liquefaction Potential, Lateral Spread, and Seismic Settlement

The CGS Seismic Hazards Zones Map indicates that the project site is at the edge of an area subject to liquefaction (Figure 5). Our field investigation indicates that site materials encountered during the field investigation consist primarily of dense to very dense alluvium overlying older alluvium consisting of hard silt and clay and dense silty sand. The medium dense silty sand will be removed as part of this project. It is our opinion that liquefaction potential at the project is considered very low.

The potential of seismic settlement and liquefaction-induced lateral spread at the site is considered remote because the site has very low liquefaction potential.

5.3. Landslide

The site is not within an area with the potential for earthquake-induced landslide (Figure 5). Based on our review of geologic and seismologic literature, our geologic mapping on-site, it is our opinion that the likelihood of earthquake-induced landslide at the site during the life of the project is considered low.

5.4. Site Class for Seismic Design

Based on the site subsurface conditions (Section 4.1 and Appendix A), average field standard penetration resistance for the upper 100 feet of soil profile is estimated to be greater than 50 blows per foot. The site may be classified as Site Class C for seismic design according to Chapter 20 of ASCE 7-16.

5.5. Deaggregated Seismic Source Parameters

We performed a seismic hazard de-aggregation analysis for the peak ground acceleration with a probability of exceedance of 2% in 50 years. The analysis used the USGS Unified Hazard Tool based on the 2014 USGS seismic source model. The results of the analysis indicate the controlling modal moment magnitude Mw and fault distance R are 7.7 and 3.6 miles (5.75 km), respectively.

5.6. Mapped CBC Seismic Design Parameters

Our recommendations for seismic design parameters have been developed in accordance with the 2019 CBC and ASCE 7-16 (ASCE 2017) standards for Site Class C conditions. Table 1 presents the seismic design parameters for the site based on coordinates latitude 34.189972°N longitude 118.18177°W at the north end and latitude 34.187198°N longitude 118.18024°W at the south end.

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Table 1 – 2019 California Building Code Seismic Design Parameters for Design Based on Exception 2 in Section 11.4.8 of ASCE 7-16

Design Parameters	Value
Site Class	C
Mapped Spectral Acceleration Parameter at Period of 0.2-Second, S_s (g)	1.972
Mapped Spectral Acceleration Parameter at Period 1-Second, S_1 (g)	0.733
Site Coefficient, F_a	1.2
Site Coefficient, F_v	1.4
Adjusted MCE_R[1] Spectral Response Acceleration Parameter, S_{MS} (g)	2.366
Adjusted MCE_R[1] Spectral Response Acceleration Parameter, S_{M1} (g)	1.026
Design Spectral Response Acceleration Parameter, S_{DS} (g)	1.578
Design Spectral Response Acceleration Parameter, S_{D1} (g)	0.684
Risk Coefficient C_{RS}	0.898
Risk Coefficient C_{R1}	0.897
Peak Ground Acceleration, PGA_M[2] (g)	1.019
Seismic Design Category[3]	D
Long-Period Transition Period, T_L (seconds)	8
$Ts = S_{D1} / S_{DS}$	0.434

Notes: [1] Risk-Targeted Maximum Considered Earthquake.
[2] Peak Ground Acceleration adjusted for site effects.
[3] For S_1 greater than or equal to 0.75 g, the Seismic Design Category is E for risk category I, II, and III structures and F for risk category IV structures.

6. GEOTECHNICAL ENGINEERING RECOMMENDATIONS

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

6.1. General Considerations

Based on our evaluation of the site conditions, it is our opinion that the trail can be improved using a gabion retaining wall. Design of the wall (e.g., number of rows, depth, and height of gabion units) should consider the global stability of the trail slope and internal stability of the gabion wall. We recommend that gabion wall should extend at least 2 feet below the bottom of scour of the Flint Wash or 2 feet into the underlying older alluvium, whichever is deeper, to prevent scour and erosion from undermining the wall and to provide sufficient bearing capacity for the wall. Evaluation of the scour depth is beyond the scope of this investigation. A scour depth of 5 feet is assumed in our analysis, based on our communications with PACE. Additionally, the gabion wall should extend to a sufficient depth for global slope stability, as shown in Table 4 discussed in Section 6.8.5.

The following sections present our geotechnical recommendations for the gabion wall internal stability evaluation by the wall designer, our evaluation of global stability of the trail slope with and without the gabion wall, as well as recommendations for earth work.

Our recommendations are based on our understanding of the site conditions, subsurface conditions encountered during our field exploration, the results of laboratory testing on soil samples taken from the site, and our engineering analyses. If the site conditions or subsurface conditions during construction differ substantially from those encountered during our field explorations, then our recommendations would be subject to revision based on our evaluation of the differences.

6.2. Soil Collapse and Expansion Potential

Based on our evaluation of subsurface conditions encountered during our field exploration and results of and laboratory tests, site soils have low collapse potential and very low expansion potential.

6.3. Corrosive Soil Evaluation

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

Corrosive soil may be defined as the soil has minimum electrical resistivity less than 1,000 ohm-centimeters, or chloride concentration greater than 500 parts per million (ppm), or sulfate concentration in soils greater than 2,000 ppm, or a pH less than 5.5 (e.g., based on the County of Los Angeles criteria or the California Department of Transportation criteria).

6.3.1. Reinforced Concrete

Laboratory tests indicate that the soil has less than 100 ppm or 0.01% of water soluble sulfate (SO_4) by weight. Based on ACI 318, concrete in contact with the site soils will have a sulfate exposure class S0. As a minimum, we recommend that Type II cement and a water-cement ratio of no greater than 0.50 be used on the project.

Test results indicate that the soil has less than 81 ppm of water soluble chlorides by weight and the potential is negligible for chloride attack of reinforcing steel in concrete structures and pipes in contact with soil. However, if needed, a corrosion specialist may be consulted for protection from chloride attack.

6.3.2. Buried Metal

A factor for evaluating corrosivity to buried metal is electrical resistivity. The electrical resistivity of a soil is a measure of resistance to electrical current. Corrosion of buried metal is directly proportional to the flow of electrical current from the metal into the soil. As resistivity of the soil decreases, the corrosivity generally increases. Test results indicate the site soils have a minimum electrical resistivity value of 1,800 ohm-centimeters. Based on the criteria of the County of Los Angeles and the California Department of Transportation, the soils are not considered corrosive to buried metals.

Correlations between resistivity and corrosion potential published by the National Association of Corrosion Engineers (NACE, 1984) indicate that the site soils are moderately corrosive. Corrosion protection may include the use of epoxy or asphalt coatings. A corrosion specialist should be consulted regarding appropriate protection for buried metals and suitable types of piping if any.

6.4. Earthwork and Site Preparation

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

6.4.1. Site Preparation

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

6.4.2. Temporary Excavations

Unsurcharged temporary excavations less than with vertical sides less than 4 feet high are generally expected stable. Where space is available, temporary, un-surcharged excavation sides over 4 feet in height should be sloped back at 1.5H:1V (horizontal:vertical) or flatter.

The tops of the excavation sides should be barricaded so that vehicles and storage loads are away from the top edge of the excavated slopes with a distance at least equal to the height of the slopes. A greater setback may be necessary when considering heavy vehicles, such as concrete trucks and cranes. Twining should be advised of such heavy vehicle loadings so that specific setback requirements can be established. If the temporary construction slopes are to be maintained during the rainy season, berms are recommended to be graded along the tops of the slopes in order to prevent runoff water from entering the excavation and eroding the slope faces.

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

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

6.4.3. Subgrade Preparation

As discussed in Section 6.1, the proposed gabion retaining wall should extend at least 2 feet below the scour depth of the Flint Wash or 2 feet into the older alluvium, whichever is greater. The depth of the gabion wall should also consider global stability of the trail slope as shown in Table 4 discussed in Section 6.8.5. The gabion units should be placed on a 2-inch-thick crushed aggregate base course overlying the older alluvium.

Undocumented fill and alluvium will be removed to its full depth as a result of foundation excavation. Over-excavation is not required either vertically or laterally. However, the extent and depths of all removal should be evaluated by Twining's representative in the field based on the materials exposed. The exposed excavation bottom should be evaluated and approved by Twining. Prior to placement of any fill, the geotechnical engineer or their representative should review the bottom of the excavation for conformance with the recommendations of this report.

6.4.4. Materials for Fill

In general, on-site soils have very low expansion potential and are considered suitable for use as fill materials. All fill soils should be free of organics, debris, rocks or lumps over 3 inches in largest dimension, other deleterious material, and not more than 40 percent larger than ¾ inch. Larger chunks, if generated during excavation, may be broken into acceptably sized pieces or may be disposed offsite.

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

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

6.4.5. Compacted Fill

Prior to placement of compacted fill, the exposed excavation bottoms should be observed by Twining. Unless otherwise recommended, the exposed bottom should then be scarified to a depth of approximately 6 inches and moisture conditioned, as needed, to achieve generally consistent moisture contents at or near the optimum moisture content. The scarified materials should then be compacted to 90 percent relative compaction in accordance with the latest version of ASTM Test Method D1557.

Fill materials should be moisture conditioned to approximately 2% above optimum moisture content prior to placement. The optimum moisture content will vary with material type and other factors. Moisture conditioning of fill soils should be generally consistent within the soil mass. Continue to place the compacted fill in horizontal lifts of approximately 6 to 8 inches in loose thickness. Prior to compaction, each lift should be watered or dried as needed, mixed, and then compacted by mechanical methods, using multiple wheel pneumatic tired rollers, sheepsfoot rollers, or other appropriate compacting rollers, to a relative compaction of 90 percent as evaluated by the latest version of ASTM D1557. Successive lifts should be treated in a like manner until the desired finish grades are achieved.

The evaluation of compaction by Twining should not be considered to preclude any requirements for observation or approval by governing agencies. It is the contractor's responsibility to notify Twining and the appropriate governing agency when project areas are ready for observation, and to provide reasonable time for that review.

6.4.6. Backfill for Utility Trench

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

At locations where the trench bottom is yielding or otherwise unstable, pipe support may be improved by placing 12 inches of crushed aggregate base (CAB) or crushed miscellaneous base (CMB) as defined in the “Greenbook” Standard Specifications for Public Works Construction (SSPWC).

The trench should be bedded with clean sand extending to at least 6 inches below the bottom of the pipe and one foot over the top of pipe. Pipe bedding as specified in SSPWC can be used. Bedding material should consist of clean sand having a sand equivalent (SE) of 30 or greater. Alternative materials such as ½-inch crushed rock meeting the intent of the bedding specifications are also acceptable. Samples of materials proposed for use as bedding should be provided to the engineer for inspection and testing before the material is imported for use on the project. The onsite sandy materials segregated from the clayey material are suitable for bedding. The pipe bedding material should be placed over the full width of the trench. After placement of the pipe, the bedding should be brought up uniformly on both sides of the pipe and mechanically compacted to reduce the potential for unbalanced loads. No void or uncompacted areas should be left beneath the pipe haunches.

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

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

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

6.4.7. Rippability

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

6.4.8. Construction Dewatering

During our field exploration, groundwater was encountered at approximately 24 feet bgs, which corresponds to the approximately 1 foot above the bottom of Flint Canyon Wash. As the bottom of the gabion wall is expected to extend at least 2 feet below the wash bottom, temporary dewatering is anticipated during construction.

Disposal of pumped water should be performed in accordance with the City of La Cañada Flintridge requirements and/or guidelines of the Regional Water Quality Control Board.

6.5. Gabion Wall Internal Stability

The gabion wall should be evaluated for internal stability including bearing capacity, overturning stability, and sliding stability by the wall designer. For this evaluation, the gabion properties should be based on the evaluation of actual gabion fill. For preliminary analysis and design, the gabions may be assumed to have a unit weight of 150 pounds per cubic foot (pcf) and a friction angle of 40 degrees between gabions. Other geotechnical parameters for gabion wall internal stability evaluation are provided in Sections 6.6 and 6.7 of this report.

6.6. Gabion Wall Foundation Recommendations

Gabion wall should be directly placed on a 2-inch-thick base course overlying the older alluvium as described in Section 6.4. The base course should consist of crushed aggregate base (Caltrans Class 2 aggregate base or crushed aggregate base per Greenbook).

6.6.1. Bearing Capacity and Settlement

In design, a net allowable bearing capacity of 2,660 pounds per square foot (psf) may be used for the founding soils. Locally, a net allowable bearing capacity of 4,000 psf may be used. A total static settlement less than one inch with a differential settlement less than 0.50 inches over 50 feet is estimated for the foundation materials. The majority static settlement is expected to occur at the end of construction.

6.6.2. Lateral Resistance

The total lateral resistance can be taken as the sum of the friction at the base of the gabion wall and the passive resistance in front of the toe of the wall. An allowable coefficient of friction of 0.35 may be used for calculating the based friction. A passive pressure equivalent to a fluid weighing 300 pcf may be used to estimate the allowable lateral passive resistance. The scour depth should be neglected when calculating the passive resistance. The passive resistance value may be increased by one-third when considering transient loads from wind or seismic forces (corresponding to a factor of safety of 1.5).

6.7. Gabion Wall Backfill and Lateral Earth Pressures

Recommendations for gabion wall lateral loads, backfill, and drainage are provided below. Lateral resistance may be based on Section 6.6.2 of this report. The wall should be designed to have a factor of safety of 1.5 for static stability and 1.1 for stability due to transient loads from wind or seismic.

6.7.1. Backfill and Drainage of Walls

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

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

In addition, we recommend geotechnical filter fabric be installed behind the wall to prevent fine-grained soils from migrating into the gabion wall.

6.7.2. Lateral Earth Pressure

Lateral earth pressures are presented below for level and sloping backfill conditions. The recommended design lateral earth pressure is calculated assuming that a drainage system will be installed behind the wall to provide adequate drainage and that external hydrostatic pressure will not develop behind the walls. Where wall backfill does not have adequate drainage, the full hydrostatic pressure should be considered in design.

Gabion walls that have adequate drainage may be designed for the active earth pressure equivalent to a fluid weighing 127K_a pcf, where K_a is Coulomb's active earth pressure coefficient. Based on the Coulomb active earth pressure theory, K_a can be calculated using the following equation.

$$K_a = \frac{\cos^2(\phi - \beta)}{\cos^2(\beta)\cos(\beta + \delta)\left[1 + \sqrt{\frac{\sin(\phi + \delta)\sin(\phi - \alpha)}{\cos(\beta + \delta)\cos(\beta - \alpha)}}\right]^2} \tag{1}$$

where:

ϕ = angle of internal friction of wall backfill = 30 degrees for simplicity;

δ = angle of friction between the wall and the backfill and can be assumed equal to ϕ;

α = angle of wall backfill surface with the horizontal;

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β = angle of the back face of gabion wall with the vertical (positive value if the wall back face inclines away from the wall backfill, and negative value if the wall back face inclines toward the wall backfill.

A vertical surcharge pressure (q) within a 1:1 plane projected from the bottom of the wall distributed over retained soils should be considered as an additional uniform horizontal pressure acting on the wall. This additional pressure can be estimated as approximately qK_a.

The resultant (P_a) of the lateral pressures on the wall is inclined at an angle δ with the plane normal to the wall back face so that the horizontal and vertical components (P_h and P_v) of P_a are as follows:

$$P_h = P_a \cos(\delta + \beta) \quad (2)$$

$$P_v = P_a \sin(\delta + \beta) \quad (3)$$

6.7.3. Seismic Lateral Earth Pressure

Walls retaining more than 6 feet high earth should be designed for seismic lateral earth pressure $127K_{ae}$ pcf instead of the lateral earth pressure $127K_a$ pcf discussed in Section 6.7.2, where K_{ae} is the seismic active earth pressure coefficient. The value of K_{ae} may be estimated using the Mononobe-Okabe method as follows:

$$K_{ae} = \frac{\cos^2(\phi - \beta - \varphi)}{\cos^2(\beta)\cos(\beta + \delta + \varphi)\left[1 + \sqrt{\frac{\sin(\phi + \delta)\sin(\phi - \alpha - \varphi)}{\cos(\beta + \delta + \varphi)\cos(\beta - \alpha)}}\right]^2} \quad (4)$$

where $\varphi = \tan^{-1}(PGA_M/3)$ = 19 degrees, since PGA_M = 1.019 g as provided in Table 1.

The seismic pressure distribution may be considered a triangle with the maximum pressure at the bottom. Additional earth pressures due to vertical surcharge behind the wall should be considered according to the Section 6.7.2 discussions.

6.8. Global Slope Stability

The global stability of the trail slope was evaluated by analyzing several representative cross sections under critical loading conditions. The primary purpose of analysis is to estimate the minimum depth for the gabion wall for global slope stability. This section presents the critical loading conditions, cross sections and material properties, water levels inside and outside the slopes, analysis approach, and results of analysis.

6.8.1. Critical Loading Conditions and Required Minimum Factor of Safety

The global stability analysis was first performed for existing conditions under long term static loading and then for the conditions improved with gabions under several potentially critical loading conditions including:

- end-of-construction static loading,
- long-term static loading,
- rapid drawdown,
- scoured condition under static loading, and
- seismic loading.

The “end-of-construction” case represents the stability of the trail slope immediately following completion of construction. The “long-term” case represents the stability of the trail slope under long term static loading conditions. The minimum required factor of safety is 1.25 for end of construction and 1.5 for long term.

The “rapid drawdown” case represents the stability of the trail slope when the water level outside the slope is lowered significantly and quickly but the water level within the fine-grained soils behind the wall do not respond immediately because of the relatively low permeability of the fine-grained soils. Rather, for some period of time, the water level within the fine-grained soils remains elevated and the outside face of the slopes do not have the buttressing effect of hydrostatic loading acting upon it. The minimum required factor of safety is 1.25 for rapid drawdown.

The “scoured condition under static loading” case represents the stability of the trail slope when there is scour in the canyon wash. It is assumed maintenance will take place after scour occurs, and the condition will be temporarily. The minimum required factor of safety is 1.2.

The “seismic” case represents the stability of the trail under transient seismic loading conditions. According to CGS special publication 117A, slopes that have a pseudo-static factor of safety greater than 1.0 are considered stable if the analysis uses a seismic coefficient derived from the screening analysis procedure of Stewart et al. (2003). Our analysis used a horizontal seismic coefficient of 0.41, which corresponds to a slope deformation of approximately 15 cm or 6 inches, based on the Stewart et al. (2003) procedure. According to Blake (2002), a deformation of 15 cm is appropriate for global stability of the site.

6.8.2. Cross Sections and Material Properties

The cross sections for evaluation were selected based on the slope configuration, height, and existing gradient. Representative cross sections with the greatest height and steepest existing gradient were shown on Figure 4A through Figure 4G. The locations of the cross sections are shown on Figure 2. The lateral extent of the sections was limited to the trail vicinity and did not extend to the I-210, because the stability of the slope in a greater extent is beyond the scope of this investigation.

Because in the stress range along the slip surface the ultimate effective shear strength is smaller than the unconsolidated-undrained (UU) shear strength for the fine-grained soils, the analysis used ultimate effective shear strengths for all soils, except for the seismic case. Seismic loading is rapid and transient in nature, and thus the seismic case used peak shear strength.

Properties of the geologic units used in the analysis were based on results of field exploration and laboratory testing programs. Shear strength parameters used in the analysis are presented in Table 2.

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Table 2 – Shear Strength Parameters

Material	Unit weight	Unconsolidated-Undrained (UU) Shear Strength	Ultimate Shear Strength Parameters		Peak Shear Strength Parameters	
	(pcf)	(psf)	Cohesion (psf)	Friction Angle (degrees)	Cohesion (psf)	Friction Angle (degrees)
Silty Sand Alluvium	125	Not Considered	280	28	330	31
Silt/Clay Older Alluvium	115	1,400	200	28	160	31
Fill	127	Not Considered	150	29	200	29
Gabion	150	Infinite strength is assumed so that the slip surface will not go through the gabion units and global stability may be evaluated				

6.8.3. Water Levels

Water levels used in the analysis varies with loading conditions as shown in Table 3. Our analysis assumed that water levels inside the slopes are the same as outside slopes, except for the rapid drawdown case. When water levels inside and outside the slopes are the same, the low water level conditions are more critical due to the slope face does not have the buttress effect from ponded water. For rapid drawdown, the water level inside the slopes remains high and outside the slopes is assumed at the bottom of the wash. The low water level used in the analysis is assumed at the bottom of the wash and the 100-year flood water level from PACE is assumed for the high-water level.

Table 3 – Water Levels used in Global Stability Analysis

Analysis Case	Water Level Outside Slope	Water Level Inside Slope
Existing Conditions		
Static Loading	at the bottom of the wash	at the bottom of the wash
Improved with Gabions		
End-of-Construction Static Loading	at the bottom of the wash	at the bottom of the wash
Long-Term Static Loading	at the bottom of the wash	at the bottom of the wash
Rapid Drawdown	at the bottom of the wash	100-year flood level
Scoured and under static loading	at the bottom of the wash	at the bottom of the wash
Seismic Loading	at the bottom of the wash	at the bottom of the wash

6.8.4. Analysis Approach

The stability analyses were performed using the computer program Slide version 7.0 (Rocscience, 2019). Slide is a commercially available program that uses the limit equilibrium theory to estimate the factors of safety for earth and rock slopes. The comprehensive formulation of Slide makes it possible to select a variety of methods for computing the factor of safety, and to analyze both simple and complex geometric, stratigraphic, and loading conditions. Spencer's method was used to analyze the stability of the slopes. The method satisfies both force and moment equilibrium, and accounts for inter-slice forces.

Seismic slope stability is analyzed using the pseudo-static analysis procedure in accordance with the general requirements of CGS special publication 117A (CGS, 2008). In this procedure, seismic loading is modeled with a horizontal seismic coefficient of 0.41 applied to the sliding mass. Determination of the seismic coefficient is discussed in Section 6.8.1.

6.8.5. Analysis Results

The global slope stability analysis was performed to iterate the minimum depth for the gabion wall to meet the required minimum factors of safety discussed in 6.8.1. We note that the wall design should also meet the minimum embedment depth requirements discussed in Section 6.4.3.

Results of the analysis are summarized in Table 4 when no scour occurs. Detailed analysis results along with material properties and water levels for each cross section are provided in Appendix C – Global Slope Stability Analysis. As indicated in the table, under existing conditions, the static factors of safety for the trail slope global stability vary from 1.259 to 1.958. With the gabion wall, the factors of safety for the end-of-construction and long-term cases are the same, because the same shear strengths and water level are used as discussed earlier.

Results of analysis assuming 5 feet of sour are shown in Table 5. The minimum depth below the wash bottom required for the gabion wall global stability varies from 7 to 8 feet.

In addition, we analyzed stability assuming a scour depth of 6 feet for Sections RS 20+00 and RS 21+00 and 7 feet for Sections RS 22+00 and 24+00. Results of the analysis are shown in Table 6.

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Table 4 – Results of Global Slope Stability Analysis Assuming No Scour

Analysis Case	Section RS 17+40	Section RS 18+00	Section RS 19+00	Section RS 20+00	Section RS 21+00	Section RS 22+00	Section RS 24+00
Factor of Safety for Slopes under Existing Conditions							
Static Loading	1.263	1.259	1.516	1.320	1.523	1.700	1.958
Factor of Safety for Slopes Improved with Gabions							
End-of-Construction Static Loading	1.604	1.516	1.603	1.506	1.583	1.743	2.117
Long-Term Static Loading	1.604	1.516	1.603	1.506	1.583	1.743	2.117
Rapid Drawdown	1.400	1.320	1.393	1.318	1.395	1.439	1.648
Seismic Loading	1.142	1.010	1.091	1.056	1.153	1.098	1.148
Depth of Gabion Wall below the Wash Bottom							
Minimum Depth (feet)	3	5	8	6	8	2.6	1.2

Table 5 – Minimum Wall Depth Below Bottom of Wash Assuming 5 feet of Scour

Section/Station	**Section RS 17+40**	**Section RS 18+00**	**Section RS 19+00**	**Section RS 20+00**	**Section RS 21+00**	**Section RS 22+00**	**Section RS 24+00**
Factor of Safety for Static Loading with Scour	1.336	1.215	1.267	1.263	1.214	1.225	1.479
Minimum Depth of Gabion Wall below the Wash Bottom (feet)	8	8	8	8	8	7	7

Table 6 – Factor of Safety and Minimum Wall Depth Below Bottom of Wash

Section/Station	**Section RS 20+00**	**Section RS 21+00**	**Section RS 22+00**	**Section RS 24+00**
Depth of Scour (feet)	6	6	7	7
Factor of Safety for Static Loading with Scour	1.247	1.201	1.395	1.485
Minimum Depth of Gabion Wall below the Wash Bottom (feet)	9	9	9	9

6.9. Temporary Shoring

If the project involves excavations that lack sufficient space for sloped excavations, cantilever, tied-back or braced shoring may be considered. However, cantilevered shoring should only be utilized where some deflection is acceptable (away from existing structures and improvements). A tied-back or braced shoring system should be utilized to support adjacent improvements or structures.

For vertical excavations less than approximately 15 feet in height, cantilevered shoring may be used. Where cantilevered shoring is used for deeper excavations, the total deflection at the top of the wall tends to exceed acceptable magnitudes. Shoring of excavations deeper than approximately 15 feet may need to be accomplished with the aid of tied-back earth anchors or internal bracings.

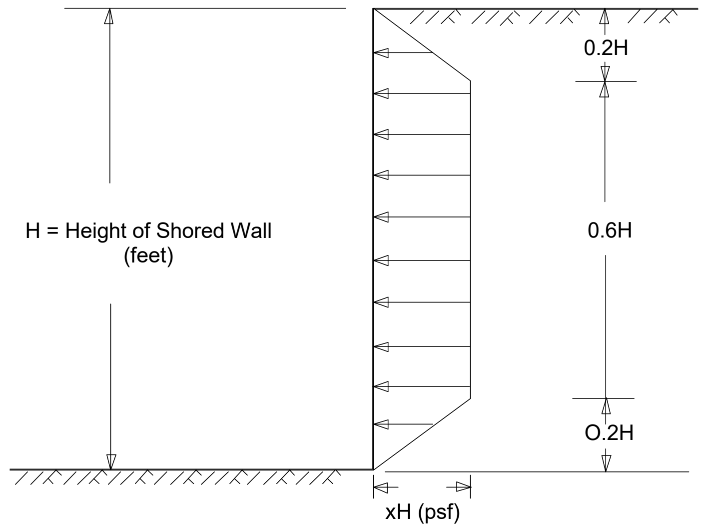
The shoring design should be provided by a California Registered Civil Engineer experienced in the design and construction of shoring under similar conditions. Once the final excavation and shoring plans are complete, the plans and the design should be reviewed by Twining for conformance with the design intent and recommendations. Further, the shoring system should satisfy applicable requirements of CalOSHA.

6.9.1. Temporary Lateral Earth Pressures

For design of cantilevered shoring, a triangular distribution of lateral earth pressure equal to $127K_a$ pcf equivalent fluid pressure, where K_a is determined according to Equation (1) with $\beta = \delta = 0$, provided that retained soils are drained.

Tied-back or braced shoring should be designed to resist a trapezoidal distribution of lateral earth pressure. The recommended pressure distribution is provided in Diagram 1 below.

Any surcharge (live, including traffic, or dead load) located within a 1:1 plane projected upward from the base of the shored excavation, including adjacent structures, should be added to the lateral earth pressures. The lateral contribution of a uniform surcharge load located immediately behind the temporary shoring may be calculated by multiplying the vertical surcharge pressure by K_a for cantilevered shoring and by 0.5 for tied-back or braced shoring. Lateral load contributions of surcharges located at a distance behind the shored wall may be provided once the load configurations and layouts are known. As a minimum, a 250 psf vertical uniform surcharge is recommended to account for nominal construction and/or traffic loads. More detailed lateral pressure and loading information can be provided, if needed, for specific loading scenarios as recognized through the design process.



where x = 104K_a, and K_a is determined according to Equation (1) with $\beta = \delta = 0$, provided that retained soils are drained.

Diagram 1 – Earth Pressure Distribution for Tie-back or Braced Shoring Wall

6.9.2. Soldier Piles

If soldier piles and lagging are opted for the shoring system, the soldier piles should be spaced no closer than 2.5D on center, where D is the diameter of the drilled shaft for the soldier piles. Design of the soldier piles may be based on an allowable lateral passive resistance of 600 pcf equivalent fluid pressure, up to a maximum of 6,000 psf. The upper 1D below the lowest adjacent excavation bottom should be neglected when calculating the lateral resistance. The allowable lateral passive resistance incorporates a factor of safety of 2.

To develop the full lateral resistance, provisions should be taken to assure firm contact between the soldier piles and the soils. The portion of the soldier piles below the lowest excavated level should be concreted to assure firm contact between the pile and surrounding soils. To develop firm contact between the upper portion of the shoring and the retained soils, the upper portion of the soldier pile excavation should be filled with a lean mix concrete or sand-cement slurry.

Soldier piles adjacent to one another be drilled alternately on different days to minimize disturbance to the open excavations. Drilling of the soldier pile shafts can be accomplished using conventional drilling equipment. Caving is anticipated. In the event of soil caving, it may be necessary to use casing and/or drilling mud to permit the installation of the soldier piles. Drilled holes for soldier piles should not be left open overnight. Concrete for piles should be placed immediately after the drilling of the hole is complete. The concrete should be pumped to the bottom of the drilled shaft using the tremie method. Once concrete pumping is initiated, the bottom of the tremie should remain below the surface of the concrete to prevent contamination of the concrete by soil inclusions. If steel casing is used, the casing should be removed as the concrete is placed.

6.9.3. Lagging and Sheeting

To limit sloughing and caving of the earth materials, it is recommended that lagging or gunite be used between soldier piles. Lagging should be installed such that no more than five vertical feet of earth is exposed. Lumber lagging to be left in the ground should be pressure-treated in accordance with Specification C-2 of the American Wood Preservers Association (AWPA). While the soldier piles and anchors should be designed for the full anticipated lateral pressure, the pressure on the lagging will be less due to arching in the soils where lagging is relatively flexible to wales or soldier beams. We recommend that the lagging be designed for a semi-circular distribution of earth pressure where the maximum pressure is 400 pounds per square foot at the mid-line between soldier piles, and 0 pounds per square foot at the soldier piles.

6.9.4. Sheet Piles

If solid sheet piles or a similar continuous shoring system is used, it should be designed using an allowable lateral resistance of 300 pcf equivalent fluid pressure. The passive values should not exceed 4,500 psf. The upper one foot below the lowest adjacent excavation bottom should be neglected when calculating the lateral resistance. The resistance incorporates a factor of safety of 2.

7. LIMITATIONS

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

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

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

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

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

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

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

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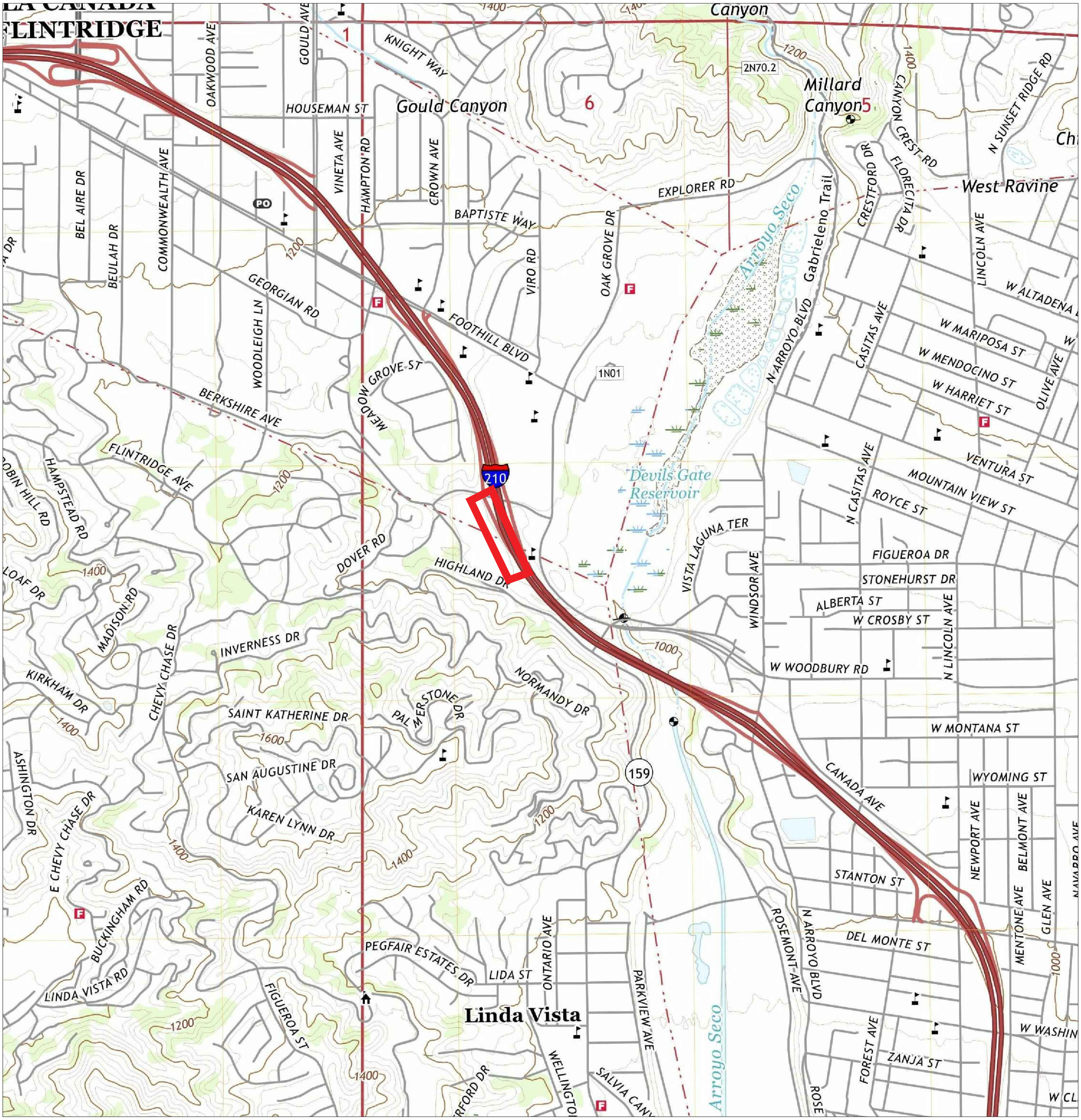
FIGURES



APPROXIMATE LOCATION OF PROJECT

SCALE IN FEET

0 2000 4000

REFERENCE: USGS (2018)

TWINING

SITE LOCATION MAP

FLINT CANYON WASH TRAIL
LA CANADA FLINTRIDGE, CA

PROJECT NO.	REPORT DATE	FIGURE 1
200376.1	December 2020	

NOTE: ALL DIMENSIONS AND LOCATIONS ARE APPROXIMATE. ELEVATIONS ARE FEET (MSL).

TWINING

LEGEND

B-1 TD=16.5' — APPROXIMATE LOCATION OF BORING BY TWINING TOTAL DEPTH IN FEET

APPROXIMATE GEOLOGIC CONTACT

A—A' APPROXIMATE LOCATIONS OF CROSS SECTIONS

Qaf – ARTIFICIAL FILL

Qcol – COLLUVIUM

Qd – QUARTZ DIORITE

0 60 120

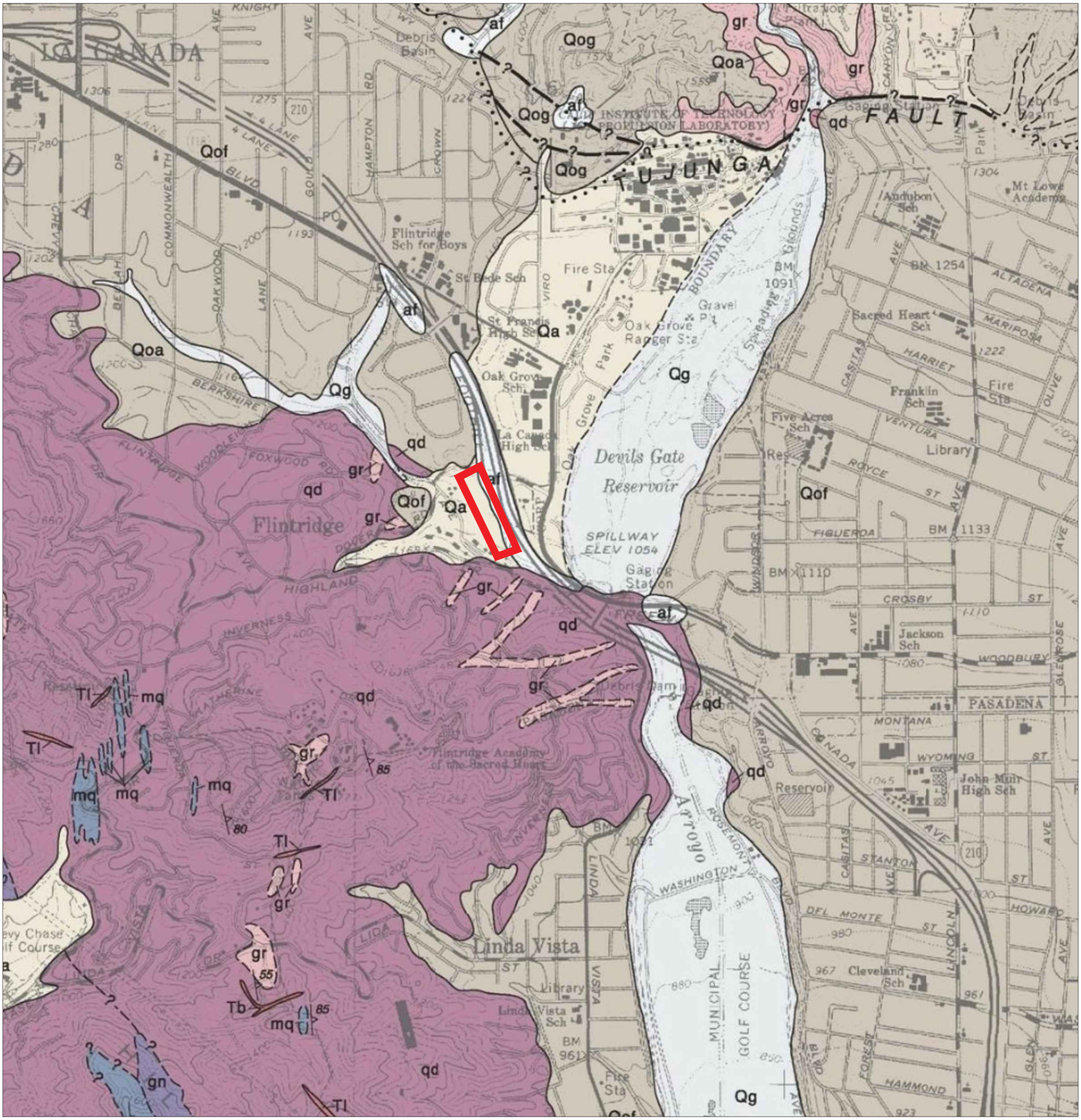
APPROXIMATE SCALE IN FEET

REFERENCE: LA CANADA (2020)

SITE PLAN AND BORING LOCATION MAP

FLINT CANYON WASH TRAIL
LA CANADA FLINTRIDGE, CA

PROJECT NO. 200376.1	REPORT DATE December 2020	FIGURE 2



APPROXIMATE LOCATION OF PROJECT

Qa	Alluvium
af	Artificial fill
qd	Quartz diorite
gr	Granitic rocks
Qg	Stream channel deposits

N

SCALE IN FEET

0 2000 4000

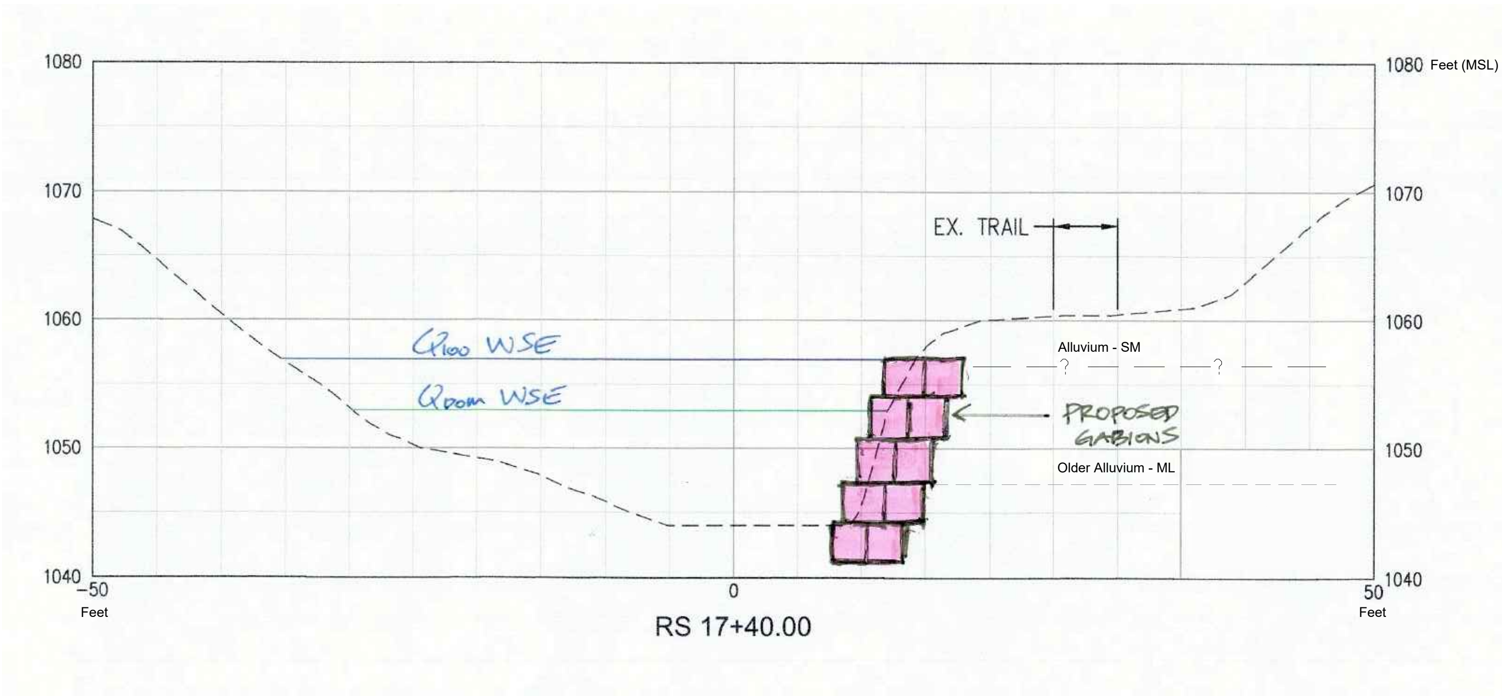
REFERENCE: DIBBLEE (1989)

TWINING

GEOLOGIC MAP

FLINT CANYON WASH TRAIL
LA CANADA FLINTRIDGE, CA

PROJECT NO.	REPORT DATE	FIGURE 3
200376.1	December 2020	



SCALE IN FEET

0 15 30

TWINING

LEGEND

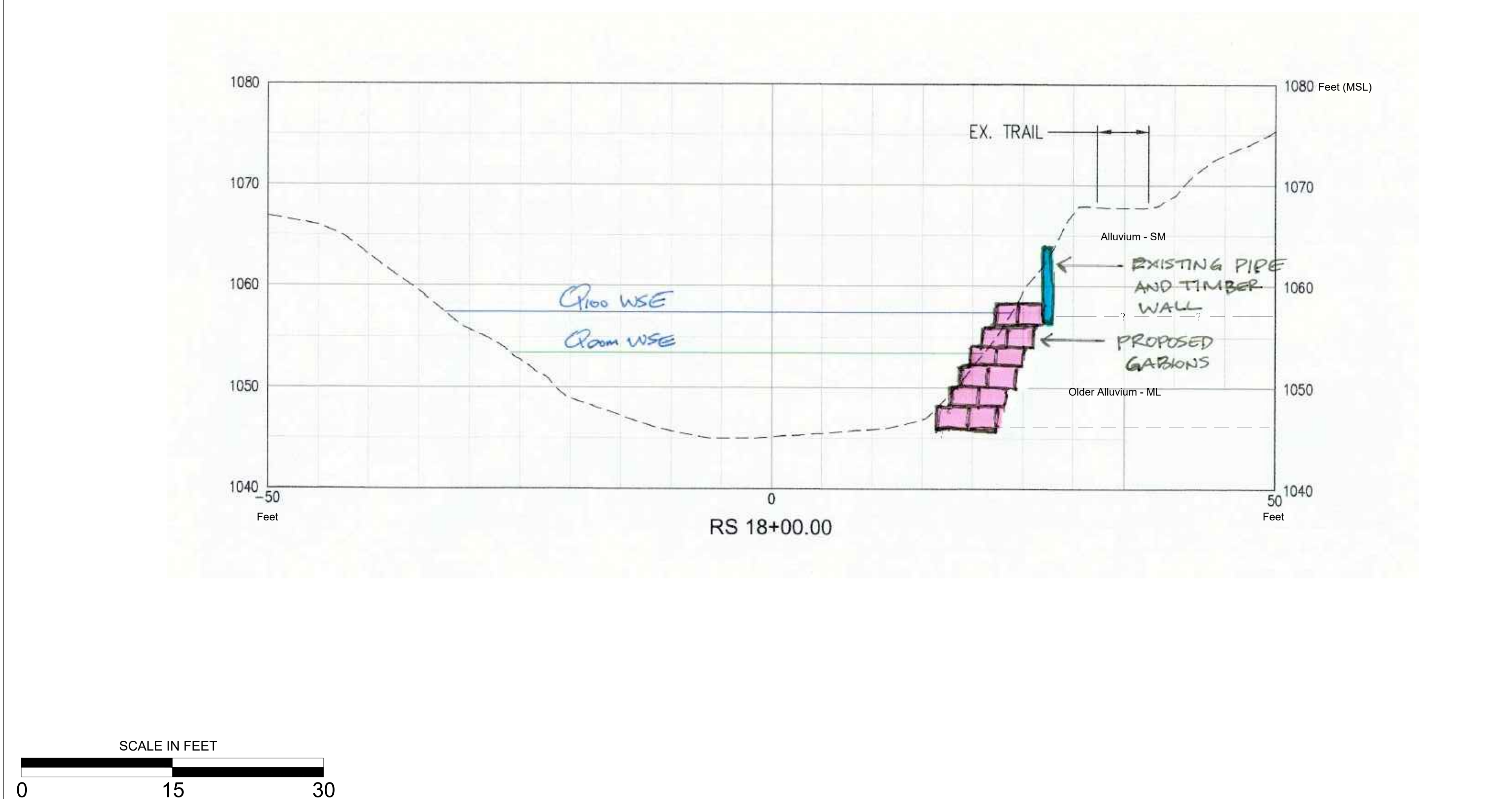
Q_{100} WSE	Water Surface Elevation During 100-year Flood
Q_{DOM} WSE	Water Surface Elevation During Dominant Flood
— — ? — —	Approximate Geologic Contact
— — — —	Approximate Depth of Exploration

REFERENCE: PACE (2020)

CROSS SECTION 17+40

FLINT CANYON WASH TRAIL
LA CANADA FLINTRIDGE, CALIFORNIA

PROJECT No.	REPORT DATE	
200376.1	December 2020	FIGURE 4A



TWINING

LEGEND

Q_{100} WSE — Water Surface Elevation During 100-year Flood

Q_{DOM} WSE — Water Surface Elevation During Dominant Flood

— — ? — — Approximate Geologic Contact

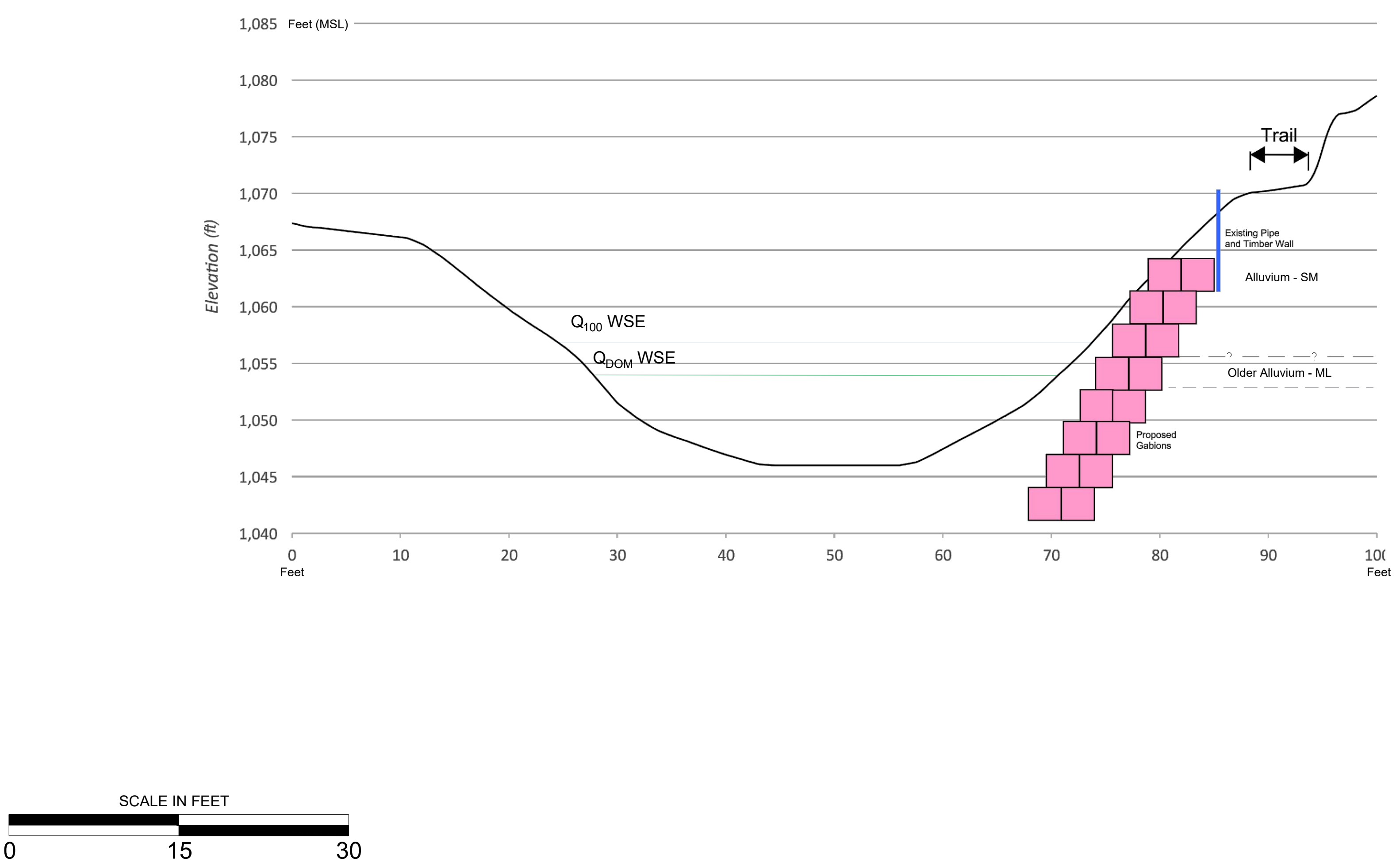
- - - - - - Approximate Depth of Exploration

REFERENCE: PACE (2020)

CROSS SECTION 18+00

FLINT CANYON WASH TRAIL
LA CANADA FLINTRIDGE, CALIFORNIA

PROJECT No.	REPORT DATE	FIGURE 4B
200376.1	December 2020	



TWINING

LEGEND

Q_{100} WSE — Water Surface Elevation During 100-year Flood

Q_{DOM} WSE — Water Surface Elevation During Dominant Flood

Approximate Geologic Contact

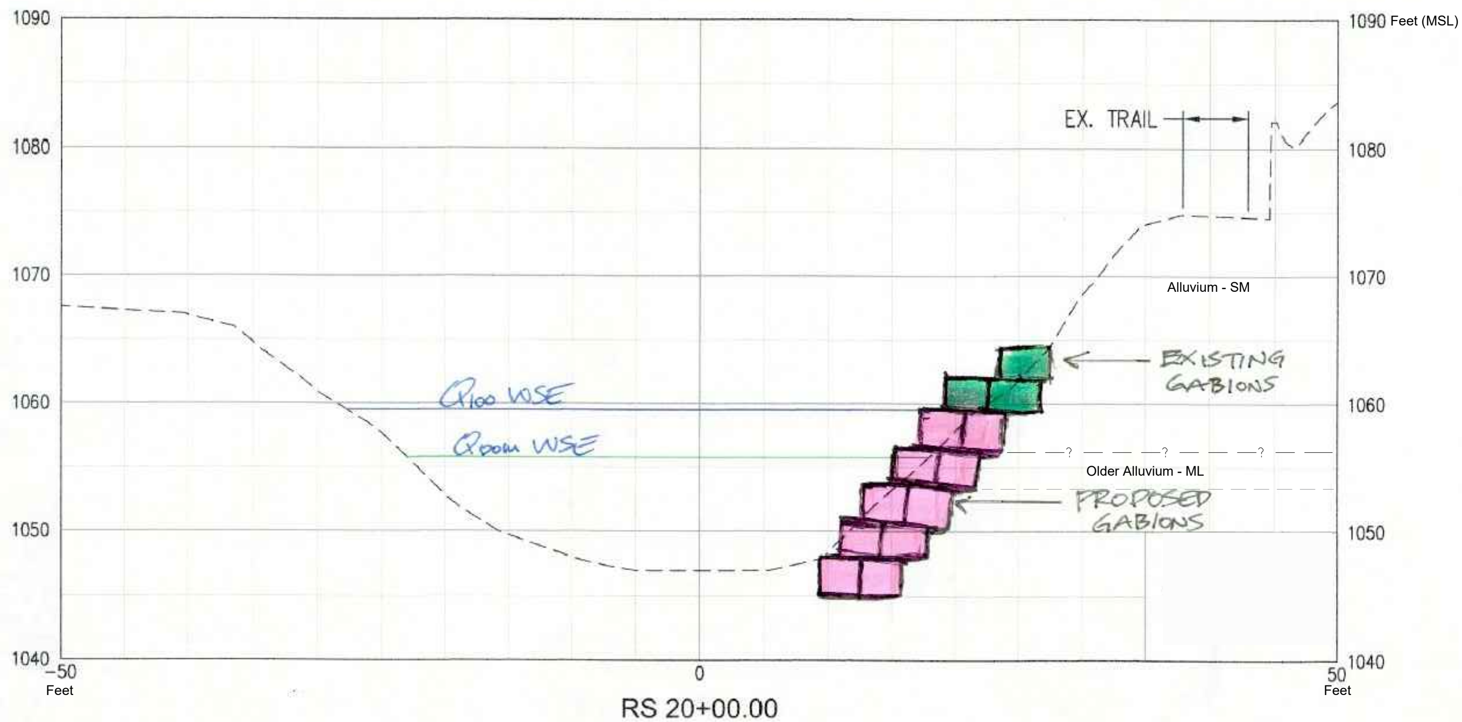
Approximate Depth of Exploration

REFERENCE: PACE (2020)

CROSS SECTION 19+00

FLINT CANYON WASH TRAIL
LA CANADA FLINTRIDGE, CALIFORNIA

PROJECT No.	REPORT DATE	
200376.1	December 2020	FIGURE 4C



SCALE IN FEET

0 15 30

TWINING

LEGEND

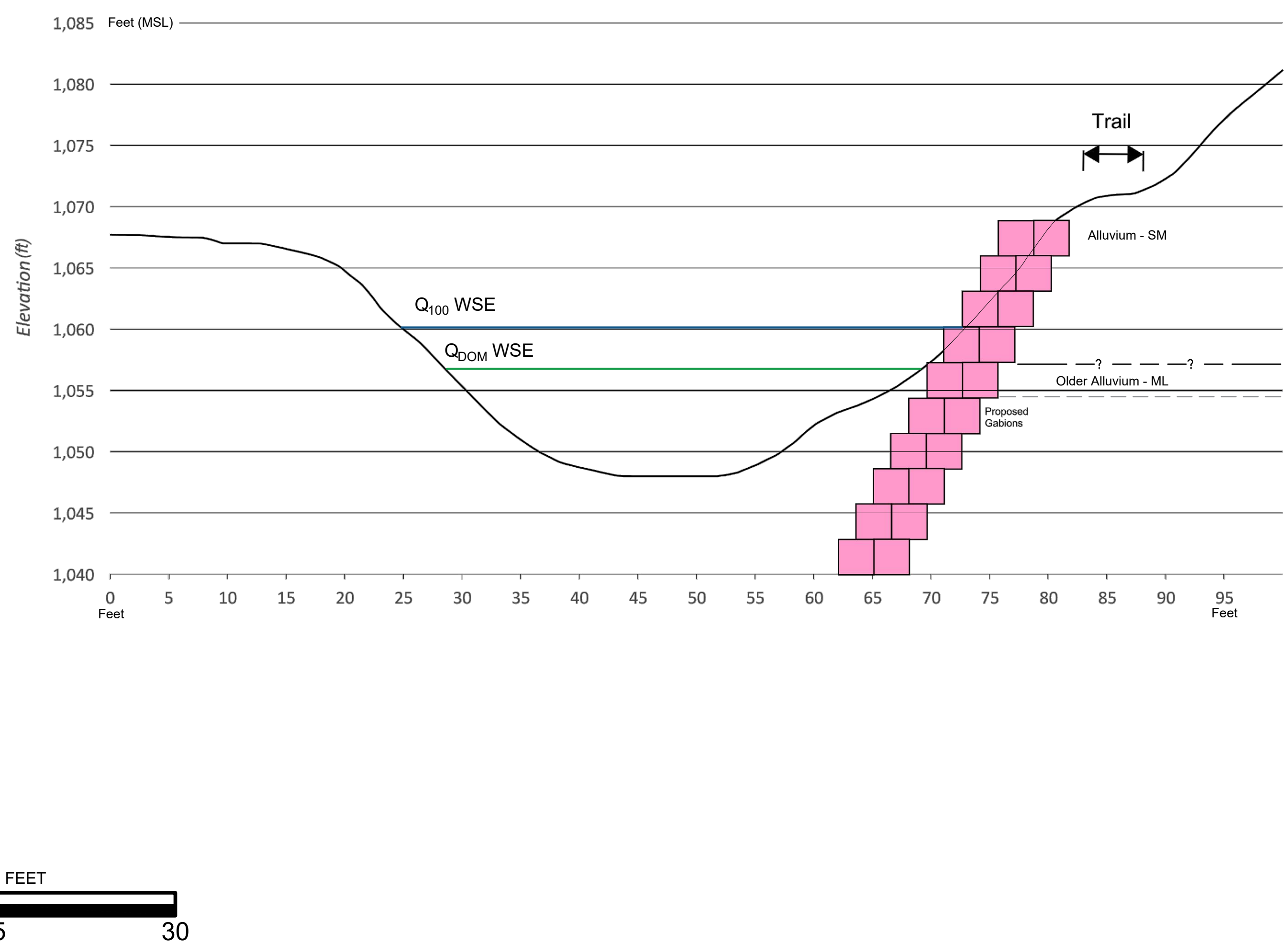
Q_{100} WSE	Water Surface Elevation During 100-year Flood
Q_{DOM} WSE	Water Surface Elevation During Dominant Flood
— — ? — —	Approximate Geologic Contact
- - - - - -	Approximate Depth of Exploration

REFERENCE: PACE (2020)

CROSS SECTION 20+00

FLINT CANYON WASH TRAIL
LA CANADA FLINTRIDGE, CALIFORNIA

PROJECT No.	REPORT DATE	FIGURE 4D
200376.1	December 2020	



TWINING

LEGEND

Q_{100} WSE — Water Surface Elevation During 100-year Flood

Q_{DOM} WSE — Water Surface Elevation During Dominant Flood

—— — ? — —— Approximate Geologic Contact

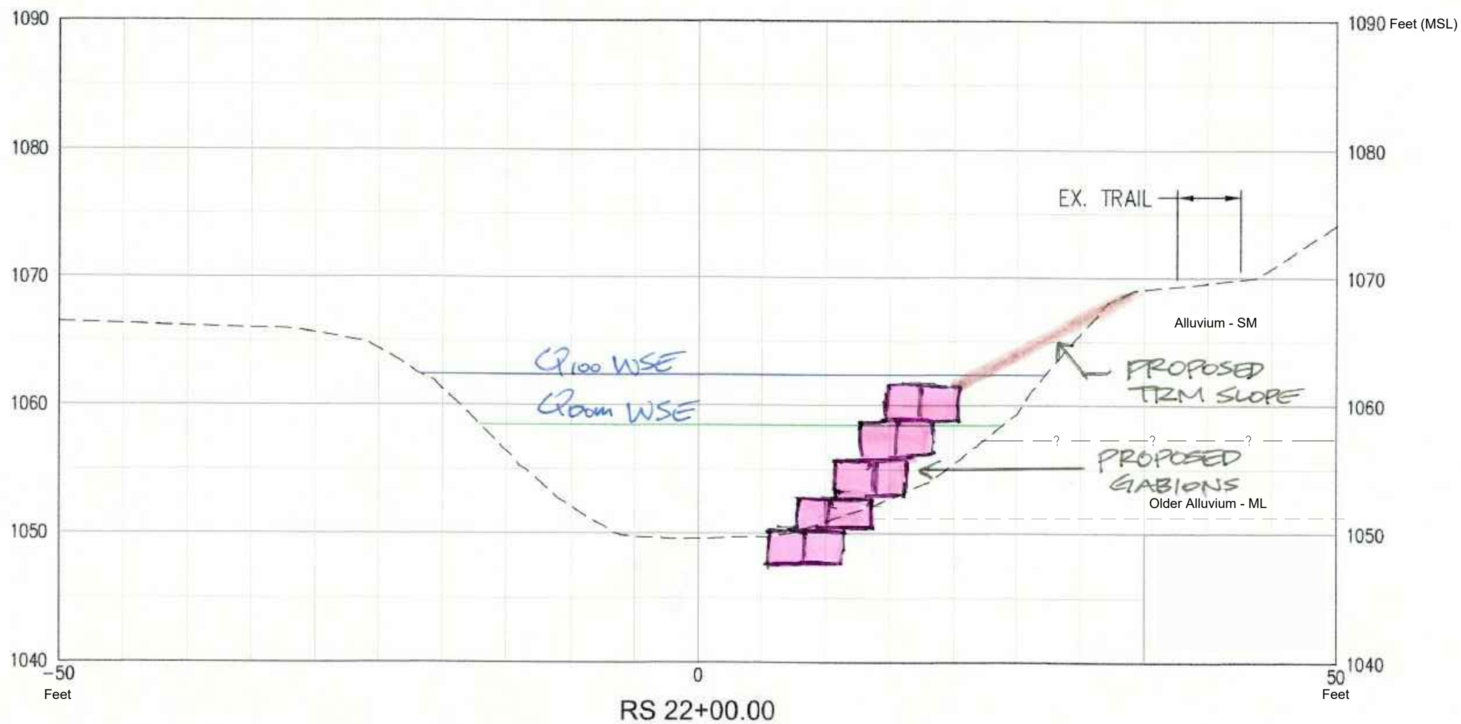
— — — — — — — Approximate Depth of Exploration

REFERENCE: PACE (2020)

CROSS SECTION 21+00

FLINT CANYON WASH TRAIL
LA CANADA FLINTRIDGE, CALIFORNIA

PROJECT No.	REPORT DATE	
200376.1	December 2020	FIGURE 4E



SCALE IN FEET

0 15 30

TWINING

LEGEND

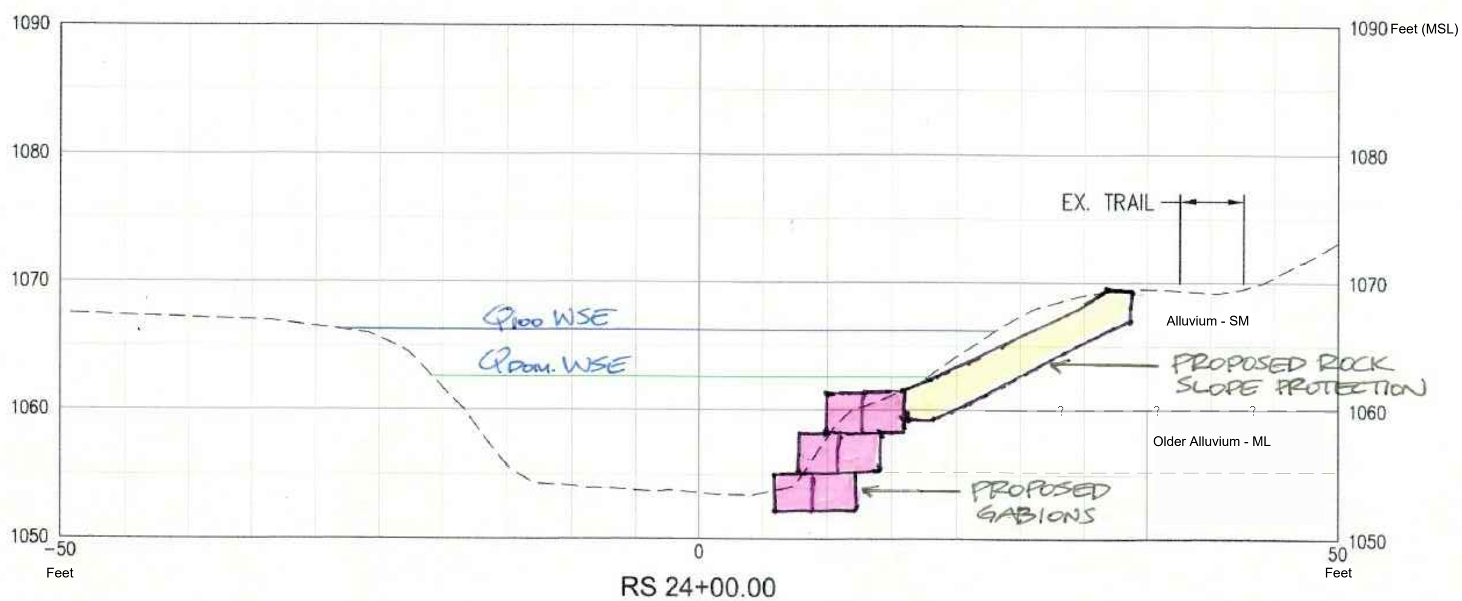
Q_{100} WSE	Water Surface Elevation During 100-year Flood
Q_{DOM} WSE	Water Surface Elevation During Dominant Flood
— — ? — —	Approximate Geologic Contact
- - - - - -	Approximate Depth of Exploration

REFERENCE: PACE (2020)

CROSS SECTION 22+00

FLINT CANYON WASH TRAIL
LA CANADA FLINTRIDGE, CALIFORNIA

PROJECT No.	REPORT DATE	FIGURE 4F
200376.1	December 2020	



SCALE IN FEET

0 15 30

TWINING

LEGEND

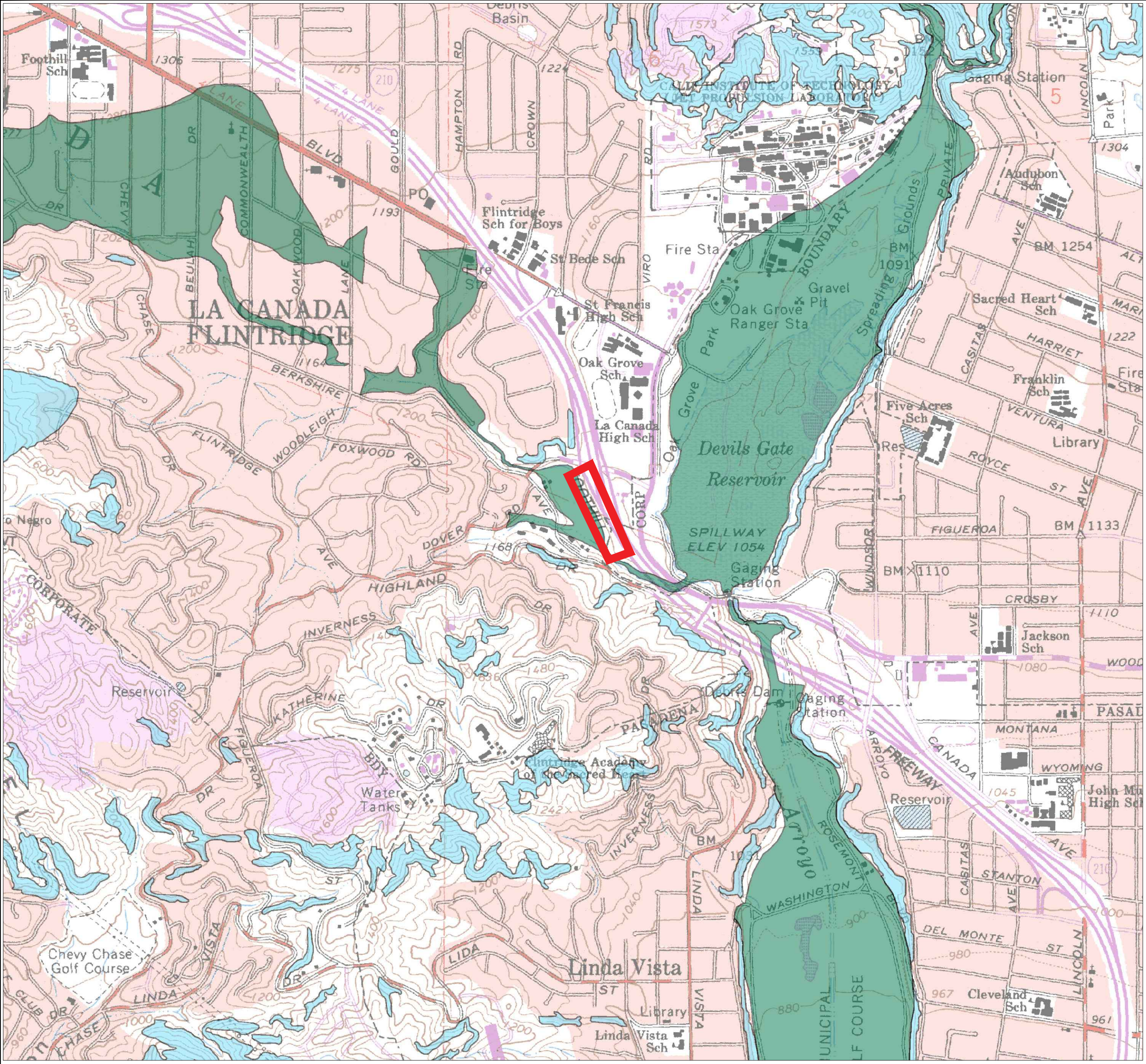
Q_{100} WSE	Water Surface Elevation During 100-year Flood
Q_{DOM} WSE	Water Surface Elevation During Dominant Flood
— — ? — —	Approximate Geologic Contact
- - - - - -	Approximate Depth of Exploration

REFERENCE: PACE (2020)

CROSS SECTION 24+00

FLINT CANYON WASH TRAIL
LA CANADA FLINTRIDGE, CALIFORNIA

PROJECT No.	REPORT DATE	FIGURE 4G
200376.1	December 2020	



MAP EXPLANATION

EARTHQUAKE FAULT ZONES

Earthquake Fault Zones
Zone boundaries are delineated by straight-line segments; the boundaries define the zone encompassing active faults that constitute a potential hazard to structures from surface faulting or fault creep such that avoidance as described in Public Resources Code Section 2621.5(a) would be required.

Active Fault Traces
Faults considered to have been active during Holocene time and to have potential for surface rupture: Solid Line in Black or Red where Accurately Located; Long Dash in Black or Solid Line in Purple where Approximately Located; Short Dash in Black or Solid Line in Orange where Inferred; Dotted Line in Black or Solid Line in Rose where Concealed; Query (?) indicates additional uncertainty. Evidence of historic offset indicated by year of earthquake-associated event or C for displacement caused by fault creep.

SEISMIC HAZARD ZONES

Liquefaction Zones
Areas where historical occurrence of liquefaction, or local geological, geotechnical and ground water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Earthquake-Induced Landslide Zones
Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

N

APPROXIMATE LOCATION OF PROJECT

SCALE IN FEET
0 4000 8000

REFERENCE: CGS (1999)

TWINING

SEISMIC HAZARDS MAP

FLINT CANYON WASH TRAIL
LA CANADA FLINTRIDGE, CA

PROJECT NO.	REPORT DATE	FIGURE 5
200376.1	December 2020	

APPENDIX A
FIELD EXPLORATION AND BORING LOGS

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Appendix A
Field Exploration and Boring Logs

General

The subsurface exploration program for the proposed project consisted of drilling, testing, sampling and logging 5 solid-stem-auger (SSA) exploratory borings (B-1 through B-5) at the site on July 27 and 28, 2020.

The SSA borings were advanced to depths of approximately 15 to 16 feet below the existing ground surface (bgs). Drilling operation was performed by Pacific drilling Co. of San Diego, California using a limited access track-mounted drill rig (Mini Mole) equipped with 6-inch diameter SSA.

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

Drilling and Sampling

An explanation of the boring logs is presented as Figure A-1. The boring logs are presented as Figures A-2 through A-6. The boring logs describe the earth materials encountered, samples obtained, and show the field and laboratory tests performed. The logs also show the boring number, drilling date, and the name of the logger and drilling subcontractor. The borings were logged by a Twining field engineer. The boundaries between soil types shown on the logs are approximate because the transition between different soil layers may be gradual. Drive and bulk samples of representative earth materials were obtained from the borings.

Disturbed samples were obtained from select depths using a Standard Penetration Test (SPT) sampler. This sampler consists of a 2-inch O.D., 1.4-inch I.D. split barrel shaft with room for liner but liner was not used. Soil samples obtained by the SPT sampler were retained in plastic bags. A California modified sampler was also used to obtain drive samples of the soils from select depths. This sampler consists of a 3-inch outside diameter (O.D.), 2.4-inch inside diameter (I.D.) split barrel shaft. The samples were retained in brass rings for laboratory testing.

When the boring was drilled to select depths, the sampler was lowered to the bottom of the boring and then driven a total of 18-inches into the soil using an automatic hammer weighing 140 pounds dropped from a height of 30 inches. The number of blows required to drive the samplers the final 12 inches is presented on the boring logs. If only 6 inches or less was driven after 50 blows, the penetration test was stopped, and the boring was advanced to the next depth.

During drilling, groundwater was encountered at 24 feet bgs in boring B-2 and not encountered in other borings.

Upon completion of the borings, the boreholes were backfilled with cement-bentonite grout, and the surface was repaired to match existing conditions.

UNIFIED SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO 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 (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

COARSE-GRAINED SOILS / FINE-GRAINED SOILS

Relative Density	SPT (blows/ft)	Relative Density (%)	Consistency	SPT (blows/ft)
Very Loose	<4	0 - 15	Very Soft	<2
Loose	4 - 10	15 - 35	Soft	2 - 4
Medium Dense	10 - 30	35 - 65	Medium Stiff	4 - 8
Dense	30 - 50	65 - 85	Stiff	8 - 15
Very Dense	>50	85 - 100	Very Stiff	15 - 30
			Hard	>30

NOTE: SPT blow counts based on 140 lb. hammer falling 30 inches

Sample Symbol	Sample Type	Description
	SPT	1.4 in I.D., 2.0 in. O.D. driven sampler
	California Modified	2.4 in. I.D., 3.0 in. O.D. driven sampler
	Bulk	Retrieved from soil cuttings
	Thin-Walled Tube	Pitcher or Shelby Tube

LABORATORY TESTING ABBREVIATIONS

ATT	Atterberg Limits
C	Consolidation
CORR	Corrosivity Series
DS	Direct Shear
EI	Expansion Index
GS	Grain Size Distribution
K	Permeability
MAX	Moisture/Density (Modified Proctor)
O	Organic Content
RV	Resistance Value
SE	Sand Equivalent
SG	Specific Gravity
TX	Triaxial Compression
UC	Unconfined Compression

TWINING

EXPLANATION FOR LOG OF BORINGS

Trail Restoration
Flint Canyon Wash Trail
La Canada Flintridge, California

PROJECT NO.	REPORT DATE	
200376.1	December 2020	FIGURE A-1

STANDARD LOG EXPLANATION 200376.1 - LA CANADA FLINT CANYON WASH TRAIL.GPJ TWINING LABS.GDT 12/15/20

DATE DRILLED	7/27/2020	LOGGED BY	DHC	BORING NO.	B-1
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	N/E
DRILLING METHOD	6" Solid Stem	DRILLER	Pacific Drilling	SURFACE ELEVATION (ft.)	1068 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
	0							SM	**ALLUVIUM:** Silty SAND; light reddish brown; dry to slightly moist
1063	5			21				SM	-- same; medium dense; with approximately 5% fine gravel
1058	10			50 for 5"	29.0	99.4		SM	**OLDER ALLUVIUM:** Silty SAND; very dense; slightly moist; trace clay; with approximately 10% gravel
1053	15			30				SM	-- same; medium dense; moist to wet
									Total Depth = 16.5 feet Backfilled on 7/27/2020 Groundwater not encountered. Backfilled with bentonite.
1048	20								
1043	25								
1038	30								
1033	35								

TWINING

LOG OF BORING

Trail Restoration
Flint Canyon Wash Trail
La Canada Flintridge, California

PROJECT NO.	REPORT DATE	
200376.1	December 2020	FIGURE A - 2

DATE DRILLED	7/27/2020	LOGGED BY	DHC	**BORING NO.**	**B-2**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	24
DRILLING METHOD	6" Solid Stem	DRILLER	Pacific Drilling	SURFACE ELEVATION (ft.)	1073 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
	0				10.9		ATT, DS, GS, MAX		SM	**FILL:** Silty SAND; light reddish brown; dry to slightly moist; with approximately 10% gravel
1068	5			50	2.4	119.7			SM	**ALLUVIUM:** Silty SAND; dense; light reddish brown; dry to slightly moist; with some black and white granite gravel
1063	10			33					SM	-- same; dense; trace clay
1058	15			36/50 for 5"	15.6	91.4	DS		ML	**OLDER ALLUVIUM:** Sandy SILT; hard; yellow brown to grey; slightly moist
1053	20			33			#200, ATT		ML	SILT; hard; reddish brown to grey; slightly moist; some sand inclusions
	24									▼
1048	25			12/50 for 6"	15.0	112.6			CL	-- same; hard; wet; with some coarse sand
1043	30			37/50 for 6"					CL	-- same; hard; wet
	31.5									Total Depth = 31.5 feet Backfilled on 7/27/2020 Groundwater encountered at approximately 24 feet below ground surface. Backfilled with bentonite.
1038	35									

BORING LOG 200376.1 - LA CANADA FLINT CANYON WASH TRAIL.GPJ TWINING LABS.GDT 12/15/20

TWINING

LOG OF BORING

Trail Restoration
Flint Canyon Wash Trail
La Canada Flintridge, California

PROJECT NO.	REPORT DATE	
200376.1	December 2020	FIGURE A - 3

DATE DRILLED	7/27/2020	LOGGED BY	DHC	**BORING NO.**	**B-3**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	N/E
DRILLING METHOD	6" Solid Stem	DRILLER	Pacific Drilling	SURFACE ELEVATION (ft.)	1067 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
									SM	**ALLUVIUM:** Silty SAND; light reddish brown; dry to slightly moist
1062	5			50 for 6"	2.7	122.5			SM	-- same; very dense; with some weathered black and white granite gravel
1057	10			58			#200, ATT		ML	**OLDER ALLUVIUM:** SILT; hard; yellowish brown to tan; dry to slightly moist; weathered
1052	15			50 for 6"	23.0	94.9			ML	-- same; hard
										Total Depth = 16.5 feet Backfilled on 7/27/2020 Groundwater not encountered. Backfilled with bentonite.
1047	20									
1042	25									
1037	30									
1032	35									

BORING LOG 200376.1 - LA CANADA FLINT CANYON WASH TRAIL.GPJ TWINING LABS.GDT 12/15/20

TWINING

LOG OF BORING

Trail Restoration
Flint Canyon Wash Trail
La Canada Flintridge, California

PROJECT NO.	REPORT DATE	
200376.1	December 2020	FIGURE A - 4

DATE DRILLED	7/28/2020	LOGGED BY	DHC	**BORING NO.**	**B-4**
DRIVE WEIGHT	140 lbs.	DROP	30 inches	DEPTH TO GROUNDWATER (ft.)	N/E
DRILLING METHOD	6" Solid Stem	DRILLER	Pacific Drilling	SURFACE ELEVATION (ft.)	1071 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
									SM	**ALLUVIUM:** Silty SAND; light reddish brown; dry to slightly moist; with approximately 5% gravel and 5% cobble
1066	5			50 for 5"	5.3	101.8	DS		SM	-- same; very dense
1061	10			39			#200, ATT		ML	Sandy SILT; hard; light reddish brown; slightly moist
1056	15			50 for 6"	13.2	85.8			ML	**OLDER ALLUVIUM:** SILT with sand; hard; tan to reddish and yellowish brown; dry to slightly moist
										Total Depth = 16.5 feet Backfilled on 7/28/2020 Groundwater not encountered. Backfilled with bentonite.
1051	20									
1046	25									
1041	30									
1036	35									

BORING LOG 200376.1 - LA CANADA FLINT CANYON WASH TRAIL.GPJ TWINING LABS.GDT 12/15/20

TWINING

LOG OF BORING

Trail Restoration
Flint Canyon Wash Trail
La Canada Flintridge, California

PROJECT NO.	REPORT DATE	
200376.1	December 2020	FIGURE A - 5

DATE DRILLED 7/28/2020 | LOGGED BY DHC | **BORING NO. B-5**

DRIVE WEIGHT 140 lbs. | DROP 30 inches | DEPTH TO GROUNDWATER (ft.) N/E

DRILLING METHOD 6" Solid Stem | DRILLER Pacific Drilling | SURFACE ELEVATION (ft.) 1062 ±(MSL)

ELEVATION (feet)	DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS / FOOT	MOISTURE (%)	DRY DENSITY (pcf)	ADDITIONAL TESTS	GRAPHIC LOG	U.S.C.S. CLASSIFICATION	DESCRIPTION
							#200, EI		SM	**ALLUVIUM:** Silty SAND; light reddish brown; dry to slightly moist
1057	5			38			#200, ATT		ML	**OLDER ALLUVIUM:** SILT with sand; hard; tan to reddish and yellowish brown; dry to slightly moist; weathered
1052	10			72	6.9	116.1	DS		SM	Silty SAND; dense; dark reddish brown to black; slightly moist
1047	15			39			#200, ATT		ML	Sandy SILT; hard; dark reddish brown; moist
										Total Depth = 16.5 feet Backfilled on 7/28/2020 Groundwater not encountered. Backfilled with bentonite.
1042	20									
1037	25									
1032	30									
1027	35									

BORING LOG 200376.1 - LA CANADA FLINT CANYON WASH TRAIL.GPJ TWINING LABS.GDT 12/15/20

TWINING

LOG OF BORING

Trail Restoration
Flint Canyon Wash Trail
La Canada Flintridge, California

PROJECT NO.	REPORT DATE	
200376.1	December 2020	FIGURE A - 6

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APPENDIX B
LABORATORY TESTING

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Appendix B
Laboratory Testing

Laboratory Moisture Content and Density Tests

The moisture content and dry densities of selected driven samples obtained from the exploratory borings were evaluated in general accordance with the latest version of ASTM D2937 and D2216. The results are shown on the boring logs in Appendix A, and also summarized in Table B-1.

No. 200 Wash Sieve

The fines content passing the No. 200 sieve was evaluated in accordance with ASTM D1140. The results are presented in Table B-2.

Atterberg Limits

Tests were performed on selected representative fine-grained soil samples to evaluate the liquid limit, plastic limit, and plasticity index in general accordance with ASTM D4318. These test results were utilized to evaluate the soil classification in accordance with the Unified Soil Classification System. The test results are summarized in Table B-3.

Sieve Analysis

The grain size distribution of a representative soil sample was evaluated in accordance with ASTM D 6913. The results are presented in Figures B-1 through B-4.

Expansion Index

The expansion index of a select soil sample was evaluated in general accordance with ASTM D4829. The specimen was molded under a specified compactive energy at approximately 50 percent saturation. The prepared 1-inch thick by 4-inch diameter specimen was loaded with a surcharge of 144 pounds per square foot (psf) and was inundated with tap water. Readings of volumetric swell were made for a period of 24 hours. The result of expansion index test is presented in Table B-4.

Direct Shear

Direct shear tests were performed on remolded and representative intact soil samples in general accordance with the latest version of ASTM D 3080 to evaluate the shear strength characteristics of the selected materials. The samples were inundated during shearing to represent adverse field conditions. Test results are presented in Figures B-5 through B-8.

Maximum Density and Optimum Moisture

Modified Proctor testing was performed on near-surface soils to determine the maximum dry density and optimum water content for compaction. The tests were performed in accordance with ASTM D1557 Method A. Test results are presented in Figure B-9.

Unconsolidated Undrained (UU) Shear Strength

Unconsolidated undrained (UU) triaxial compression testing was performed on the older alluvium samples to determine the UU shear strength of the older alluvium. The tests were performed by Hushmand Associates, Inc. (HAI) of Irvine, California in accordance with ASTM D2850. Test results are presented in Table B-5 and the HAI report included in this appendix.

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Corrosivity

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

Table B-1
Moisture Content and Dry Density

Boring No.	Depth (feet)	Moisture Content (%)	Dry Density (pcf)
B-1	10	29.0	99.4
B-2	5	2.4	119.7
B-2	15	15.6	91.4
B-2	25	15.0	112.6
B-3	5	2.7	122.5
B-3	15	23.0	94.9
B-4	5	5.3	101.8
B-4	15	13.2	85.8
B-5	10	6.9	116.1

Table B-2
Number 200 Wash Results

Boring No.	Depth (feet)	Percent Passing #200
B-2	0-5	34.0
B-2	20	87.3
B-3	10	94.4
B-4	10	63.6
B-5	0-5	44.8
B-5	5	74.7
B-5	15	56.8
S-1	0	0.5
S-2	0	0.9
S-3	0	0.3

Note: S-1, S-2, and S-3 are samples grabbed from the stream bed.

Table B-3
Atterberg Limits

Boring No.	Depth (feet)	Liquid Limit	Plastic Limit	Plasticity Index	Soil Description
B-2	0-5	NP	NP	NP	Silty Sand (SM)
B-2	20	41	26	15	Silt (ML)
B-3	10	NP	NP	NP	Silt (ML)
B-4	10	NP	NP	NP	Sandy Silt (ML)
B-5	5	NP	NP	NP	Silt with Sand (ML)
B-5	15	NP	NP	NP	Sandy Silt (ML)

Note: NP= Non-plastic

Table B-4
Expansion Index

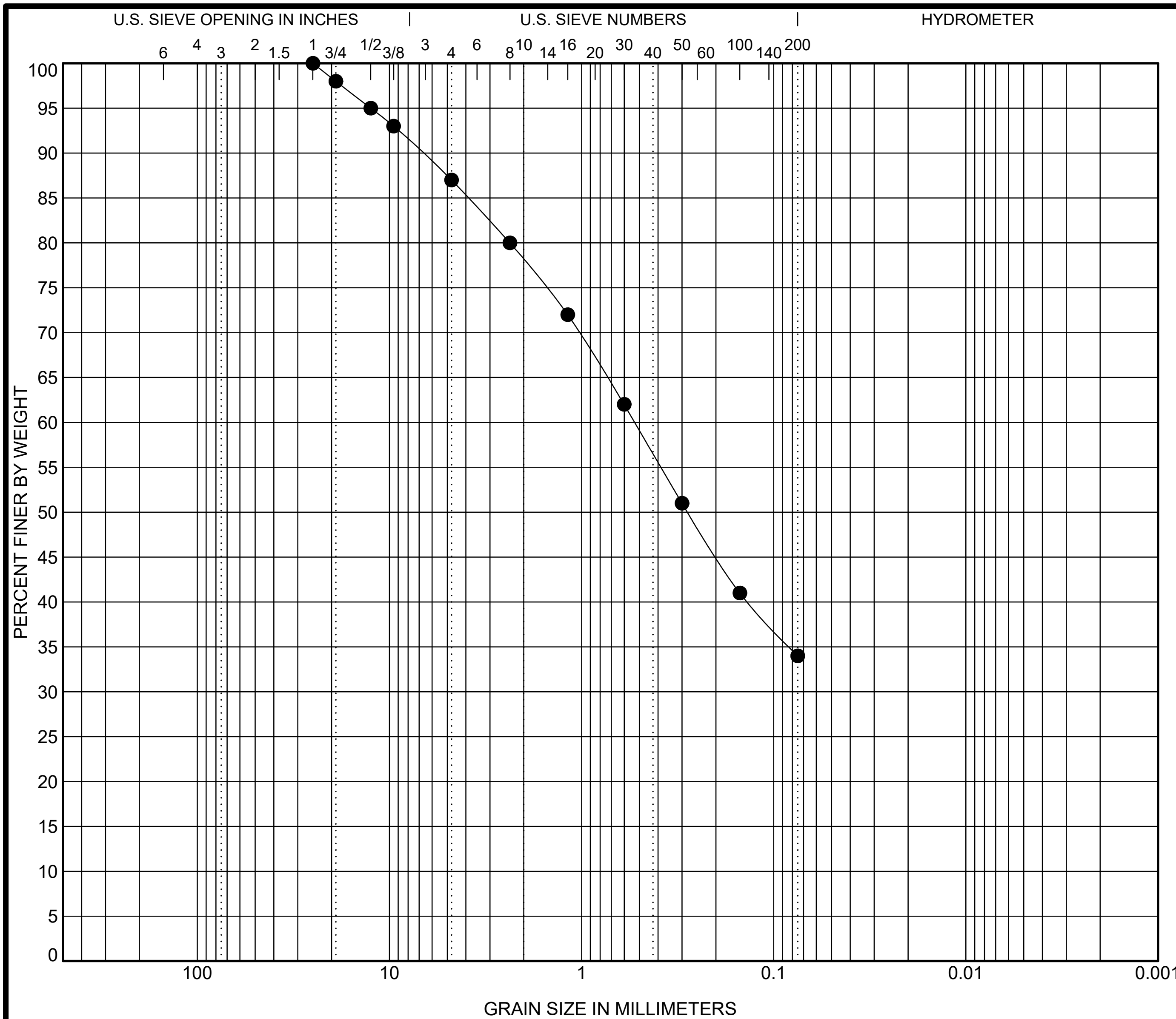
Boring No.	Depth (feet)	Expansion Index	Expansion Potential
B-5	0 - 5	3	Very low

Table B-5
Unconsolidated Undrained (UU) Triaxial Compression Test Results

Boring No.	Depth (feet)	Confining Stress at Failure, $\sigma_{3,f}$ (psf)	Axial Stress at Failure, $\sigma_{1,f}$ (psf)	Axial Strain at Failure, ε_f (%)	UU Shear Strength (psf)
B-3	15	4,480	1,400	4.13	1,540
B-4	15	3,930	1,390	4.70	1,270

Table B-6
Corrosivity Test Results

Boring No.	Depth (feet)	pH	Water Soluble Sulfate (ppm)	Water Soluble Chloride (ppm)	Minimum Resistivity (ohm-cm)
B-5	0-5	7.7	90	81	1,800



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Sample Location	U.S.C.S. Classification	Cc	Cu
● **B-2 at 0-5 ft**	**Silty SAND**		

D_{100}	D_{60}	D_{50}	D_{30}	D_{10}	%Gravel	%Sand	%Silt	%Clay
25	**0.529**	**0.28**			**13.0**	**53.0**	**34.0**	

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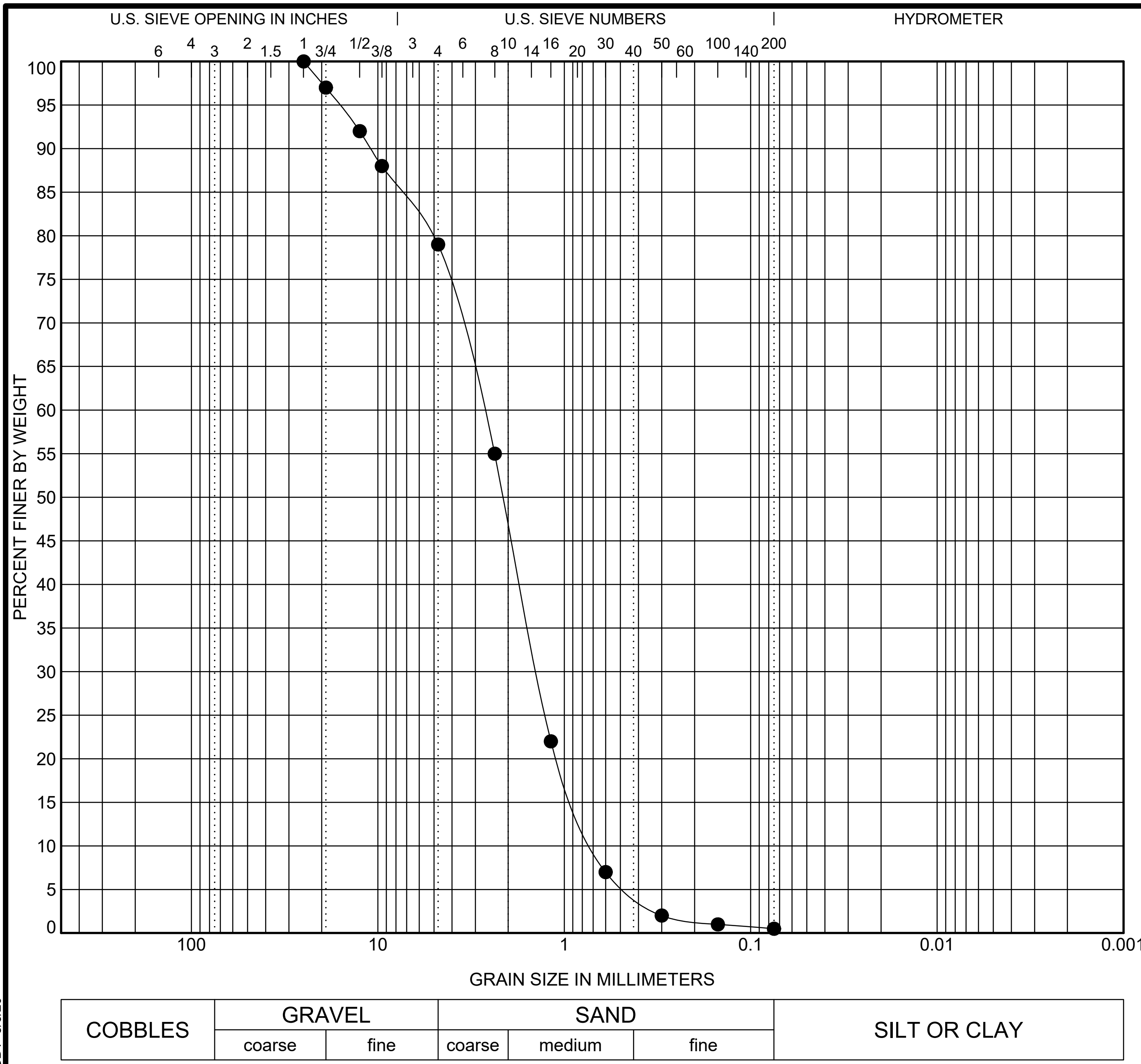
GRAIN SIZE DISTRIBUTION

Trail Restoration
Flint Canyon Wash Trail
La Canada Flintridge, California

PROJECT NO.	REPORT DATE	FIGURE B- 1
200376.1	December 2020	



Sample Location	U.S.C.S. Classification	Cc	Cu
● **S1**	**POORLY GRADED SAND with GRAVEL(SP)**	**1.04**	**3.97**

D_{100}	D_{60}	D_{50}	D_{30}	D_{10}	%Gravel	%Sand	%Silt	%Clay
25	**2.73**	**2.125**	**1.396**	**0.687**	**21.0**	**78.5**	**0.5**	

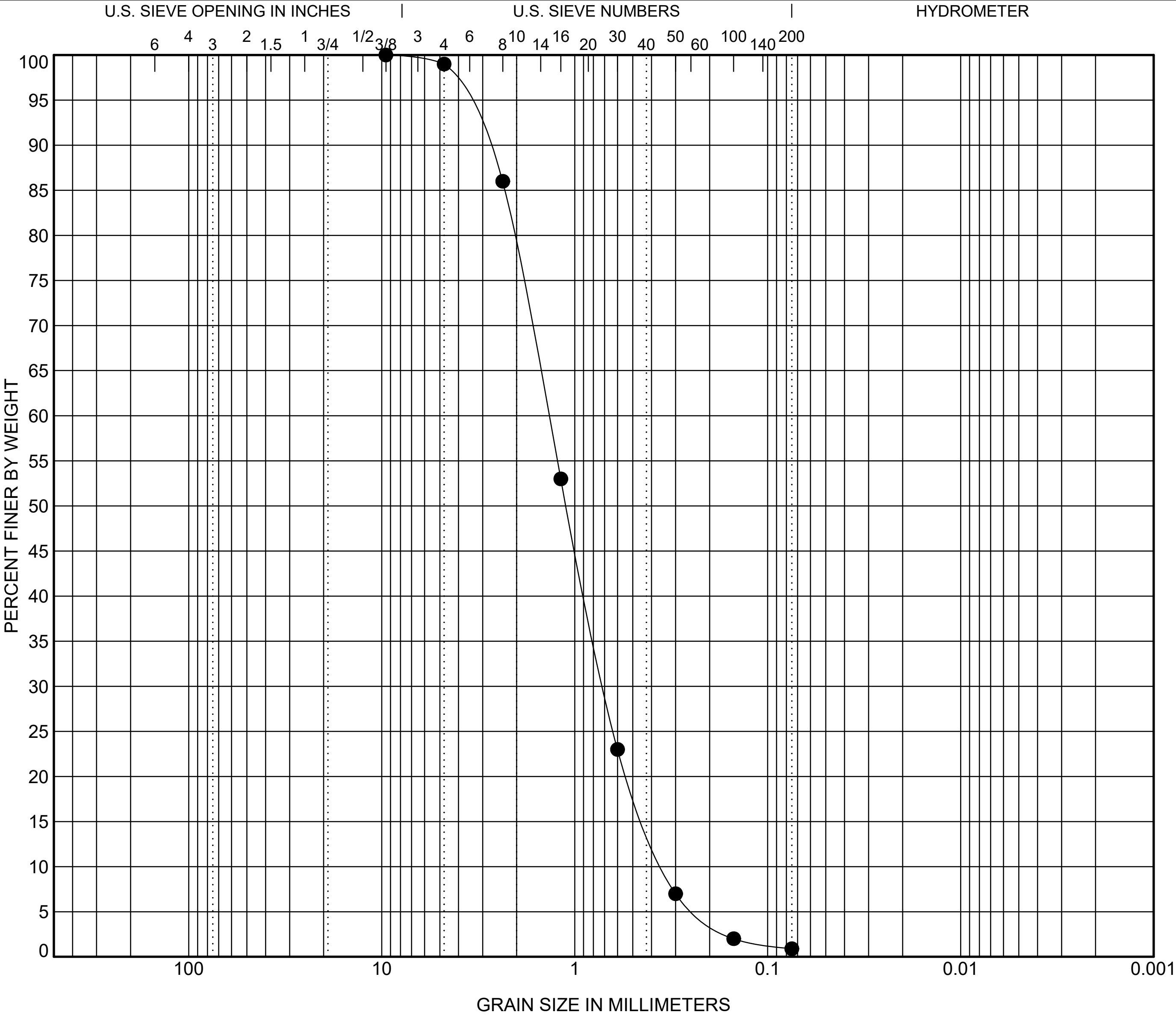
TWINING

GRAIN SIZE DISTRIBUTION

Trail Restoration
Flint Canyon Wash Trail
La Canada Flintridge, California

PROJECT NO.	REPORT DATE	
200376.1	December 2020	FIGURE B- 2

GRAIN SIZE 200376.1 - LA CANADA FLINT CANYON WASH TRAIL.GPJ TWINING LABS.GDT 8/6/20



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Sample Location	U.S.C.S. Classification	Cc	Cu
●	**S2**	**POORLY GRADED SAND(SP)**	**1.06**	**4.00**

D_{100}	D_{60}	D_{50}	D_{30}	D_{10}	%Gravel	%Sand	%Silt	%Clay
9.5	**1.367**	**1.103**	**0.703**	**0.342**	**1.0**	**98.1**	**0.9**	

GRAIN SIZE 200376.1 - LA CANADA FLINT CANYON WASH TRAIL.GPJ TWINING LABS.GDT 8/6/20

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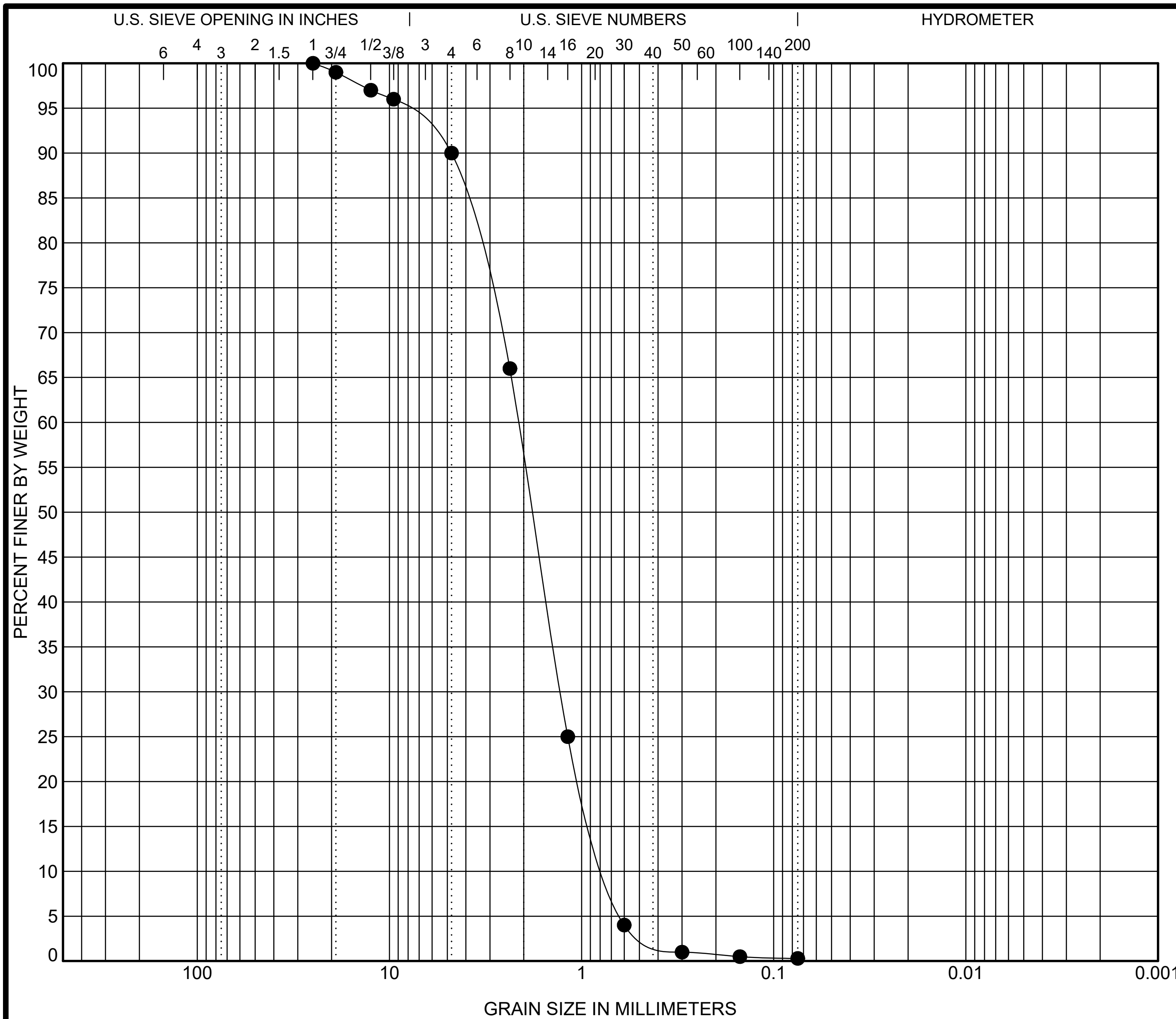
GRAIN SIZE DISTRIBUTION

Trail Restoration
Flint Canyon Wash Trail
La Canada Flintridge, California

PROJECT NO.	REPORT DATE	FIGURE B- 3
200376.1	December 2020	



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Sample Location	U.S.C.S. Classification	Cc	Cu
● **S3**	**POORLY GRADED SAND(SP)**	**1.06**	**2.93**

D_{100}	D_{60}	D_{50}	D_{30}	D_{10}	%Gravel	%Sand	%Silt	%Clay
25	**2.132**	**1.801**	**1.284**	**0.728**	**10.0**	**89.7**	**0.3**	

GRAIN SIZE 200376.1 - LA CANADA FLINT CANYON WASH TRAIL.GPJ TWINING LABS.GDT 8/6/20

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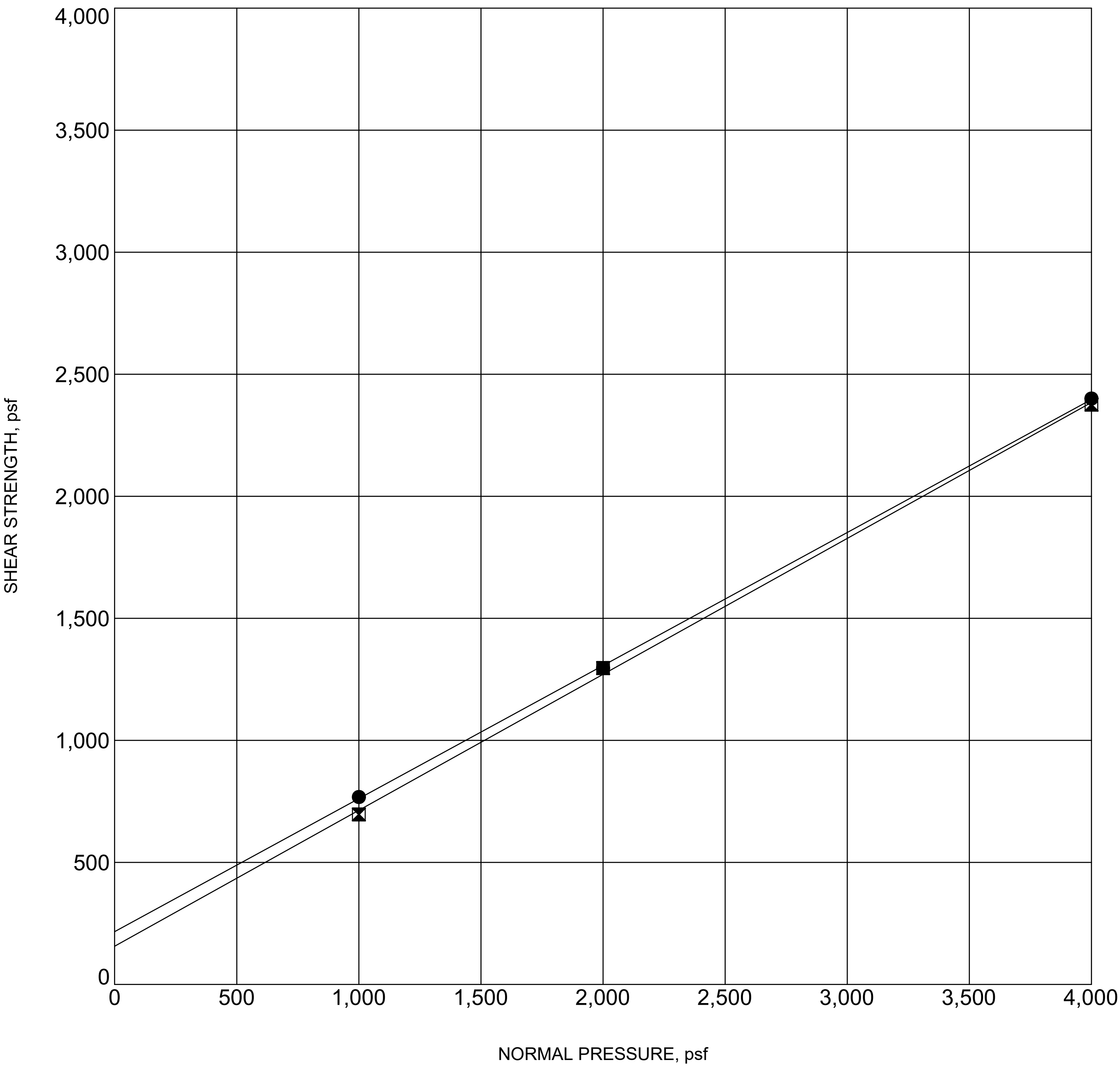
GRAIN SIZE DISTRIBUTION

Trail Restoration
Flint Canyon Wash Trail
La Canada Flintridge, California

PROJECT NO.	REPORT DATE	FIGURE B- 4
200376.1	December 2020	



Boring No.:	B-2
Sample Depth (ft):	0-5' BULK
Sample Description:	Silty SAND
Strain Rate (in./min):	0.005
Dry Density (pcf):	117.4

Shear Strength Parameters

	Peak	**Ultimate**
Cohesion, C (psf):	216	156
Friction Angle, Ø (deg):	29	29
Initial Moisture (%):	7.3	
Final Moisture (%):	10.9	

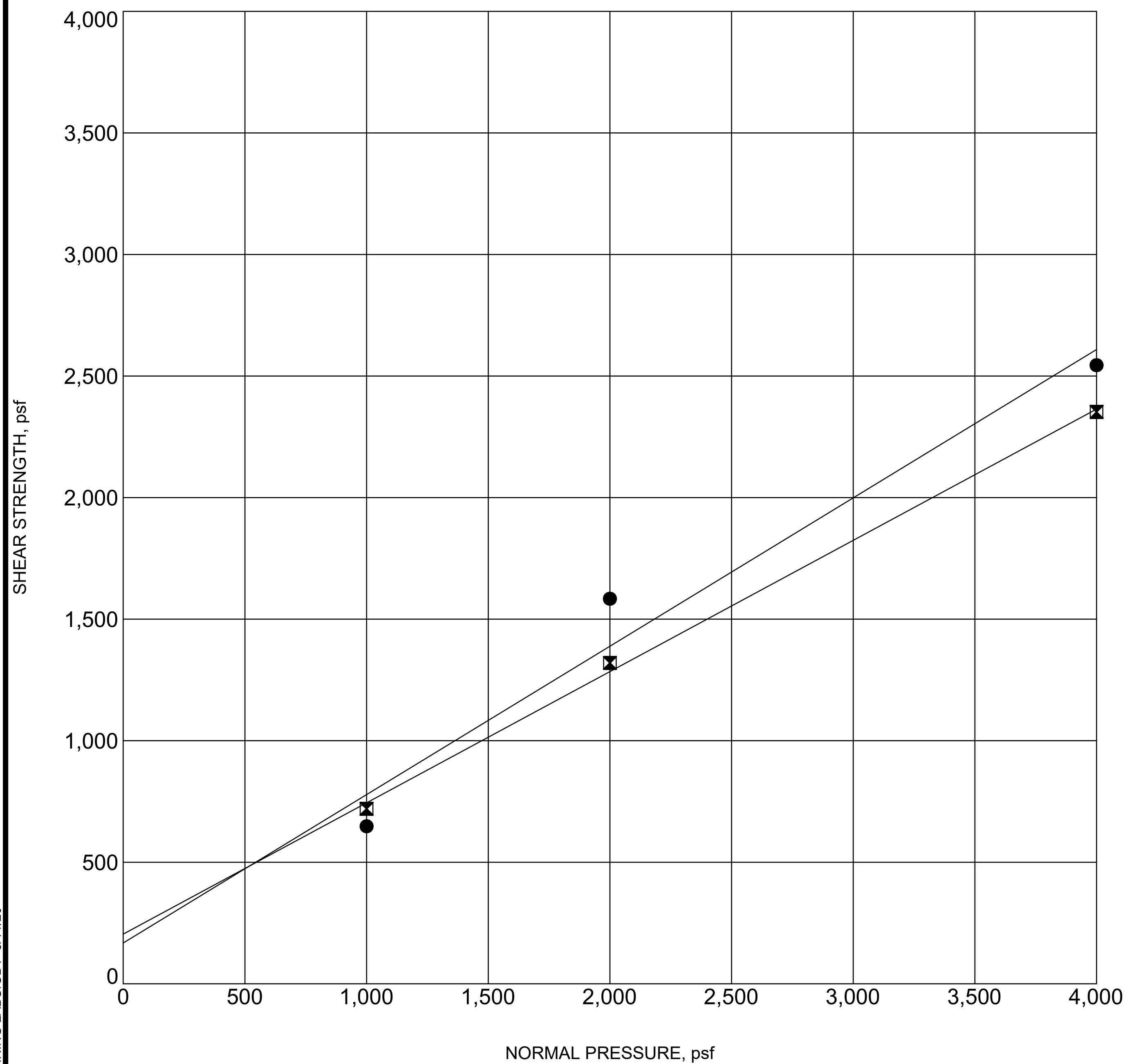
Recompacted to 90% Relative Compaction

TWINING

DIRECT SHEAR TEST

Trail Restoration
Flint Canyon Wash Trail
La Canada Flintridge, California

PROJECT NO.	REPORT DATE	FIGURE B-5
200376.1	December 2020	



Shear Strength Parameters

			Peak	Ultimate
Boring No.:	B-2 15	**Cohesion, C (psf):**	168	204
Sample Depth (ft):	15	**Friction Angle, Ø (deg):**	31	28
Sample Description:	Sandy SILT	**Initial Moisture (%):**	15.6	
Strain Rate (in./min):	0.005	**Final Moisture (%):**	25.6	
Dry Density (pcf):	91.4			

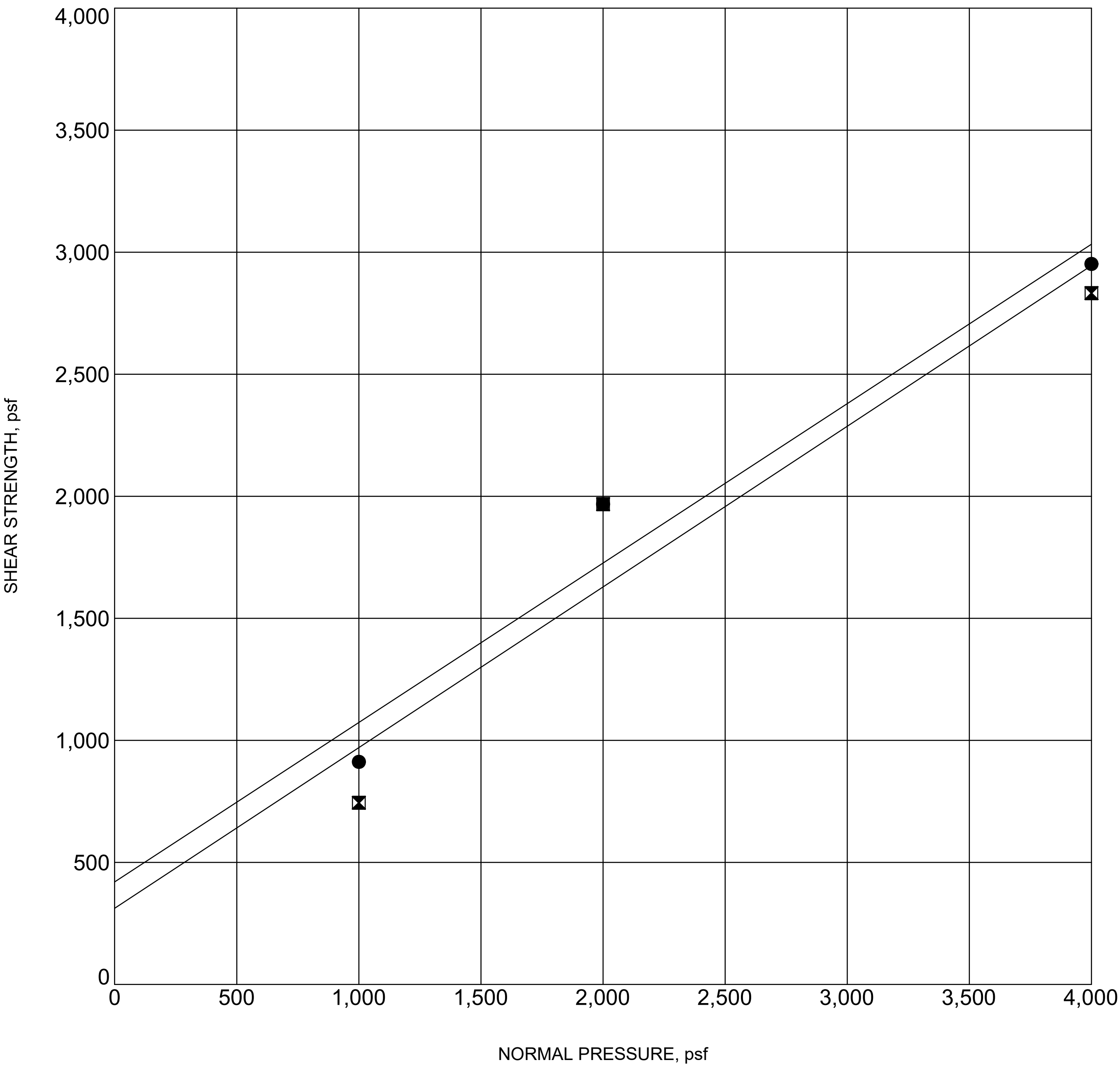
DIRECT SHEAR 200376.1 - LA CANADA FLINT CANYON WASH TRAIL.GPJ TWINING LABS.GDT 8/11/20

TWINING

DIRECT SHEAR TEST

Trail Restoration
Flint Canyon Wash Trail
La Canada Flintridge, California

PROJECT NO.	REPORT DATE	FIGURE B-6
200376.1	December 2020	



Boring No.: B-4
Sample Depth (ft): 5
Sample Description: Silty SAND
Strain Rate (in./min): 0.005
Dry Density (pcf): 101.8

Shear Strength Parameters

	Peak	Ultimate
Cohesion, C (psf):	420	312
Friction Angle, Ø (deg):	33	33

Initial Moisture (%): 5.3
Final Moisture (%): 17.6

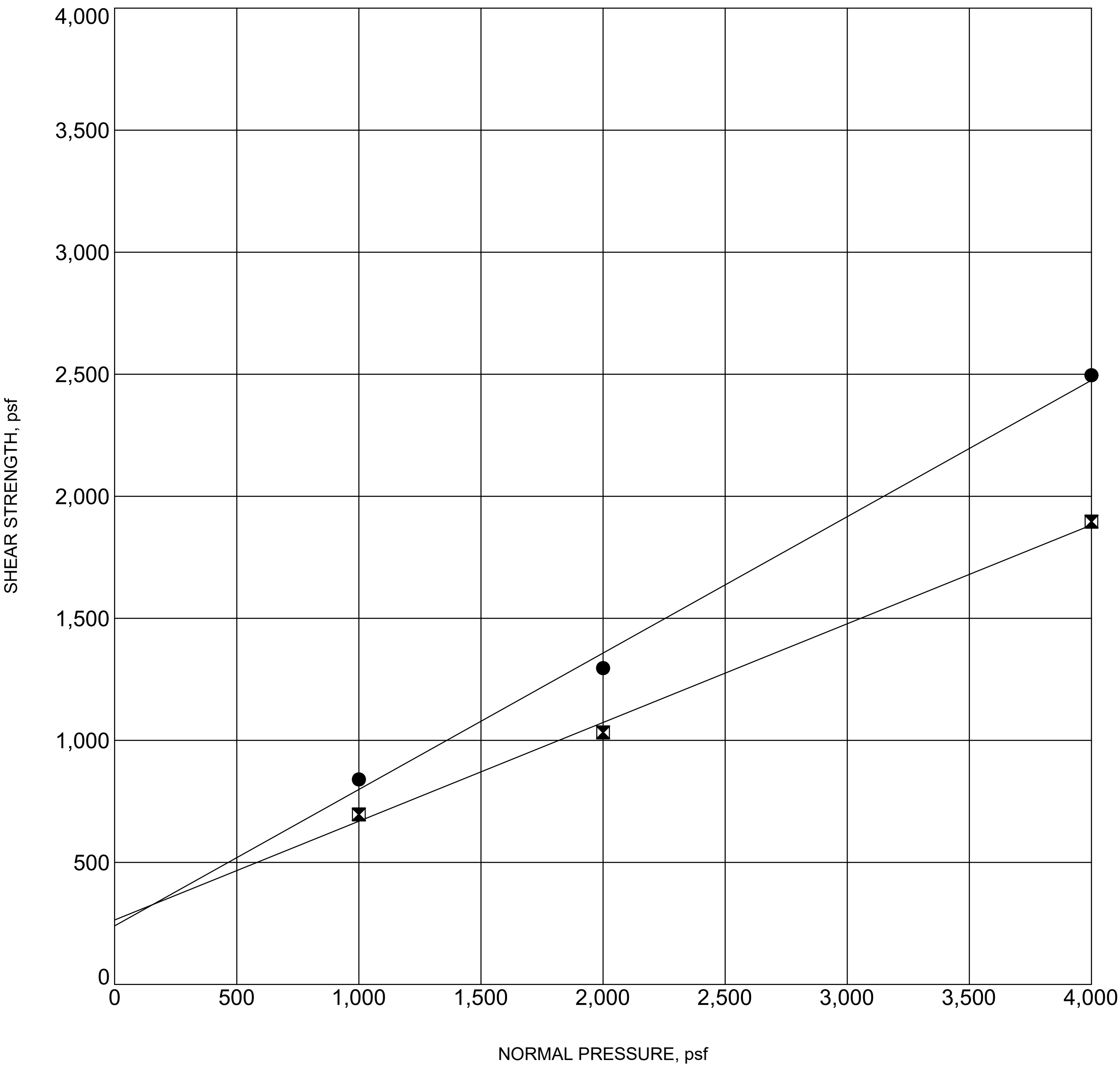
DIRECT SHEAR 200376.1 - LA CANADA FLINT CANYON WASH TRAIL.GPJ TWINING LABS.GDT 8/11/20

TWINING

DIRECT SHEAR TEST

Trail Restoration
Flint Canyon Wash Trail
La Canada Flintridge, California

PROJECT NO.	REPORT DATE	FIGURE B-7
200376.1	December 2020	



Boring No.: B-5 10
Sample Depth (ft): 10
Sample Description: Silty SAND
Strain Rate (in./min): 0.005
Dry Density (pcf): 116.1

Shear Strength Parameters

	Peak	Ultimate
Cohesion, C (psf):	240	264
Friction Angle, Ø (deg):	29	22

Initial Moisture (%): 6.9
Final Moisture (%): 21.2

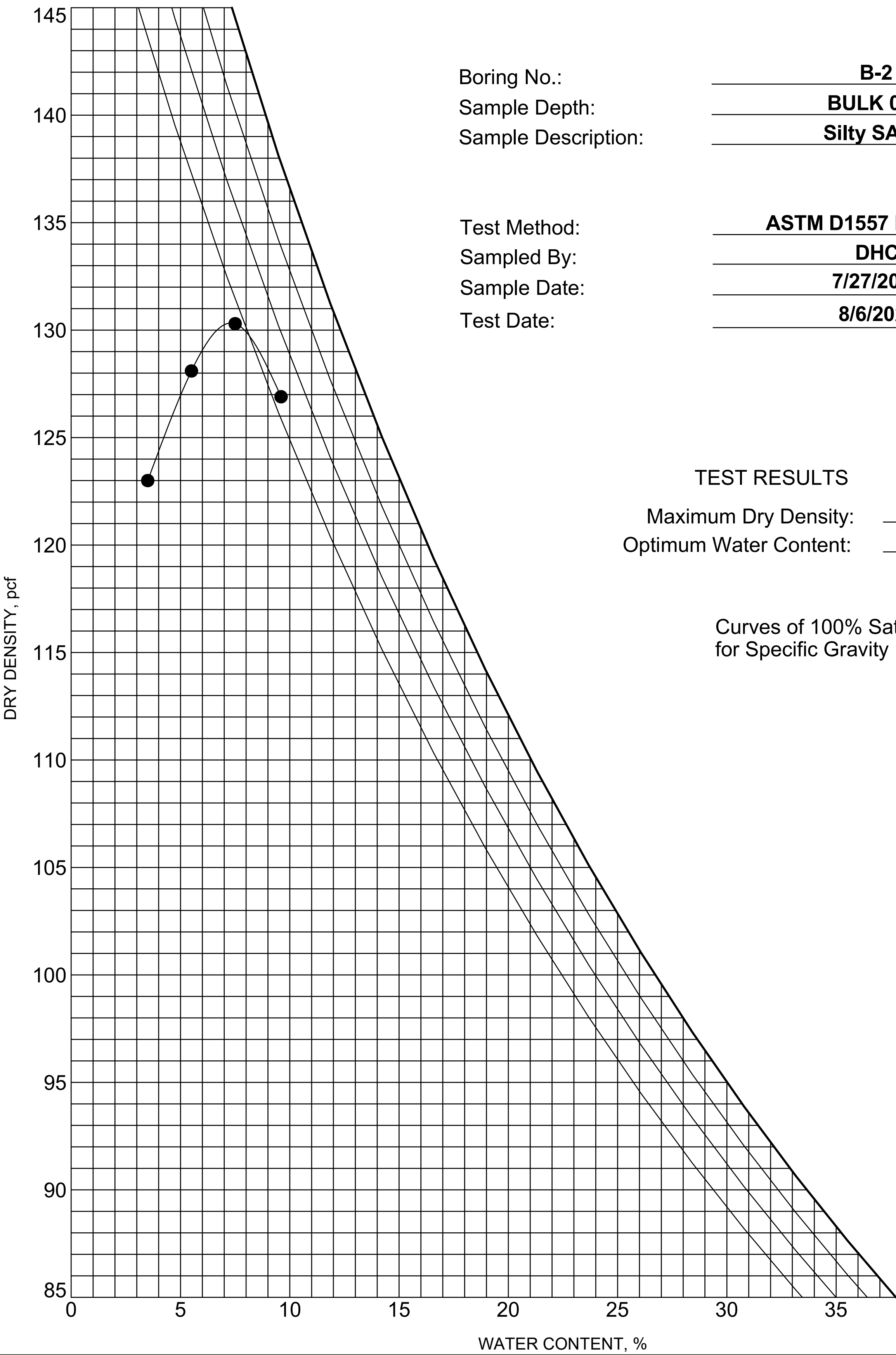
DIRECT SHEAR 200376.1 - LA CANADA FLINT CANYON WASH TRAIL.GPJ TWINING LABS.GDT 8/11/20

TWINING

DIRECT SHEAR TEST

Trail Restoration
Flint Canyon Wash Trail
La Canada Flintridge, California

PROJECT NO.	REPORT DATE	FIGURE B-8
200376.1	December 2020	



Boring No.:	**B-2**
Sample Depth:	**BULK 0-5'**
Sample Description:	**Silty SAND**

Test Method:	**ASTM D1557 Method A**
Sampled By:	**DHC**
Sample Date:	**7/27/2020**
Test Date:	**8/6/2020**

TEST RESULTS

Maximum Dry Density:	**130.5 pcf**
Optimum Water Content:	**7.5 %**

Curves of 100% Saturation
for Specific Gravity Equal to:

2.80

2.70

2.60

2.50

MOISTURE-DENSITY RELATIONSHIP

TWINING

Trail Restoration
Flint Canyon Wash Trail
La Canada Flintridge, California

PROJECT NO.	REPORT DATE	FIGURE B-9
200376.1	December 2020	

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

Hushmand Associates, Inc.
250 Goddard, Irvine,
CA 92618

p. (949) 777-1274
w. haieng.com
e. hai@haieng.com

September 18, 2020

Twining Inc.
2883 East Spring Street, Suite 300,
Long Beach, CA 90806

Attention: Mr. Doug Crayton

SUBJECT: Laboratory Test Result
Project Name: Flint Canyon
Project No.: 200376.1
HAI Project No.: TWI-20-004

Dear Mr. Crayton:

Enclosed is the result of the laboratory testing program conducted on samples from the above referenced project. The testing performed for this program was conducted in general accordance with the following test procedure:

Type of Test	Test Procedure
Triaxial (Unconsolidated, Undrained)	ASTM D2850

Attached are: two (2) Triaxial (Unconsolidated, Undrained) test results.

We appreciate the opportunity to provide our testing services to Twining Inc. If you have any questions regarding the test results, please contact us.

Sincerely,

Kang C. Lin, BS, EIT
Laboratory Manager

Woongju (MJ) Mun, PhD, PE
Senior Staff Engineer

UNCONSOLIDATED UNDRAINED (UU)
TRIAXIAL COMPRESSION TEST
ASTM D2850

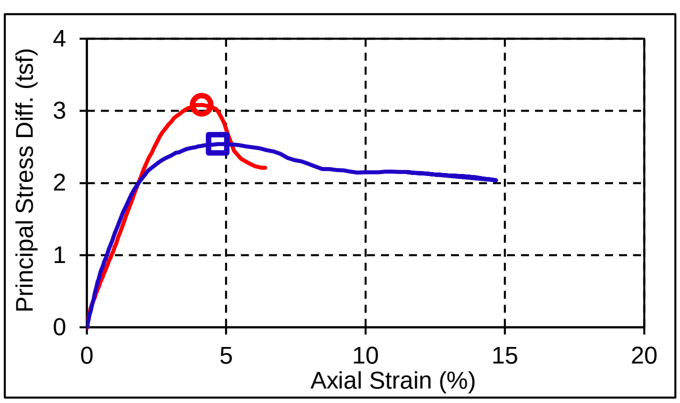
Client: Twining Inc.
Project Name: Flint Canyon
Project No.: TWI-20-004
Project Location: -
Sample Type: Undisturbed Ring

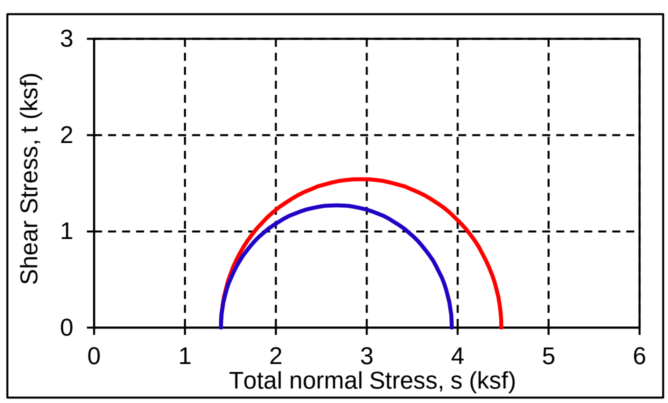
HAI Project No.: TWI-20-004
Tested By: KL
Checked by: MJ
Date: 09/10/20

Boring No.	Depth	Sample Description	Depth (ft)	Symbol	$\sigma_{1,f}$ (ksf)	$\sigma_{3,f}$ (ksf)	$\sigma_{d,f}$ (ksf)	ε_f (%)
B-3	15	Yellowish Brown, Lean Clay with Sand (CL)	15	○	4.48	1.40	3.08	4.13
B-4	15	Yellowish Brown, Lean Clay with Sand (CL)	15	□	3.93	1.39	2.54	4.70
-	-	-	-	△				

Initial Specimen Information	Sample No.	-	1	2	-	Remarks
	Height	(in.)	5.04	5.00	-	
	Diameter	(in.)	2.40	2.40	-	
	Dry Density	(pcf)	93.6	85.4	-	
	Moisture Content	(%)	23.1	14.1	-	

Failure of the Specimen





UNCONSOLIDATED UNDRAINED (UU) TRIAXIAL COMPRESSION TEST
ASTM D2850

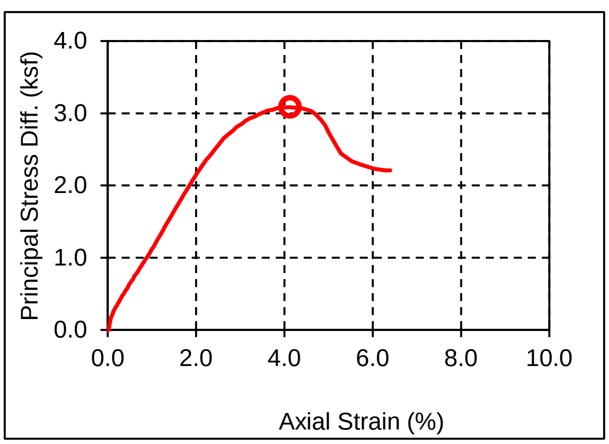
Client: Twining Inc.
Project Name: Flint Canyon
Project No.: TWI-20-004
Project Location: -
Sample Type: Undisturbed Ring

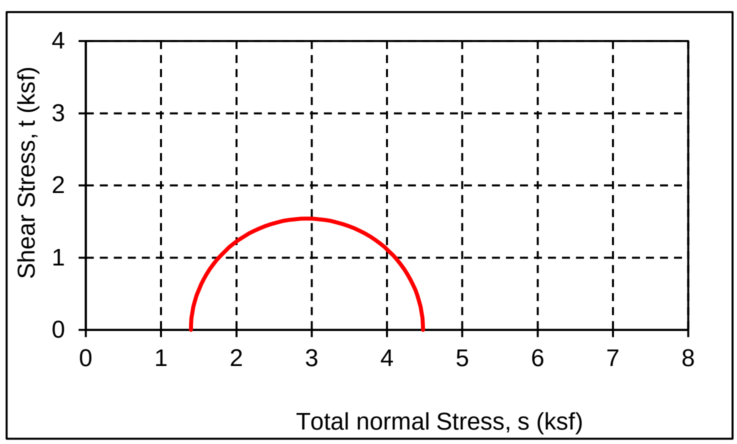
HAI Project No.: TWI-20-004
Tested By: KL
Checked by: MJ
Date: 09/10/20

Boring No.	Sample No.	Sample Description	Depth (ft)	Symbol	$\sigma_{1,f}$ (ksf)	$\sigma_{3,f}$ (ksf)	$\sigma_{d,f}$ (ksf)	ε_f (%)
B-3	-	Yellowish Brown, Lean Clay with Sand (CL)	15	○	4.48	1.40	3.08	4.13

Initial Specimen Information		
Height	5.04	in.
Diameter	2.40	in.
Dry Density	93.6	pcf
Moisture Content	23.1	%

Failure of the Specimen





Remarks:

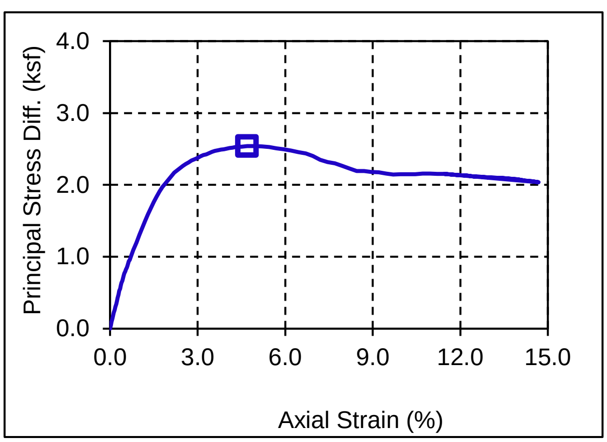
UNCONSOLIDATED UNDRAINED (UU) TRIAXIAL COMPRESSION TEST
ASTM D2850

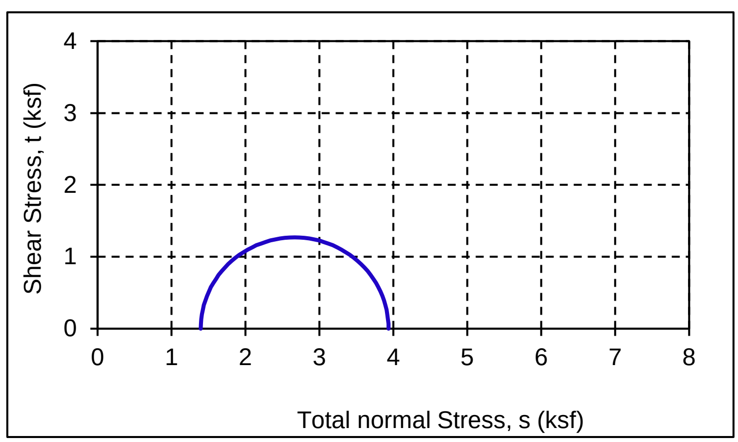
Client: Twining Inc.
Project Name: Flint Canyon
Project No.: TWI-20-004
Project Location: -
Sample Type: Undisturbed Ring

HAI Project No.: TWI-20-004
Tested By: KL
Checked by: MJ
Date: 09/10/20

Boring No.	Sample No.	Sample Description	Depth (ft)	Symbol	$\sigma_{1,f}$ (ksf)	$\sigma_{3,f}$ (ksf)	$\sigma_{d,f}$ (ksf)	ε_f (%)
B-4	-	Yellowish Brown, Lean Clay with Sand (CL)	15	□	3.93	1.39	2.54	4.70

Initial Specimen Information			Failure of the Specimen
Height	5.00	in.	
Diameter	2.40	in.	
Dry Density	85.4	pcf	
Moisture Content	14.1	%	





Remarks:

ANAHEIM TEST LAB, INC.

196 Technology Drive, Unit D
Irvine, CA 92618
Phone (949)336-6544

TWINING LABS
3310 AIRPORT WAY
LONG BEACH, CA 90806

DATE: 09/11/2020

P.O. NO: Soils 090920

LAB NO: C-4081

SPECIFICATION: CTM-643/417/422

MATERIAL: Soil

Project No.: 200376.1
Project Name: Flint Canyon
Date sampled: 07/28/2020
Sample ID: B-5, Bulk

ANALYTICAL REPORT

CORROSION SERIES
SUMMARY OF DATA

pH	MIN. RESISTIVITY per CT. 643 ohm-cm	SOLUBLE SULFATES per CT. 417 ppm	SOLUBLE CHLORIDES per CT. 422 ppm
7.7	1,800	90	81

RESPECTFULLY SUBMITTED

WES BRIDGER LAB MANAGER

TWINING

2883 East Spring Street
Suite 300
Long Beach CA 90806

Tel 562.426.3355
Fax 562.426.6424

APPENDIX C

GLOBAL SLOPE STABILITY ANALYSIS

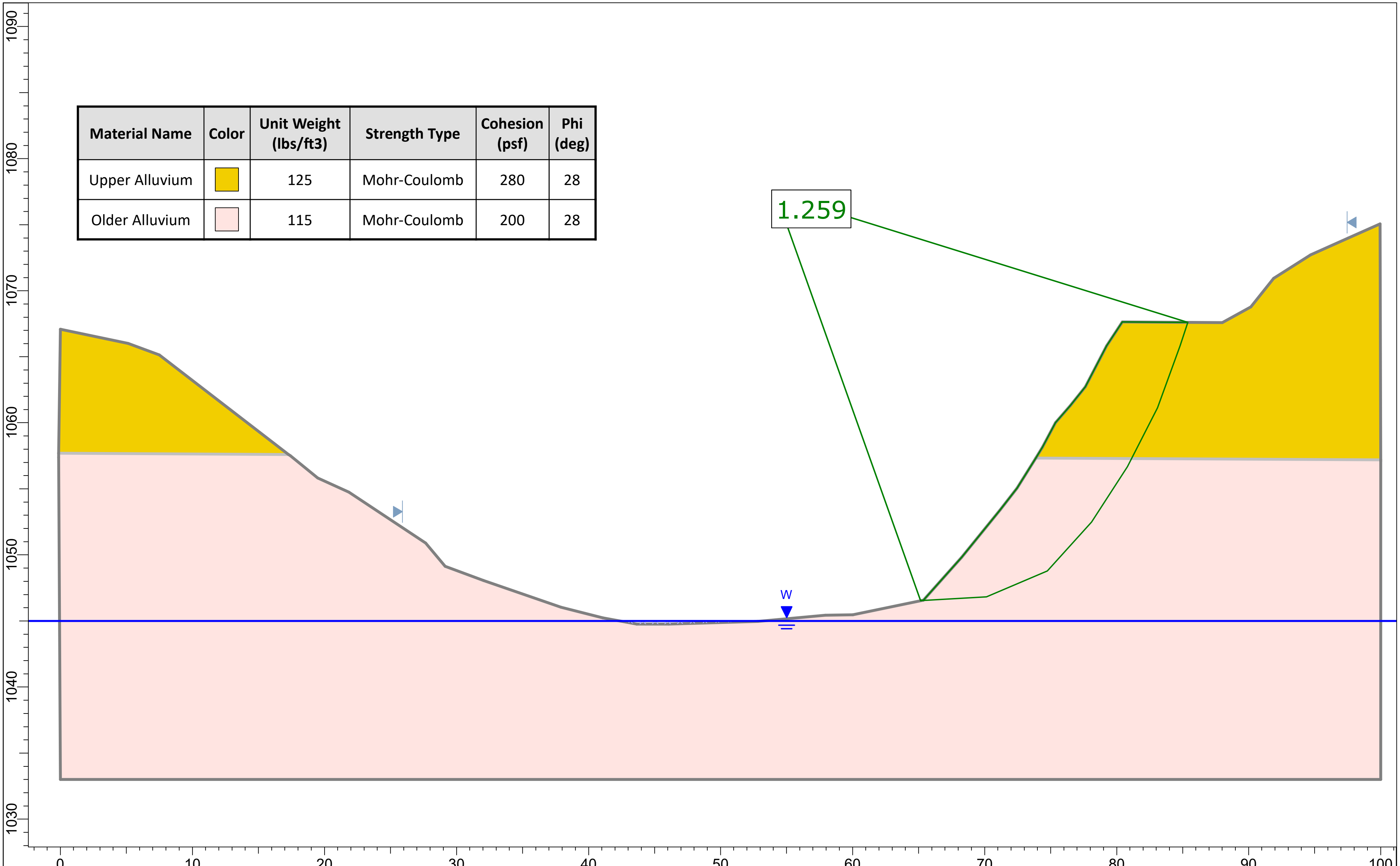
TWINING

2883 East Spring Street
Suite 300
Long Beach CA 90806

Tel 562.426.3355
Fax 562.426.6424

Appendix C
Global Slope Stability Analysis

Attached are examples of the graphical results from our analysis for the cross-sections of several stations. For each station shown, examples are provided of the output from the different scenarios analyzed such as “end-of-construction,” “rapid drawdown”, etc. The numerical results of the analyses from all of the stations have been summarized in Table 4 through Table 6.



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Upper Alluvium		125	Mohr-Coulomb	280	28
Older Alluvium		115	Mohr-Coulomb	200	28

TWINING

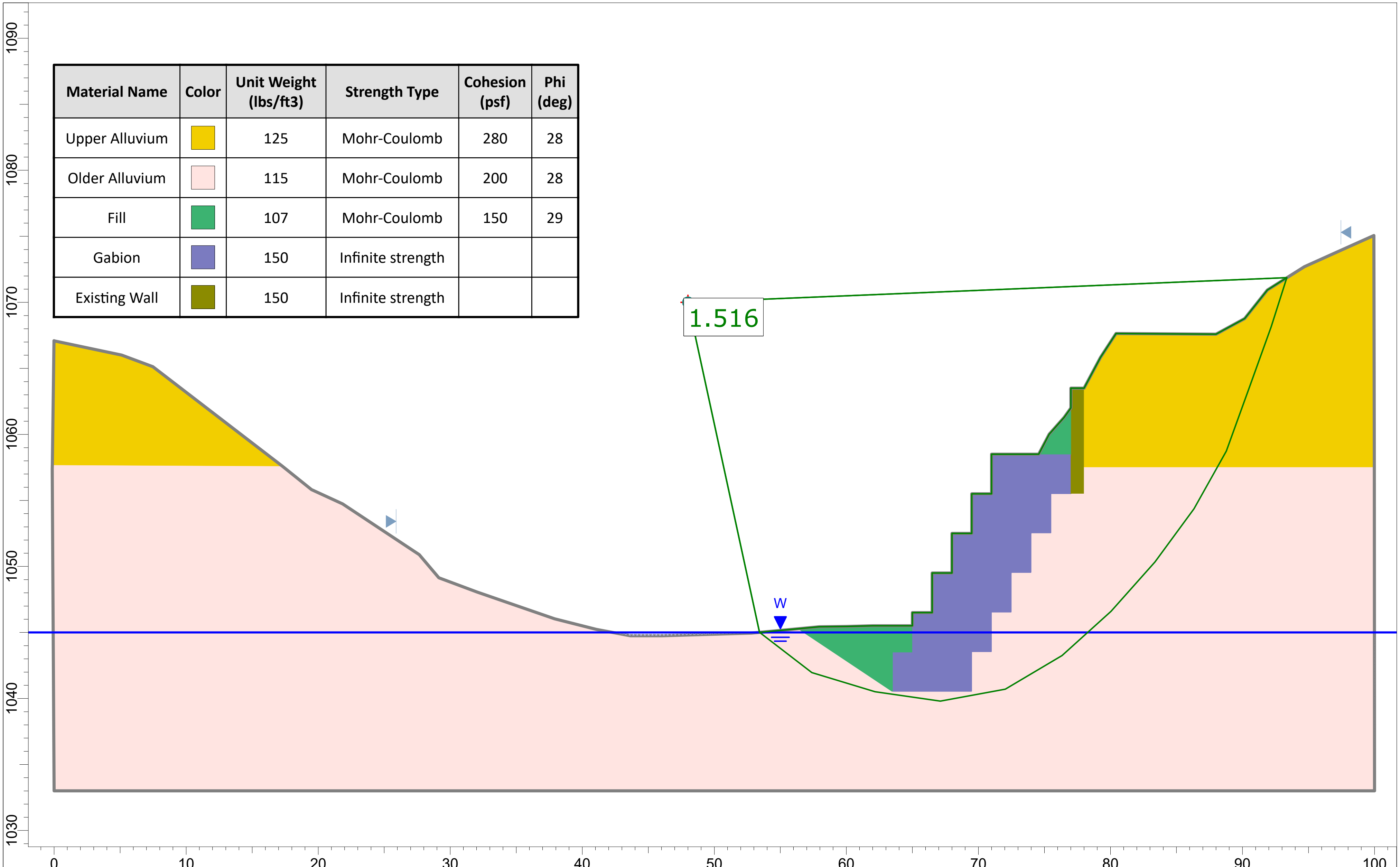
EXISTING CONDITIONS
CROSS SECTION 18+00

Horizontal axis: distance in feet
Vertical axis: elevation in feet above mean sea level

FLINT CANYON WASH TRAIL
LA CANADA FLINTRIDGE, CALIFORNIA

DATE	PROJECT NO.	FIGURE C-6
9/16/2020	200376.1	

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Upper Alluvium		125	Mohr-Coulomb	280	28
Older Alluvium		115	Mohr-Coulomb	200	28
Fill		107	Mohr-Coulomb	150	29
Gabion		150	Infinite strength		
Existing Wall		150	Infinite strength		



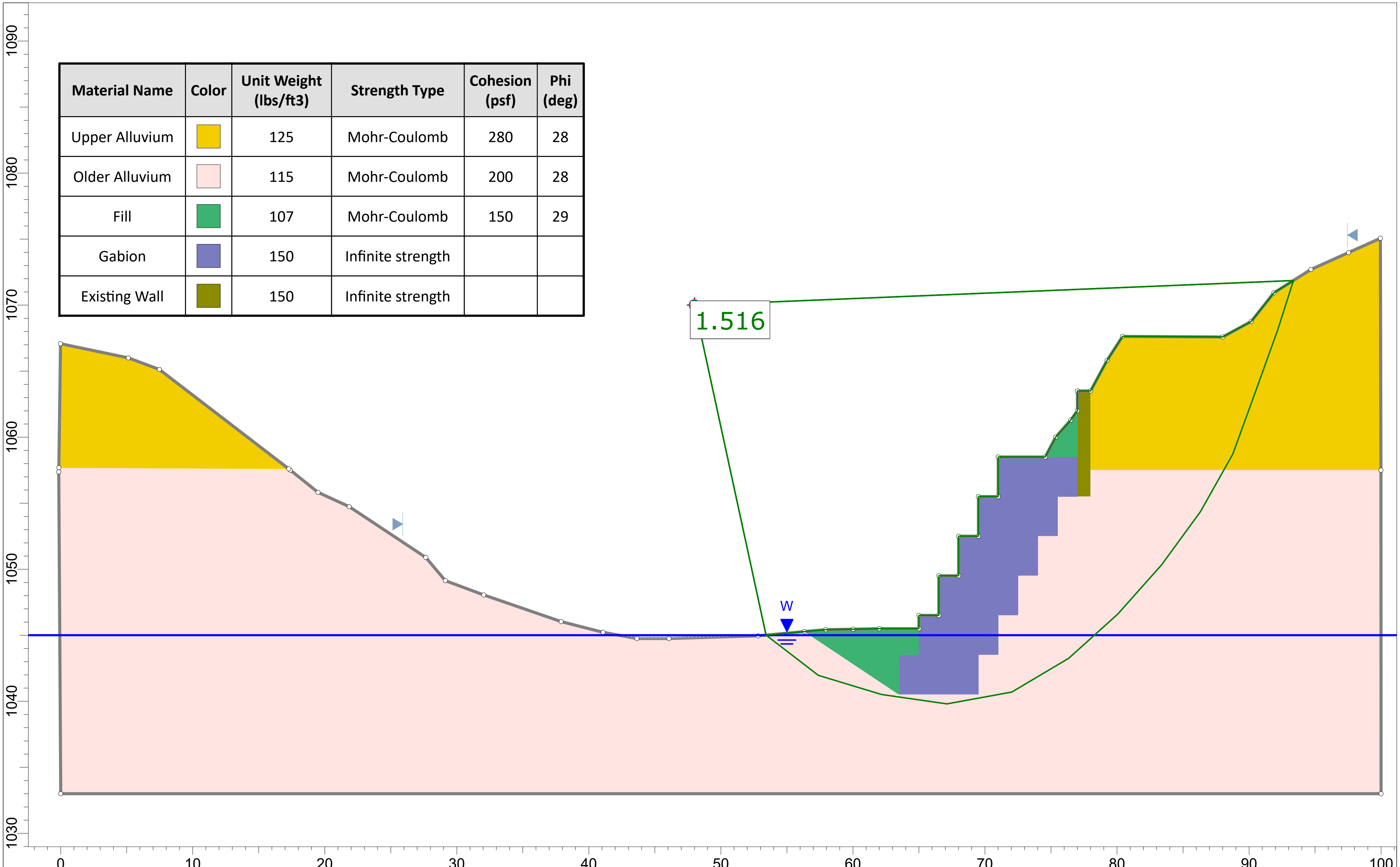
TWINING

END OF CONSTRUCTION
CROSS SECTION 18+00

Horizontal axis: distance in feet
Vertical axis: elevation in feet above mean sea level

FLINT CANYON WASH TRAIL
LA CANADA FLINTRIDGE, CALIFORNIA

DATE	PROJECT NO.	FIGURE C-7
9/16/2020	200376.1	



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Upper Alluvium		125	Mohr-Coulomb	280	28
Older Alluvium		115	Mohr-Coulomb	200	28
Fill		107	Mohr-Coulomb	150	29
Gabion		150	Infinite strength		
Existing Wall		150	Infinite strength		

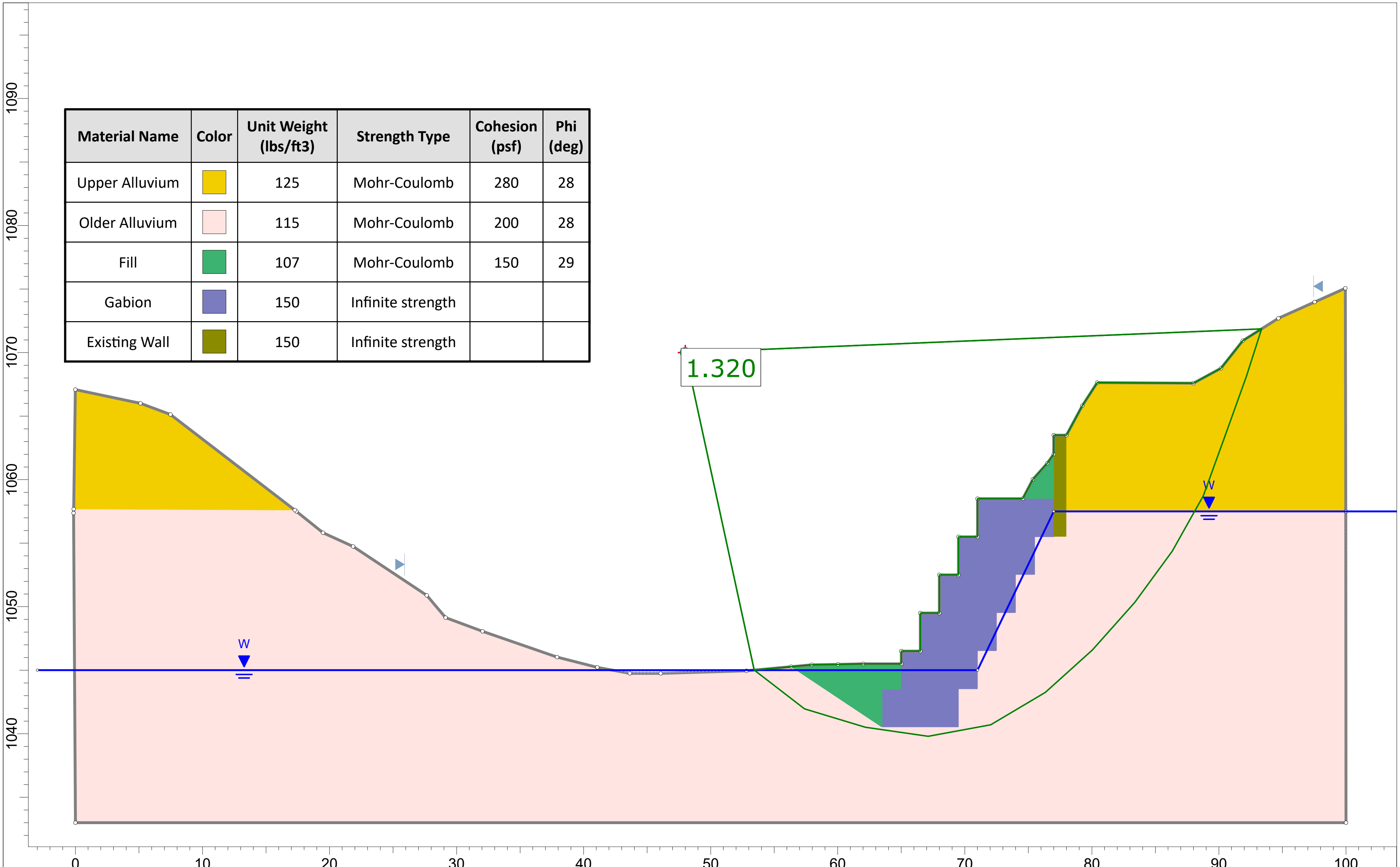
TWINING

LONG-TERM STATIC
CROSS SECTION 18+00

Horizontal axis: distance in feet
Vertical axis: elevation in feet above mean sea level

FLINT CANYON WASH TRAIL
LA CANADA FLINTRIDGE, CALIFORNIA

DATE	PROJECT NO.	
9/16/2020	200376.1	FIGURE C-8



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Upper Alluvium		125	Mohr-Coulomb	280	28
Older Alluvium		115	Mohr-Coulomb	200	28
Fill		107	Mohr-Coulomb	150	29
Gabion		150	Infinite strength		
Existing Wall		150	Infinite strength		

TWINING

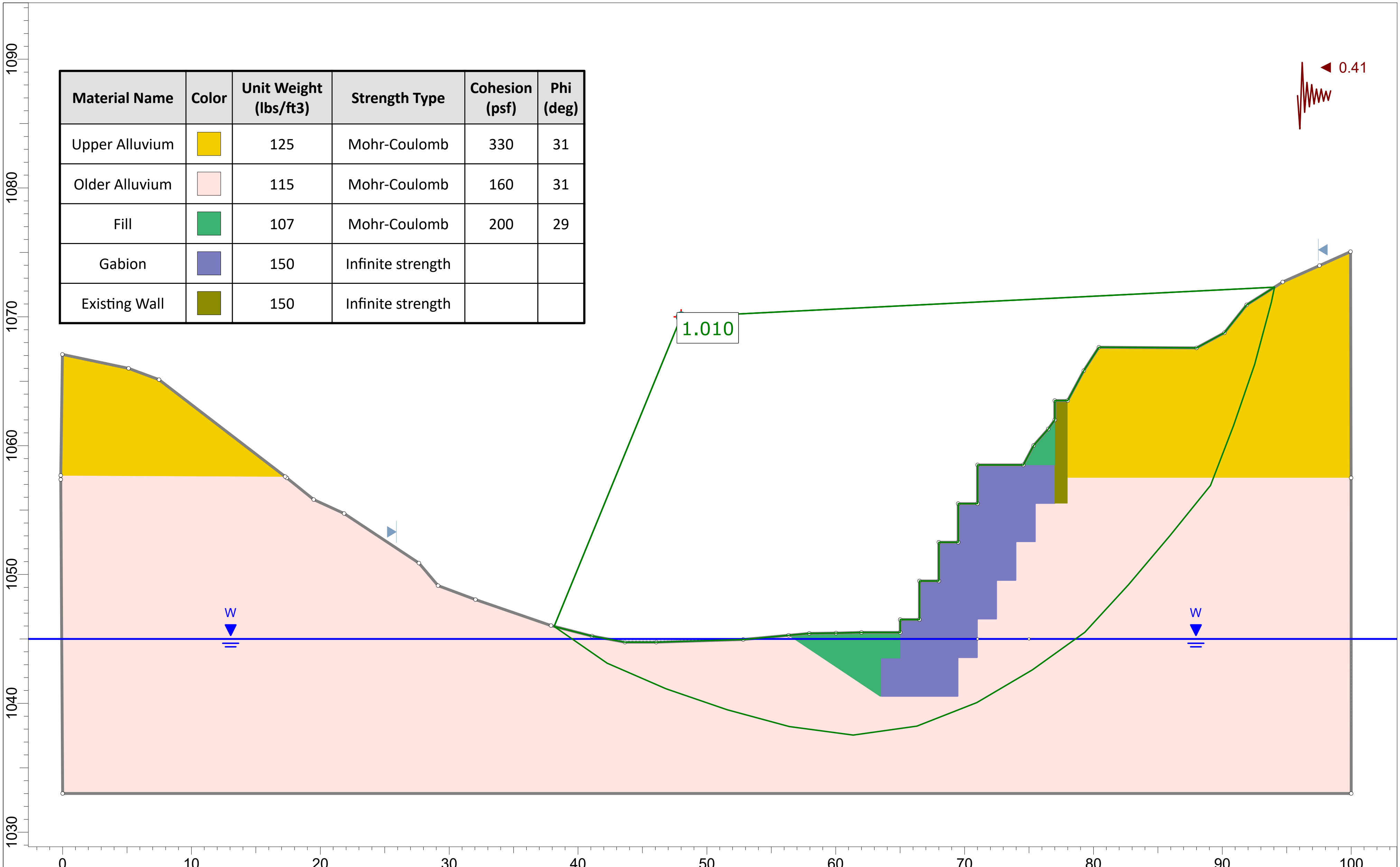
RAPID DRAWDOWN
CROSS SECTION 18+00

Horizontal axis: distance in feet
Vertical axis: elevation in feet above mean sea level

FLINT CANYON WASH TRAIL
LA CANADA FLINTRIDGE, CALIFORNIA

DATE	PROJECT NO.	
9/16/2020	200376.1	FIGURE C-9

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Upper Alluvium		125	Mohr-Coulomb	330	31
Older Alluvium		115	Mohr-Coulomb	160	31
Fill		107	Mohr-Coulomb	200	29
Gabion		150	Infinite strength		
Existing Wall		150	Infinite strength		



TWINING

SEISMIC STABILITY
CROSS SECTION 18+00

Horizontal axis: distance in feet
Vertical axis: elevation in feet above mean sea level

FLINT CANYON WASH TRAIL
LA CANADA FLINTRIDGE, CALIFORNIA

DATE	PROJECT NO.	FIGURE C-10
9/16/2020	200376.1	