Appendix E Preliminary Geotechnical Investigation Report

This Page Intentionally Left Blank

TGR

Geotechnical
Environmental
Hydrogeology
Material Testing
Construction Inspection

April 8, 2022

Project No. 22-7455

Vanita Soni Puri
1423 Georgina Ave.,
Santa Monica, CA 90402

Subject: Preliminary Geotechnical Investigation Report, Northwest Corner of E. Colton Avenue and N. Wabash Avenue, Redlands, California 92374, APN 0168-291-02.

Vanita,

In accordance with your request and authorization, TGR Geotechnical, Inc. (TGR) has performed a preliminary geotechnical investigation for the proposed development at the subject site in the city of Redlands, California. The subject site is an approximately 9-acre, undeveloped parcel of land covered in grass and vegetation. It is our understanding that the proposed development will consist of 103 single family homes with associated streets, driveways, parking, and a central common open park space. This report presents the findings of our geotechnical investigation, including site seismicity, settlement potential, infiltration rates and provides geotechnical design recommendations for the proposed improvements. The work was performed in general accordance with our proposal dated March 7, 2022.

Based on our investigation the proposed development is feasible from a geotechnical viewpoint provided the recommendations presented in this report are implemented during design and construction.

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

Respectfully submitted,

TGR GEOTECHNICAL, INC.

Robert Aguilar

Robert Aguilar
Staff Engineer

Registered Professional Engineer
Sanjay Govil
No. GE2382
EXP. 6/30/2022
Geotechnical
State of California

Registered Professional Geologist
Edward Louis Burrows
No. 1750
Certified Engineering Geologist
State of California

Sanjay Govil, PhD, PE, GE 2382
Principal Geotechnical Engineer

Edward L. Burrows, MS, PG, CEG 1750
Principal Engineering Geologist

Distribution: (1) Addressee

TGR GEOTECHNICAL
DBE & 8(a) firm
3037 S. HARBOR BLVD
SANTA ANA, CA 92704
P 714.641.7189 F 714.641.7190
www.tgrgeotech.com

ATTACHMENTS

Plate 1 – Boring Location Map

Figure 1 – Site Location Map
Figure 2 – Regional Geology Map
Figure 3 – Regional Fault Map
Figure 4 – Seismic Hazard Zone Map

Table 1 – Percolation Test Worksheet

Appendix A – References
Appendix B – Log of Borings
Appendix C – Laboratory Testing Procedures and Results
Appendix D – Site Seismic Design and Deaggregated Parameters
Appendix E – Standard Grading Specifications

INTRODUCTION

Site Descriptions and Proposed Project Development

The subject site is located on the northwest corner of E. Colton Avenue and N. Wabash Avenue in the city of Redlands, California (Figure 1). The subject site is an approximately 9-acre, undeveloped parcel of land covered in grass and vegetation. It is our understanding that the proposed development will consist of 103 single family homes with associated streets, driveways, parking, and a central common open park space. No grading plans were available at the time of this report. However, it is our understanding that minor cuts and fills will be required to reach design grades.

Scope of Work

The scope of work for this preliminary geotechnical investigation included the following:

- Site reconnaissance to assess current site conditions, mark boring locations and call Dig-Alert for utility clearance.
- Sampling and logging nine (9) borings utilizing a hollow stem drill rig to approximate depths ranging from 3 to 9 feet at the subject site to evaluate subsurface soil conditions. All borings encountered refusal due to cobbles. The borings were backfilled with cuttings and surface tamped.
- Percolation testing of the near surface soils at two (2) locations from depths of 5 to 9 feet below existing grade. The testing procedures followed the County of San Bernardino guidelines.
- Laboratory testing of selected samples to include in-situ moisture and dry density, maximum density and optimum moisture content, shear, consolidation, passing No. 200 sieve, corrosion series and R-value.
- Engineering analysis including infiltration rates, site seismicity, seismic settlement, foundation design and soils engineering/earthwork with respect to the suitability of the proposed development.
- Preparation of this report summarizing current subsurface soil conditions, findings, and presenting our recommendations for the proposed development.

Field Investigation

Field exploration was performed on March 15th, 2022 by members from our firm who logged the borings and obtained representative samples, which were subsequently transported to the laboratory for further review and testing. The approximate locations of the borings are indicated on the enclosed Boring Location Map (Plate 1).

The subsurface conditions were explored by drilling, sampling, and logging nine (9) borings with a truck mounted hollow stem auger drill rig. Borings B-1 through B-9 were advanced to approximate depths ranging from 3 to 9 feet below existing grade. All borings encountered refusal in cobbles and/or boulders. Subsequent to drilling, all borings were backfilled with excavated soil and surface tamped. The log of borings presenting soil conditions and descriptions are presented in Appendix B.

The drill rig was equipped with a sampling apparatus to allow for recovery of driven modified California Ring Sampler (CRS), 3-inch outside diameter, and 2.42-inch inside diameter and SPT samples.

The samples were driven using an automatic 140-pound hammer falling freely from a height of 30 inches. The blow counts for CRS were converted to equivalent SPT blow counts. Soil descriptions were entered on the logs in general accordance with the Unified Soil Classification System (USCS). Driven samples and bulk samples of the earth materials encountered at selected intervals were recovered from the borings. The locations and depths of the soil samples recovered are indicated on the boring logs in Appendix B.

Two (2) percolation test borings, B-5/P-1 and P-2, were advanced to an approximate depth of 9 feet below existing ground surface and percolation testing was performed at depths of approximately 5 to 9 feet below existing grade. Subsequent to percolation testing the borings were backfilled with excavated soils and surface tamped.

Percolation Testing

Upon completion of drilling and sampling Borings B-5/P-1 and P-2 were converted into a field percolation test well. Field percolation testing was performed in general accordance with the with the San Bernardino Technical Guidance for WQMP for sandy soils.

The boreholes were converted to field percolation test wells by placing approximately two inches of gravel at the bottom of the borehole, installing three-inch diameter PVC pipes and backfilling the annular space with gravel. A correction factor was applied to account for the placement of gravel.

Infiltration test rates were determined utilizing the referenced County of San Bernardino guidelines. Results of the infiltration testing are summarized in Table 1 below:

Table 1 – Infiltration Rates

Test Location	Test Depth (feet)	Infiltration Rate (Inches/hour)
B-5/P-1	5-9	10.45
P-2	5-9	7.98

Suitability Assessment Safety Factor

Factor values (v), for Factor Category A, were assigned according to the San Bernardino Technical Guidance Document for WQMP, VII.4.

Table 2 (below) presents assigned factor values and the calculated Suitability Assessment Safety Factor (Σp) in Worksheet H from the San Bernardino Technical Guidance Document for WQMP Appendix VII.

Table 2 – Worksheet H

Factor Category		Factor Description	Assigned Weight (w)	Factor Value (v)	Product (p) p = w * v
A	Suitability Assessment	Soil assessment methods	0.25	2	0.50
		Predominant soil texture	0.25	1	0.25
		Site soil variability	0.25	1	0.25
		Depth to groundwater / impervious layer	0.25	1	0.25
		Suitability Assessment Safety Factor, $S_A = \Sigma p$			1.25

The above values should be used in conjunction with Factor Category B parameters (to be determined by others) as specified in Worksheet H of the San Bernardino Technical Guidance Document for WQMP Appendix VII to evaluate the combined safety factor that should be applied to the tested infiltration rates.

Laboratory Testing

Laboratory tests were performed on representative samples to verify the field classification of the recovered samples and to evaluate the geotechnical properties of the subsurface soils. The following tests were performed:

- In-situ Moisture Content (ASTM D2216) and Dry Density (ASTM D7263);
- Maximum Dry Density and Optimum Moisture Content (ASTM D1557);
- Direct Shear Strength (ASTM D3080);
- Consolidation (ASTM D2435);
- Expansion Potential (ASTM D4829);
- Passing No. 200 Sieve (ASTM 1140);
- R-value (CAL 301); and
- Corrosion series:
  1. Soluble Sulfate (CAL.417A);
  2. Soluble Chlorides (CAL.422);
  3. Minimum Resistivity (CAL.643); and
  4. pH (CAL 747)

Laboratory tests for geotechnical characteristics were performed in general accordance with the ASTM procedures. The results of the in-situ moisture content and density tests are shown on the borings logs. The results of other laboratory tests are presented in Appendix C.

GEOTECHNICAL FINDINGS

Geology

Regional Geologic Setting

The project site is located in the east central portion of the Redlands 7.5-minute quadrangle, San Bernardino County, California. Per the Geologic Map of the Harrison Mountain/north ½ of Redlands quadrangle, California (Dibblee, 2004), the subject site is underlain by Quaternary alluvium, consisting of gravel and sand of stream channels. Figure 2 presents the Regional Geology Map.

Earth Units

Based on our subsurface investigation, the subject area is generally underlain by approximately 5 feet of light brown silty sand, with some gravel in a dry condition. The silty sand is underlain by sand, gravel and cobbles to an approximate depth of 9 feet below existing grade, the maximum depth explored. Detailed descriptions of the earth units encountered in our borings are presented in the log of the borings. (Appendix B)

Groundwater

Subsurface water was not encountered to a depth of approximately 9 feet below existing grade during the subsurface exploration.

USGS groundwater data from wells nearest to the subject site indicate a historic high groundwater of between 49 feet below existing grade and 1601 feet above NGVD 1929 (USGS 340346117080001 001S002W30C001S).

Seasonal and long-term fluctuations in the groundwater may occur as a result of variations in subsurface conditions, rainfall, run-off conditions and other factors. Therefore, variations from our observations may occur. Static groundwater is not anticipated to impact the proposed development.

Static groundwater is not anticipated to impact the proposed development.

Expansive Soil

Onsite soils have a tested expansion index of 0, correlating to a “very low” expansion potential. The recommendations provided in this report account for the expansion potential of the onsite soils.

Hydro Collapse

Laboratory testing indicates near surface soils undergo approximately 1% to 2% hydro collapse when inundated under load, correlating to a “low” potential for hydro collapse. The recommendations in this report account for the hydro collapse potential of near surface soils.

Cement Type and Corrosion

Based on laboratory testing concrete used should be designed in accordance with the provisions of ACI 318-14, Chapter 19 for Exposure Class S0: Cement with a minimum unconfined compressive strength of 2,500 psi, and for Exposure Class C1 (Moderate) – Concrete exposed to moisture but not a significant source of chlorides, per ACI 318-14 Table 19.3.1.1.

Corrosion tests indicate a mild corrosion potential for ferrous metals exposed to site soils.

TGR does not practice corrosion engineering. If needed, a qualified specialist should review the site conditions and evaluate the corrosion potential of the site soil to the proposed improvements and to provide the appropriate corrosion mitigations for the project.

Seismic Review

Faulting and Seismicity

The subject site, like the rest of Southern California, is located within a seismically active region as a result of being located near the active margin between the North American and Pacific tectonic plates. The principal source of seismic activity is movement along the northwest-trending regional faults such as the San Andreas, San Jacinto and Elsinore fault zones. These fault systems produce approximately 5 to 35 millimeters per year of slip between the plates.

We consider the most significant geologic hazard to be the potential for moderate to strong seismic shaking that is likely to occur at the subject site. The subject site is located in the highly seismic Southern California region within the influence of several faults that are considered to be Holocene-active or pre-Holocene faults. A Holocene-active fault is defined by the State of California as a fault that has exhibited surface displacement within the Holocene time (about the last 11,700 years). A pre-Holocene fault is defined by the State as a fault whose history of past movement is older than 11,700 years ago and does not meet the criteria for a Holocene-active fault.

These Holocene-active and pre-Holocene faults are capable of producing potentially damaging seismic shaking at the site. It is anticipated that the subject site will periodically experience ground acceleration as the result of small to moderate magnitude earthquakes. Other active faults without surface expression (blind faults) or other potentially active seismic sources that are not currently zoned and may be capable of generating an earthquake are known to be present under in the region.

The subject site is not included within any Earthquake Fault Zones as created by the Alquist-Priolo Earthquake Fault Zoning Act (Hart, 1997). Our review of geologic literature pertaining to the site area indicates that there are no known active or potentially active faults located within or immediately adjacent to the subject property.

The nearest fault to the subject site is the Redlands fault mapped approximately 0.7 miles southeast of the site. Other nearby faults include the Reservoir Canyon fault mapped approximately 1.6 miles to the southeast of the site, the Crafton Hills fault mapped approximately 2.9 miles southeast of the site, the Western Heights fault mapped approximately 3.1 miles southeast of the site, the South Branch San Andreas fault mapped approximately 3.1 miles northeast of the site, the Chicken Hill fault mapped approximately 4.3 miles southeast of the site, the Live Oak Canyon fault mapped approximately 4.4 miles southwest of the site, the Mill Creek fault mapped approximately 5.1 miles northeast of the site and the Loma Linda fault mapped approximately 5.6 miles to the southwest of the site. The Regional Fault Map, Figure 3, shows the location of the subject site in respect to the regional faults.

Secondary Seismic Hazards

Surface Fault Rupture and Ground Shaking

Since no known faults are located within the site, surface fault rupture is not anticipated. However, due to the close proximity of known active and potentially active faults, severe ground shaking should be expected during the life of the proposed structures.

Liquefaction

Liquefaction is a seismic phenomenon in which loose, saturated, fine-grained granular soils behave similarly to a fluid when subjected to high-intensity ground shaking. Liquefaction occurs when these ground conditions exist: 1) Shallow groundwater; 2) Low density, fine, clean sandy soils; and 3) High-intensity ground motion. Effects of liquefaction can include sand boils, settlement, and bearing capacity failures below foundations.

A review of the San Bernardino County General Plan: Geologic Hazard Overlays, Map FH31C indicates that the subject site is not located within an area mapped as having a potential for earthquake induced liquefaction (Figure 4).

Based on the above and depth to groundwater, potential for liquefaction is considered to be negligible.

Seismically Induced Settlement

Ground accelerations generated from a seismic event can produce settlements in sands or in granular earth materials both above and below the groundwater table. This phenomenon is often referred to as seismic settlement and is most common in relatively clean sands, although it can also occur in other soil materials. Based on the nature and density of site soils encountered, seismic settlement is anticipated to be negligible.

Landsliding

Landsliding involves downhill motion of earth materials during or subsequent to earth shaking. Historically, landslides triggered by earthquakes have been a significant cause of damage. Areas that are most susceptible to earthquake induced landslides are areas with steep slopes in poorly cemented or highly fractured bedrock, areas underlain by loose, weak soils, and areas on or adjacent to existing landslide deposits.

A review of the San Bernardino County General Plan: Geologic Hazard Overlays, Map FH31C, this property is not located within a mapped zone of landsliding and adjacent areas are situated on relatively flat topography. Based on the above, the general landslide susceptibility is considered to be negligible.

Lateral Spreading

Seismically induced lateral spreading involves primarily movement of earth materials due to earth shaking. Lateral spreading is demonstrated by near-vertical cracks with predominantly horizontal movement of the soil mass involved. The topography in the vicinity of the subject site is relatively flat. Based on the above and absence of liquefaction, the potential for lateral spreading at the subject site is considered very low.

DISCUSSIONS AND CONCLUSIONS

General

Based on our field exploration, laboratory testing and engineering analysis, it is our opinion that the proposed structure and proposed grading will be safe against hazard from landslide, settlement, or slippage and the proposed construction will have no adverse effect on the geologic stability of the adjacent properties provided our recommendations presented in this report are followed.

Conclusions

Based on our findings and analyses, the subject site is likely to be subjected to moderate to severe ground shaking due to the proximity of known active and potentially active faults. This may reasonably be expected during the life of the structure and should be designed accordingly.

The primary conditions affecting the proposed project site development are as follows:

- Potential for caving during excavation.
- The site is underlain by alluvium composed of gravels, cobbles, and boulders in a sandy matrix. As such, oversized materials are anticipated to be encountered during grading operations.

The engineering evaluation performed concerning site preparation and the recommendations presented are based on information provided to us and obtained by us during our office and fieldwork. This report is prepared for the development of 103 single family homes with associated streets, driveways, parking, and a central common open park space. In the event that any significant changes are made to the proposed development, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed, and the recommendations of this report are verified or modified in writing by TGR.

RECOMMENDATIONS

Seismic Design Parameters

When reviewing the 2019 California Building Code the following data should be incorporated into the design.

Parameter	Value
Latitude (degree)	34.0638
Longitude (degree)	-117.1400
Site Class	D – Stiff Soil
Site Coefficient, F_a	1.0
Site Coefficient, F_v	N/A
Mapped Spectral Acceleration at 0.2-sec Period, S_S	1.914 g
Mapped Spectral Acceleration at 1.0-sec Period, S_1	0.789 g
Spectral Acceleration at 0.2-sec Period Adjusted for Site Class, S_{MS}	2.914 g
Spectral Acceleration at 1.0-sec Period Adjusted for Site Class, S_{M1}	N/A
Design Spectral Acceleration at 0.2-sec Period, S_{DS}	1.276 g
Design Spectral Acceleration at 1.0-sec Period, S_{D1}	N/A

Site Specific Response Spectra

The USGS Unified Hazard tool, the USGS RTGM Calculator and the USGS App for Deterministic Spectra Acceleration were utilized to develop site specific ground motion spectra. The analysis was performed utilizing the following attenuation relationships that are part of NGA as required by 2019 CBC code requirements.

- Campbell & Bozorgnia (2014)
- Boore, Stewart, Seyhan & Atkinson (2014)
- Chiou & Youngs (2014)
- Abrahamson, Silva & Kamal (2014)

The results of the Site Specific Response Spectra are incorporated in Table 1 and on Figure 1 in Appendix D. The results include deterministic spectra at 5% damping, maximum rotated component at 0.84 fractile and the probabilistic spectra, maximum rotated component at 5% damping for a return period of 2475 year and subsequently multiplied by risk coefficient to obtain the MCER probabilistic spectral acceleration. The Vs30 utilized was 260 m/s.

The probabilistic response spectrum was determined using the OSHPD generated seismic values and raw output generated from the U.S. Geological Survey Unified Hazard Tool. The spectral response acceleration data generated from the U.S. Geological Survey Unified Hazard Tool was entered into the U.S. Geological Survey Risk-Targeted Ground Motion Calculator tool for each time period. The data is presented on Table 2 in Appendix D.

The deterministic response spectrum was determined using the greatest Deaggregation Contributor from the U.S. Geological Survey Unified Hazard Tool. The largest contributing fault parameters were entered into the Pacific Earthquake Engineering Research Center NGAW2 tool with a user defined sigma + 5% damping. For the deterministic analysis for the subject site, the fault utilized was the San Andreas (San Bernardino S) fault, with a characteristic magnitude M of 7.47 and a fault distance R of 5.81 km. The data is presented on Table 3 in Appendix D.

The above generated spectral accelerations were compared against the minimum code requirements in ASCE7-16 (Chapters 11 and 21) resulting in the final design response spectra which is presented in Table 1 and on Figure 1 in Appendix D.

Based on Table 1 and Figure 1, the recommended Site Specific S_{DS} and S_{D1} are as follows:

$S_{DS} = 1.211$
$S_{D1} = 1.409$

Mapped values may be used in lieu of site-specific values to design structures on Site Class D sites with an S1 greater than or equal to 0.2, provided the value of the seismic response coefficient Cs is determined by Eq. (12.8-2) for values of $T \leq 1.5T_s$ and taken as equal to 1.5 times the value computed in accordance with either Eq. (12.8-3) for $T_L \geq T > 1.5T_s$ or Eq. (12.8-4) for $T > T_L$.

The structural consultant should review the above parameters and the 2019 California Building Code to evaluate the seismic design.

Conformance to the criteria presented in the above table for seismic design does not constitute any type of guarantee or assurance that significant structural damage or ground failure will not occur during a large earthquake event. The intent of the code is “life safety” and not to completely prevent damage of the structure, since such design may be economically prohibitive.

Foundation Design Recommendations

The proposed residential structures may be supported on continuous and/or spread footings. Bearing capacity recommendations for shallow foundations are presented below. These recommendations assume that the footings will be supported on a minimum of two (2) feet of engineered fill.

For foundations supported on two (2) feet of engineered fill with minimum ninety (90) percent relative compaction at near optimum moisture content, an allowable bearing pressure of 2,500 pounds per square foot may be used in design.

The allowable bearing pressure for shallow foundations supported on minimum ninety (90) percent compacted fill shall be equal to 2,000 pounds per square foot. The recommended minimum footing depth is twelve (12) inches for single story structures and eighteen (18) inches for 2-story structures.

The minimum recommended continuous footing width is fifteen (15) inches for single story structures, eighteen (18) inches for 2-story structures and twenty-four (24) inches for pad footings. A minimum reinforcement of two (2) No. 4 steel bar top and two (2) No. 4 steel bar bottom is required for continuous footings from a geotechnical viewpoint. Foundation design details such as concrete strength, reinforcements, etc should be established by the Structural Engineer.

A one-third (1/3) increase on the aforementioned bearing pressure may be used in design for short-term wind or seismic loads.

The total and differential static settlement is anticipated to be 1 inch and 0.5 inches over 30 feet or less.

Resistance to lateral loads including wind and seismic forces may be provided by frictional resistance between the bottom of concrete and the underlying fill soils and by passive pressure against the sides of the foundations. A coefficient of friction of 0.43 may be used between concrete foundation and underlying soil. The recommended passive pressure of the engineered fill may be taken as an equivalent fluid pressure of 300 pounds per cubic foot (3,000 psf max).

Footings located near property lines where the lateral removal cannot be achieved shall be designed for a reduced bearing capacity of 1,500 pounds per square foot and the passive resistance shall be ignored.

Slab-On-Grade

Slab-on-grade should be a minimum of five (5) inches thick and reinforced with a minimum of No. 4 reinforcing bar on 18-inch centers in two horizontally perpendicular directions. Reinforcing should be properly supported to ensure placement near the vertical midpoint of the slab. "Hooking" of the reinforcement is not considered an acceptable method of positioning the steel. The slab should not be structurally connected to the buildings.

Subgrade material for the slab-on-grade should be compacted to a minimum of ninety (90) percent of the maximum laboratory dry density to a minimum depth of two (2) feet. Prior to placement of concrete, the subgrade soils should be moistened to near optimum moisture content and verified by our field representative.

The actual thickness and reinforcement of the slab shall be designed by the structural engineer per the 2019 California Building Code.

For moisture sensitive flooring, the floor slab should be underlain by an impermeable polyethylene membrane (Stego Wrap, Moistop Plus, or any equivalent meeting the requirements of ASTM D1745) as a capillary break. The membrane shall be a minimum 10-mil thick and overlain and underlain by a minimum of 2-inch thick layer of moistened (not saturated) sand to both protect the membrane and provide proper concrete curing. The polyethylene membrane joints should be lapped not less than 6 inches.

Flatwork

Flatwork should be a minimum of four (4) inches thick should be reinforced with a minimum of No. 3 reinforcing bar on 24-inch centers in two horizontally perpendicular directions. Reinforcing should be properly supported to ensure placement near the vertical midpoint of the slab. "Hooking" of the reinforcement is not considered an acceptable method of positioning the steel. The subgrade material should be compacted to a minimum of ninety (90) percent of the maximum laboratory dry density (ASTM D1557) to a minimum depth of one (1) foot. Prior to placement of concrete, the subgrade soils should be moistened to near percent of optimum moisture content and verified by our field representative. The actual thickness and reinforcement of the slab shall be designed by the structural engineer and should include the anticipated loading condition.

Retaining Wall Recommendations

The following soil parameters may be used for the design of the retaining wall with level backfill and a maximum height of six (6) feet:

Conditions	Parameters
Active (Level)	35 psf/ft
Passive	300 (maximum 3,000 psf)
Friction Coefficient	0.43

- Unrestrained retaining wall, such as a cantilever wall, the active earth pressure shall be used.
- Any import backfill shall be granular non-expansive select fill with a minimum sand equivalent of 30 The import fill should be tested and approved by TGR prior to backfill.
- An allowable coefficient of friction between properly compacted on-site fill soil and concrete of 0.43 may be used with the dead-load forces.
- Passive pressure and frictional resistance could be combined in determining the total lateral resistance. However, one of them shall be reduced by 50 percent.
- The passive pressure in the upper 6 inches of soil not confined by slabs or pavement should be neglected.

Retaining structures should be provided with a drainage system to prevent buildup of hydrostatic pressure behind the walls. Provisions should be made to collect and dispose of excess water away from the wall. Wall drainage may be provided by a perforated pipe encased in gravel or crushed rock and enclosed by geo-synthetic filter fabric. We do not recommend omitting the drains behind walls.

In addition to the above lateral forces due to retained earth, surcharge due to improvements, such as an adjacent structure, should be considered in the design of the retaining wall. A minimum vertical surcharge load of 300 psf should be used in design of walls due to adjacent traffic unless the traffic is kept at least 6 feet from the walls. Loads applied within a 1:1 projection from any surcharging structure on the stem of the wall shall be considered as lateral surcharge.

For uniform lateral surcharge conditions applied to free-to-deflect walls and restrained walls, we recommend utilizing a minimum horizontal load equal to 33 percent and 50 percent of the vertical load, respectively, and should be applied uniformly over the entire height of the wall. This horizontal load should be applied below the 1:1 projection plane. To minimize the surcharge load from an adjacent footing, deepened footings may be considered.

Retaining wall footings should have a minimum embedment of twenty-four (24) inches below the lowest adjacent grade. The retaining walls footings shall be supported on a minimum two (2) feet of compacted engineered fill compacted to a minimum ninety (90) percent relative compaction as per ASTM D1557.

Shrinkage/Subsidence

Removal and recompaction of the near surface soils is estimated to result in shrinkage ranging from 5 to 10 percent. Based on our previous experience with similar projects, additional volume loss can be anticipated due to the presence of oversized materials in the near surface soils. Minor ground subsidence is expected to occur in the soils below the zone of removal, due to settlement and machinery working. The subsidence is estimated to be between one and two tenths of a foot.

Site Development Recommendations

General

During earthwork construction, all site preparation and the general procedures of the contractor should be observed, and the fill selectively tested by a representative of TGR. If unusual or unexpected conditions are exposed in the field, they should be reviewed by this office and if warranted, modified and/or additional recommendations will be offered. During demolition of the existing buildings, large concrete slab and associated site work, voids created from removal of buried elements (footings, pipelines, septic pits, etc.) shall be backfilled with engineered fill to a minimum ninety (90) percent relative compaction per ASTM D1557 under the observation of TGR.

Grading

All grading should conform to the guidelines presented in the California Building Code (2019 edition), except where specifically superseded in the text of this report. Prior to grading, TGR's representative should be present at the pre-construction meeting to provide grading guidelines, if needed, and review any earthwork. Oversize particles may be encountered during grading. All particles greater than 4-inches shall be removed and disposed offsite.

Oversized materials may be crushed to 1" minus and mixed with onsite soil in a controlled manner as recommended by the geotechnical consultant and used as engineered fill.

The footings and slab-on-grade shall be supported on a minimum two (2) feet of engineered fill. A minimum one (1) foot of engineered fill is recommended under flatwork and pavement. Site soils may be reused as engineered fill provided, they are free of oversized particles and the recommendations presented in this report are implemented. Exposed bottoms should be scarified a minimum of 6-inches, moisture conditioned to near optimum moisture and compacted to a minimum ninety (90) percent relative compaction. Subsequently, site fill soils should be re-compacted to a minimum of ninety (90) percent relative compaction at near optimum moisture content. The lateral extent of removals beyond the building/structure/footing limits should be equal to at least 5 feet.

The depth of over-excavation should be reviewed by the Geotechnical Consultant during the actual construction. Any subsurface obstruction buried structural elements, and unsuitable material encountered during grading, should be immediately brought to the attention of the Geotechnical Consultant for proper exposure, removal and processing, as recommended.

Fill Placement

Prior to any fill placement TGR should observe the exposed surface soils. The site soils may be re-used as engineered fill provided, they are free of organic content and particle size greater than 4-inches. All particles greater than 4-inches shall be removed and disposed offsite. Fill shall be moisture conditioned to near optimum moisture and compacted to a minimum relative compaction of ninety (90) percent in accordance with ASTM D1557. Any import soils shall be non-expansive and approved by TGR Geotechnical Inc.

Compaction

Prior to fill placement, the exposed surface should be scarified to a minimum depth of six (6) inches, fill placed in eight (8) inch loose lifts moisture conditioned to near optimum moisture and compacted to a minimum relative compaction of ninety (90) percent in accordance with ASTM D1557.

Trenching

All excavations should conform to CAL-OSHA and local safety codes.

Temporary Excavation and Shoring

Due the dry, granular nature of onsite soils, all cuts shall be properly shored or sloped back to at least 1.H:1V (Horizontal: Vertical) or flatter. Some sloughing may be anticipated due to the granular nature of site soils. The exposed slope face should be kept moist (but not saturated) during construction to reduce local sloughing. No surcharge loads should be permitted within a horizontal distance equal to the height of cut from the toe of excavation unless the cut is properly shored. Excavations that extend below an imaginary plane inclined at 45 degrees below the edge of any nearby adjacent existing site facilities should be properly shored to maintain foundation support at the adjacent structures.

Utility Trench Backfill

All utility trench backfills in structural areas and beneath hardscape features should be brought to near optimum moisture content and compacted to a minimum relative compaction of ninety (90) percent of the laboratory standard. Flooding/jetting is not recommended.

Sand backfill, (unless trench excavation material), should not be allowed in parallel exterior trenches adjacent to and within an area extending below a 1:1 plane projected from the outside bottom edge of the footing. All trench excavations should minimally conform to CAL-OSHA and local safety codes. Soils generated from utility trench excavations may be used provided it is moisture conditioned and compacted to ninety (90) percent minimum relative compaction.

Drainage

Positive site drainage should be maintained at all times. Drainage should not flow uncontrolled down any descending slope or retaining wall. Water should be directed away from foundations and not allowed to pond and/or seep into the ground. Pad drainage should be directed toward the street/parking or other approved area. Roof gutters and down spouts should be utilized to control roof drainage. Down spouts should outlet a minimum of 5 feet from the proposed structure or into an approved subsurface drainage system. We would recommend that any proposed open-bottom planters adjacent to proposed structures be eliminated for a minimum distance of 10 feet. As an alternative, closed-bottom type planters could be utilized. An outlet placed in the bottom of the planter could be installed to direct drainage away from structures or any exterior concrete flatwork.

Preliminary Pavement Design

The Caltrans method of design was utilized to develop the following asphalt pavement section. The section was developed based on a tested “R-Value” for compacted site subgrade soils of 73.

Traffic indices of 4.5, 5 and 6 were assumed for use in the evaluation of the asphalt pavement sections. The traffic indices are subject to approval by controlling authorities and shall be approved by the project civil engineer.

ASPHALT PAVEMENT SECTION			
Traffic Index	Asphalt (Inch)	Aggregate Base (Inch)	Total (Inch)
4.5	3.0	4.0	7.0
5.0	3.0	6.0	9.0
6.0	4.0	6.0	10.0

Aggregate base material for Asphalt Pavement should consist of CAB/CMB complying with the specifications in Section 200-2.2/200-2.4 of the current “Standard Specifications for Public Works Construction” and should be compacted to at least ninety-five (95) percent of the maximum dry density (ASTM D1557). The surface of the base should exhibit a firm and unyielding condition just prior to the placement of asphalt concrete paving. The asphalt concrete shall be compacted to a minimum of ninety-five (95) percent relative compaction.

The pavement subgrade should be constructed in accordance with the recommendations presented in the grading section of this report.

The R-value and the associated pavement section should be confirmed at the completion of site grading.

Geotechnical Review of Plans

All grading and foundation plans should be reviewed and accepted by the geotechnical consultant prior to construction. If significant time elapses since preparation of this report, the geotechnical consultant should verify the current site conditions, and provide any additional recommendations (if necessary) prior to construction.

Geotechnical Observation/Testing During Construction

Per sections 1705.6 and table 1705.6 of the 2019 California Building Code, periodic special inspection shall be performed to:

- Verify materials below shallow foundations are adequate to achieve the design bearing capacity;
- Verify excavations are extended to the proper depth and have reached proper material;
- Verify classification and test compacted materials; and
- Prior to placement of compacted fill, inspect subgrade and verify that the site has been prepared properly.

Per sections 1705.6 and table 1705.6 of the 2019 California Building Code, continuous special inspection shall be performed to:

- Verify use of proper materials, densities and lift thickness during placement and compaction of compacted fill.

The geotechnical consultant should also perform observation and/or testing at the following stages:

- During any grading and fill placement;
- After foundation excavation and prior to placing concrete;
- Prior to placing slab and flatwork concrete;
- During placement of aggregate base and asphalt or Portland cement concrete; and
- When any unusual soil conditions are encountered during any construction operation subsequent to issuance of this report.

Limitations

This report was prepared for a specific client and a specific project, based on the client's needs, directions and requirements at the time.

This report was necessarily based upon data obtained from a limited number of observances, site visits, soil and/or other samples, tests, analyses, histories of occurrences, spaced subsurface exploration and limited information on historical events and observations. Such information is necessarily incomplete. Variations can be experienced within small distances and under various climatic conditions. Changes in subsurface conditions can and do occur over time.

This report is not authorized for use by and is not to be relied upon by any party except the client with whom TGR contracted for the work. Use or reliance on this report by any other party is that party's sole risk. Unauthorized use of or reliance on this report constitutes an agreement to defend and indemnify TGR from and against any liability which may arise as a result of such use or reliance, regardless of any fault, negligence, or strict liability of TGR.

B-9
⊕ APPROXIMATE LOCATION OF EXPLORATORY BORING

P-2
○ APPROXIMATE LOCATION OF PERCOLATION BORING

TGR
Geotechnical
Environmental
Hydrogeology
Material Testing
Construction Inspection

BORING LOCATION MAP
NW CORNER OF E. COLTON AVENUE AND N. WABASH AVENUE
REDLANDS, CALIFORNIA

PROJECT NO. 22-7455

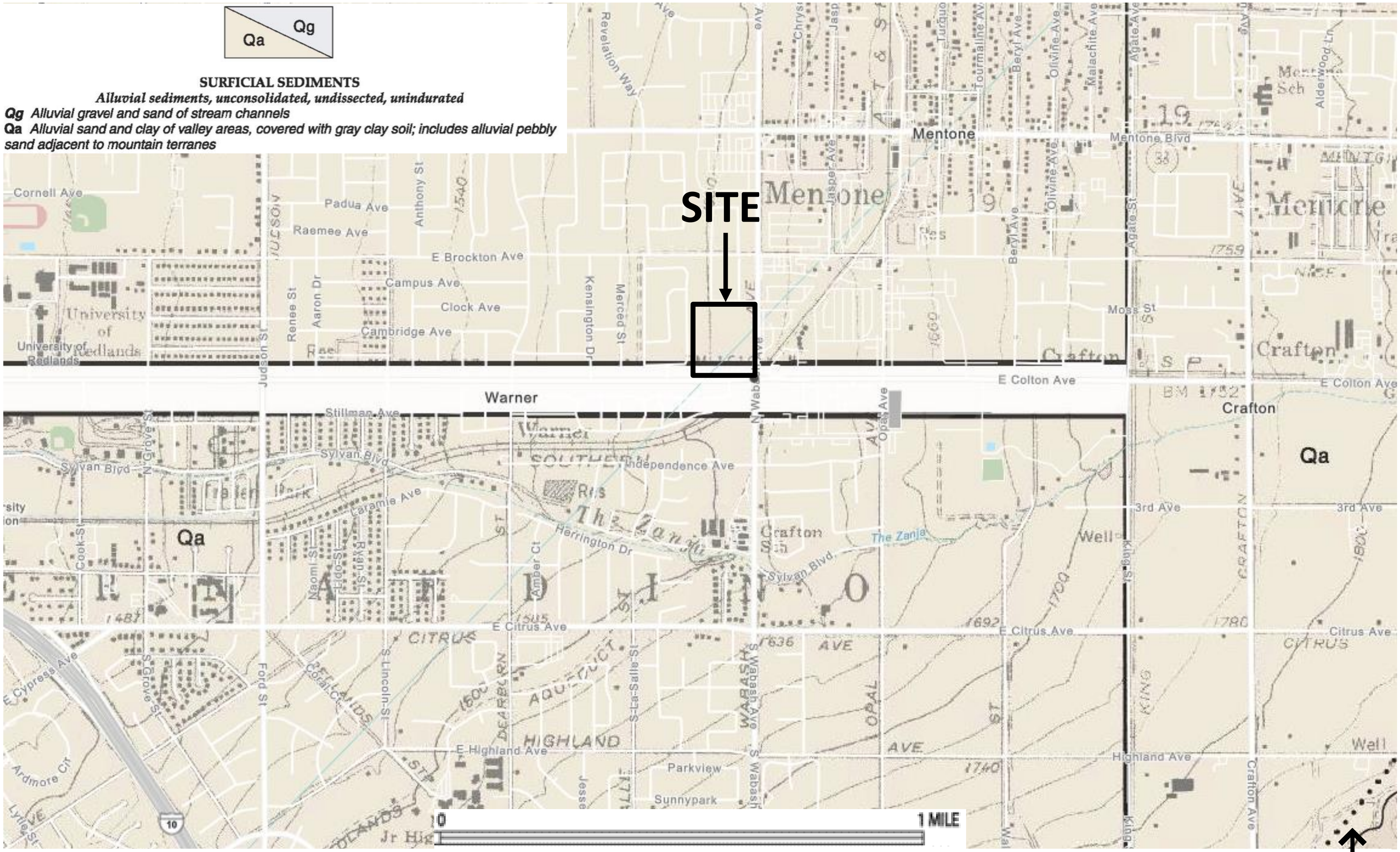
PLATE 1

Geotechnical
Environmental
Hydrogeology
Material Testing
Construction Inspection

SITE LOCATION MAP
NW CORNER OF E. COLTON AVENUE AND N. WABASH AVENUE
REDLANDS, CALIFORNIA

PROJECT NO. 22-7455

FIGURE 1



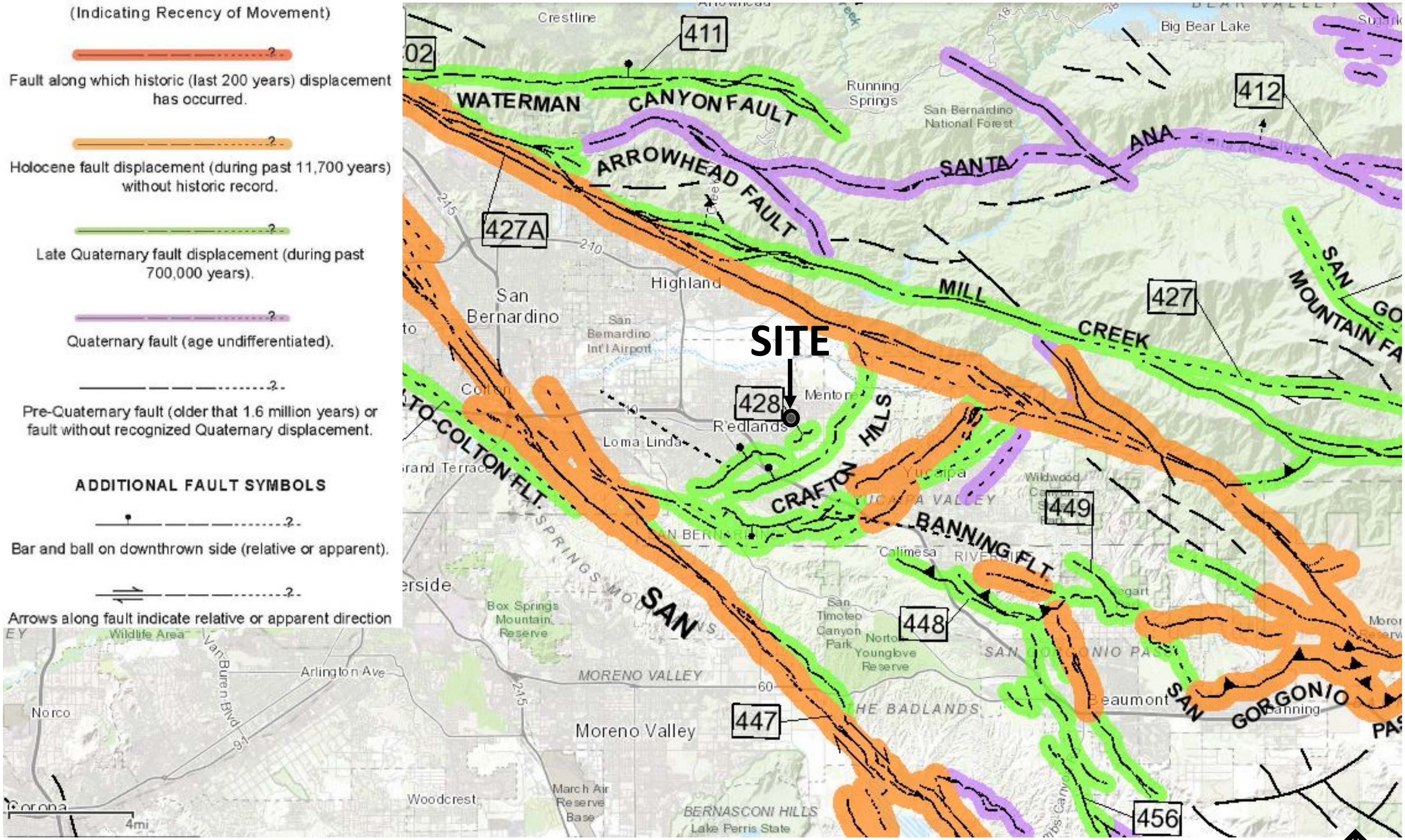
Modified From: Dibblee, T.W., and Minch, J.A., 2004, Geologic map of the Harrison Mountain/north 1/2 of Redlands quadrangles, San Bernardino and Riverside County, California: Dibblee Geological Foundation, DF-126, scale 1:24,000.

Geotechnical
Environmental
Hydrogeology
Material Testing
Construction Inspection

REGIONAL GEOLOGY MAP
NW CORNER OF E. COLTON AVENUE AND N. WABASH AVENUE
REDLANDS, CALIFORNIA

PROJECT NO. 22-7455

FIGURE 2



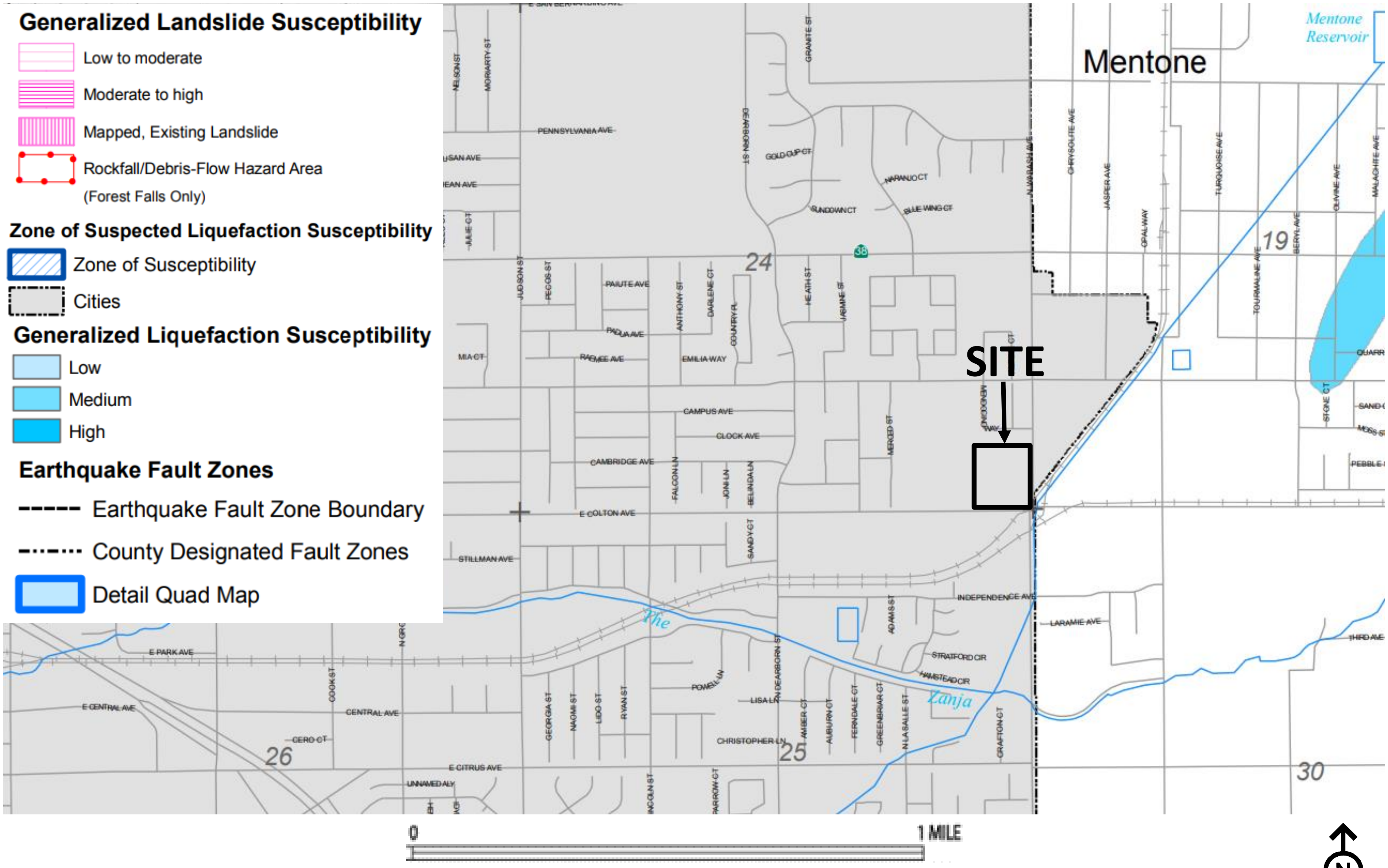
Modified From: Jennings, C. W., 2010, Fault Activity Map of California and Adjacent Areas, California Division of Mines and Geology, Geologic Data Map Series, No. 6, Scale 1:750,000.

TGR Geotechnical Environmental Hydrogeology Material Testing Construction Inspection

REGIONAL FAULT MAP
NW CORNER OF E. COLTON AVENUE AND N. WABASH AVENUE
REDLANDS, CALIFORNIA

PROJECT NO. 22-7455

FIGURE 3



Modified From: County of Sand Bernardino, Land Use Services, Geologic Hazard Maps Overlay, Map FH31C.

TGR
Geotechnical
Environmental
Hydrogeology
Material Testing
Construction Inspection

GEOLOGIC HAZARDS MAP
NW CORNER OF E. COLTON AVENUE AND N. WABASH AVENUE
REDLANDS, CALIFORNIA

PROJECT NO. 22-7455

FIGURE 4

Table 1: Percolation Test Worksheet

Test Hole	Total Depth (in)	Initial Depth (in)	Final Depth (in)	∆Water Level (in)	Initial Time (min)	Final Time (min)	∆ Time (min)	Initial Height of Water (in)	Final Height of Water (in)	Average Height of Water (in)	Gravel Factor	Infiltration Rate (in/hr)
P-1/B-5	108	70.20	96.96	26.76	0.0	5.0	5.0	37.80	11.04	24.42	0.54	13.13
	108	62.76	89.88	27.12	0.0	5.0	5.0	45.24	18.12	31.68	0.54	10.44
	108	63.60	91.44	27.84	0.0	5.0	5.0	44.40	16.56	30.48	0.54	11.11
	108	62.64	89.76	27.12	0.0	5.0	5.0	45.36	18.24	31.80	0.54	10.40
	108	64.08	91.32	27.24	0.0	5.0	5.0	43.92	16.68	30.30	0.54	10.93
	108	63.00	90.00	27.00	0.0	5.0	5.0	45.00	18.00	31.50	0.54	10.45
P-2	108	64.44	92.52	28.08	0.0	5.0	5.0	43.56	15.48	29.52	0.54	11.55
	108	64.56	89.88	25.32	0.0	5.0	5.0	43.44	18.12	30.78	0.54	10.01
	108	63.84	85.20	21.36	0.0	5.0	5.0	44.16	22.80	33.48	0.54	7.80
	108	62.76	84.36	21.6	0.0	5.0	5.0	45.24	23.64	34.44	0.54	7.68
	108	64.20	84.96	20.76	0.0	5.0	5.0	43.80	23.04	33.42	0.54	7.60
	108	63.60	85.44	21.84	0.0	5.0	5.0	44.40	22.56	33.48	0.54	7.98

$$I_t = \frac{\Delta H(60r)}{\Delta t(r + 2H_{avg})}$$

ΔH = Change in height

Δt = Time interval

r = Radius

I_t Infiltration Rate

H_{ave} Average Head Height over the time interval

APPENDIX A
REFERENCES

APPENDIX A

References

California Department of Conservation – California Geological Survey, 2018, Earthquake Fault Zones, A Guide for Government Agencies, Property Owners/Developers and Geoscience Practitioners for Assessing Fault Rupture Hazards in California, Special Publication 42

California Department of Conservation – Division of Mines and Geology, 2008, Guidelines for Evaluating and Mitigating Seismic Hazards in California, CDMG Special Publication 117A.

California Department of Conservation – Division of Mines and Geology, 1998, Maps of Known Active Fault Near – Source Zones in California and Adjacent Portions of Nevada.

County of San Bernardino, 2013, Technical Guidance Document for Water Quality Management Plans, The County of San Bernardino Areawide Stormwater Program, Effective Date: September 19, 2013.

County of San Bernardino, 2011, Technical Guidance Document Appendices, Appendix VII, Infiltration Rate Evaluation Protocol and Factor of Safety Recommendation, dated May 19, 2011.

County of San Bernardino, 2010, San Bernardino County Land Use Plan, General Plan, Geologic Hazard Overlays, San Bernardino County, California, FH31 Redlands.

Dibblee, T.W., and Minch, J.A., 2004, Geologic map of the Harrison Mountain/north 1/2 of Redlands quadrangles, San Bernardino and Riverside County, California: Dibblee Geological Foundation, DF-126, scale 1:24,000.

International Code Council (ICC), California Building Code, 2019 Edition.

Jennings, C. W., 2010, Fault Activity Map of California and Adjacent Areas, California Division of Mines and Geology, Geologic Data Map Series, No. 6, Scale 1:750,000.

APPENDIX B
LOG OF BORINGS

THE FOLLOWING DESCRIBES THE TERMS AND SYMBOLS USED ON THE LOG OF BORINGS TO SUMMARIZE THE RESULTS OBTAINED IN THE FIELD INVESTIGATION AND SUBSEQUENT LABORATORY TESTING

DENSITY AND CONSISTENCY

The consistency of fine grained soils and the density of coarse grained soils are described on the basis of the Standard Penetration Test as follows:

COARSE GRAINED SOILS		ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (Tsf)	FINE GRAINED SOILS	
Very Loose	< 4	< 0.25	Very Soft	< 2
Loose	4 – 10	0.35 – 0.50	Soft	2 – 4
Medium	10 – 30	0.50 – 1.0	Firm (Medium)	4 – 8
Dense	30 – 50	1.0 – 2.0	Stiff	8 – 15
Very Dense	> 50	2.0 – 4.0	Very Stiff	15 – 30
		> 4.0	Hard	> 30

PARTICLE SIZE DEFINITION (As per ASTM D2487 and D422)

Boulder	⇒ Larger than 12 inches	Coarse Sands	⇒ No. 10 to No. 4 sieve
Cobbles	⇒ 3 to 12 inches	Medium Sands	⇒ No. 40 to No. 10 sieve
Coarse Gravel	⇒ 3/4 to 3 inches	Fine Sands	⇒ No. 200 to 40 sieve
Fine Gravel	⇒ No. 4 to 3/4 inches	Silt	⇒ 5µm to No. 200 sieve
		Clay	⇒ Smaller than 5µm

SOIL CLASSIFICATION

Soils and bedrock are classified and described based on their engineering properties and characteristics using ASTM D2487 and D2488.

Percentage description of minor components:

Trace	1 – 10%	Some	20 – 35%
Little	10 – 20%	And or y	25 – 50%

Stratified soils description:

Parting	0 to 1/16 inch thick	Layer	½ to 12 inches thick
Seam	1/16 to ½ inch thick	Stratum	> 12 inches thick

Geotechnical
Environmental
Hydrogeology
Material Testing
Construction Inspection

LOG OF BORING EXPLANATION

SOIL CLASSIFICATION CHART

UNIFIED SOIL CLASSIFICATION AND SYMBOL CHART

COARSE-GRAINED SOILS (more than 50% of material is larger than No. 200 sieve size.)			
	Clean Gravels (Less than 5% fines)		
GRAVELS More than 50% of coarse fraction larger than No. 4 sieve size	▨	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 (More than 12% fines)		
	▨	GM	Silty gravels, gravel-sand-silt mixtures
	▨	GC	Clayey gravels, gravel-sand-clay mixtures
	Clean Sands (Less than 5% fines)		
SANDS 50% or more of coarse fraction smaller than No. 4 sieve size	▨	SW	Well-graded sands, gravelly sands, little or no fines
	▨	SP	Poorly graded sands, gravelly sands, little or no fines
	Sands with fines (More than 12% fines)		
	▨	SM	Silty sands, sand-silt mixtures
	▨	SC	Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS (50% or more 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 of 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 50% or greater	▨	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
	▨	CH	Inorganic clays of high plasticity, fat clays
	▨	OH	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS	▨	PT	Peat and other highly organic soils

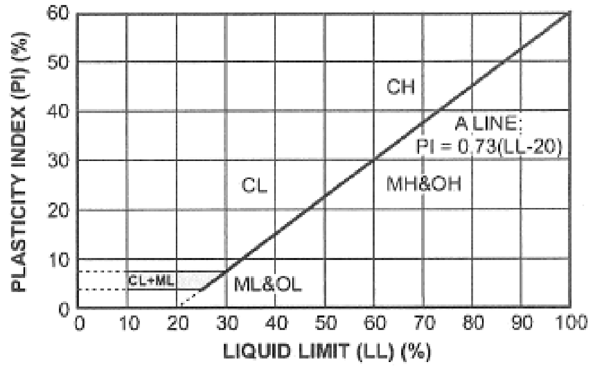
LABORATORY CLASSIFICATION CRITERIA

GW	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{D_{30}}{D_{10} \times D_{60}}$ between 1 and 3	
GP	Not meeting all gradation requirements for GW	
GM	Atterberg limits below "A" line or P.I. less than 4	Above "A" line with P.I. between 4 and 7 are borderline cases requiring use of dual symbols
GC	Atterberg limits above "A" line with P.I. greater than 7	
SW	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{D_{30}}{D_{10} \times D_{60}}$ between 1 and 3	
SP	Not meeting all gradation requirements for GW	
SM	Atterberg limits below "A" line or P.I. less than 4	Limits plotting in shaded zone with P.I. between 4 and 7 are borderline cases requiring use of dual symbols.
SC	Atterberg limits above "A" line with P.I. greater than 7	

Determine percentages of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows:

Less than 5 percent .... GW, GP, SW, SP
More than 12 percent .... GM, GC, SM, SC
5 to 12 percent .... Borderline cases requiring dual symbols

PLASTICITY CHART



PARTICLE SIZE LIMITS

COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

3" ¾" NO. 4 NO. 10 NO. 40 NO. 200

Geotechnical
Environmental
Hydrogeology
Material Testing
Construction Inspection

LOG OF BORING EXPLANATION

LOG OF EXPLORATORY BORING B-1

Sheet **1** of **1**

Project Number: **22-7455**
Project Name: **Colton Ave. and Wabash Ave., Redlands**
Date Drilled: **3/15/22 - 3/15/22**
Ground Elev: **1604**

Logged By: **RA**
Project Engineer: **SG**
Drill Type: **Hollow Stem**
Drive Wt & Drop: **140lbs / 30in**

■ Shelby Tube
☒ Standard Split Spoon
◉ No recovery
⧗ Modified California
▼ Water Table ATD

Elevation (ft)	Depth (ft)	Graphic Log	FIELD RESULTS Bulk Sample	Drive Sample	SPT blows/ft (or equivalent N)	Pocket Pen (tsf)	USCS	SUMMARY OF SUBSURFACE CONDITIONS	LAB RESULTS Moisture Content (%)	Dry Density, (pcf)	Other Tests
								Surface is grass and vegetation.			
								NATIVE: Silty SAND- light brown, dry, medium dense, very fine to fine grained sand.			
1600											
	5			⧗							
					39		SP	SAND- grey and white, dry, dense, fine to coarse grained, fine to coarse grained gravel, cobbles.	1	117	Consol
								Total Depth: 8 feet due to refusal in cobbles. No groundwater encountered during drilling. No caving observed. Boring backfilled with soil cuttings upon completion. Ground elevation estimated with Google Earth.			
1595											
	10										
1590											

This Boring Log should be evaluated in conjunction with the complete geotechnical report. This Boring Log represents conditions observed at the specific location and date indicated, it is not warranted to be representative of subsurface conditions at other locations and times.

PLATE 2

TGR GEOTECHNICAL, INC.

LOG OF EXPLORATORY BORING B-2

Sheet **1** of **1**

Project Number: **22-7455**
Project Name: **Colton Ave. and Wabash Ave., Redlands**
Date Drilled: **3/15/22 - 3/15/22**
Ground Elev: **1606**

Logged By: **RA**
Project Engineer: **SG**
Drill Type: **Hollow Stem**
Drive Wt & Drop: **140lbs / 30in**

■ Shelby Tube
☒ Standard Split Spoon
◉ No recovery
⧗ Modified California
▼ Water Table ATD

Elevation (ft)	Depth (ft)	Graphic Log	FIELD RESULTS					SUMMARY OF SUBSURFACE CONDITIONS	LAB RESULTS		
			Bulk Sample	Drive Sample	SPT blows/ft (or equivalent N)	Pocket Pen (tsf)	USCS		Moisture Content (%)	Dry Density, (pcf)	Other Tests
1606	0		▬					Surface is grass and vegetation.			
1605	1		▬					NATIVE: Silty SAND- light brown, dry, stiff, very fine to fine grained sand, some fine to coarse grained gravel.			EI, Corrosion, R-Value
1601	5			⧗	46		SM	...Same as above, some cobbles.	2	109	
1600	6										
	6.5							Total Depth: 6.5 feet due to refusal in cobbles. No groundwater encountered during drilling. No caving observed. Boring backfilled with soil cuttings upon completion. Ground elevation estimated with Google Earth.			
	10										
1595											

This Boring Log should be evaluated in conjunction with the complete geotechnical report. This Boring Log represents conditions observed at the specific location and date indicated, it is not warranted to be representative of subsurface conditions at other locations and times.

PLATE 3

TGR GEOTECHNICAL, INC.

LOG OF EXPLORATORY BORING B-3

Sheet **1** of **1**

Project Number:	**22-7455**	Logged By:	**RA**
Project Name:	**Colton Ave. and Wabash Ave., Redlands**	Project Engineer:	**SG**
Date Drilled:	**3/15/22 - 3/15/22**	Drill Type:	**Hollow Stem**
Ground Elev:	**1611**	Drive Wt & Drop:	**140lbs / 30in**

■ Shelby Tube
☒ Standard Split Spoon
◉ No recovery
⧗ Modified California
▼ Water Table ATD

Elevation (ft)	Depth (ft)	Graphic Log	FIELD RESULTS					SUMMARY OF SUBSURFACE CONDITIONS	LAB RESULTS		
			Bulk Sample	Drive Sample	SPT blows/ft (or equivalent N)	Pocket Pen (tsf)	USCS		Moisture Content (%)	Dry Density, (pcf)	Other Tests
								Surface is grass and vegetation.			
								NATIVE: Silty SAND- light brown, dry, stiff, very fine to fine grained sand, some fine to coarse grained gravel.			
1610											
								Total Depth: 3 feet due to refusal in cobbles. No groundwater encountered during drilling. No caving observed. Boring backfilled with soil cuttings upon completion. Ground elevation estimated with Google Earth.			
	5										
1605											
	10										
1600											

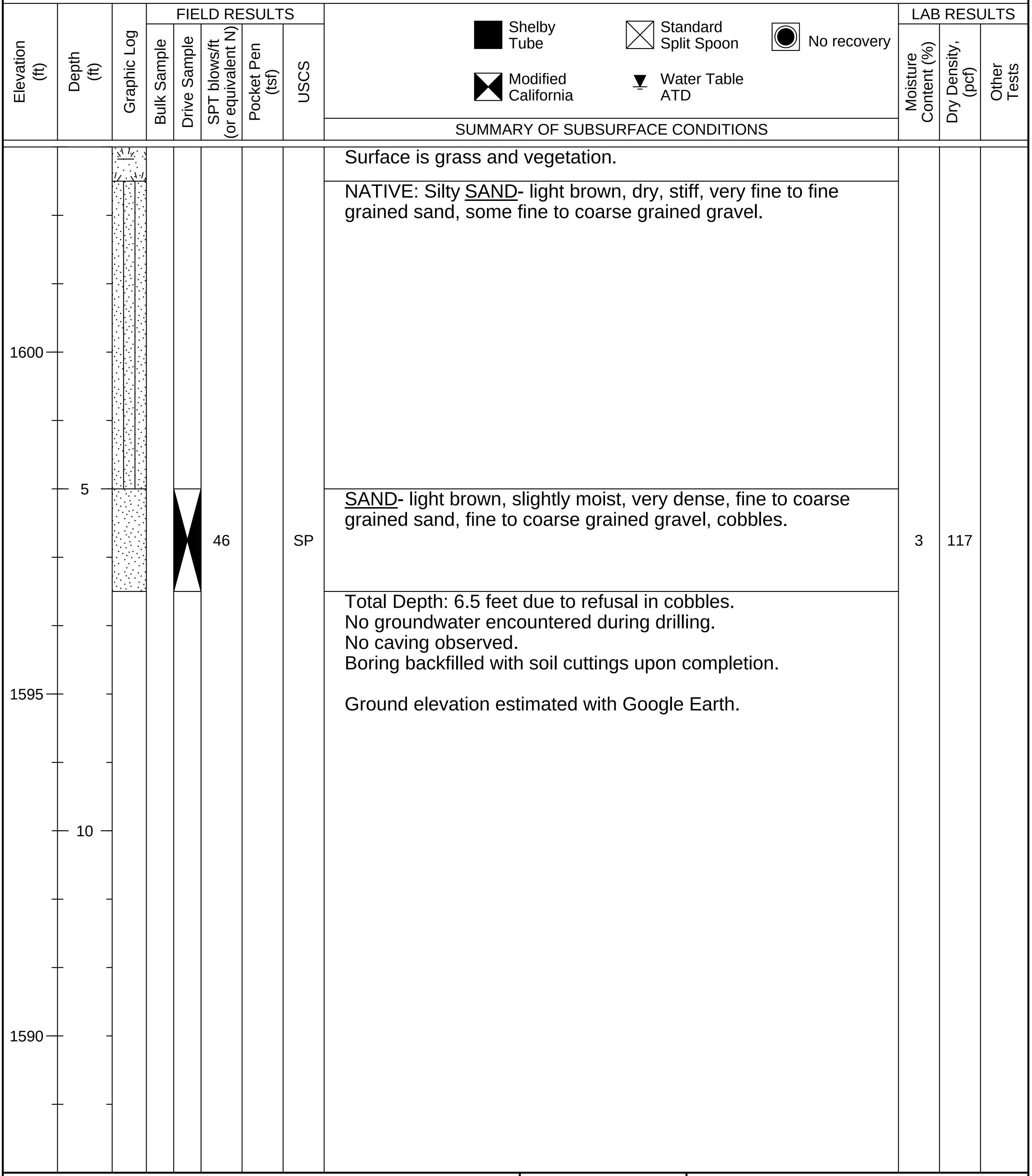
PLATE 4

TGR GEOTECHNICAL, INC.

LOG OF EXPLORATORY BORING B-4

Sheet **1** of **1**

Project Number: **22-7455**
Project Name: **Colton Ave. and Wabash Ave., Redlands**
Date Drilled: **3/15/22 - 3/15/22**
Ground Elev: **1603**
Logged By: **RA**
Project Engineer: **SG**
Drill Type: **Hollow Stem**
Drive Wt & Drop: **140lbs / 30in**



Depth (ft)	SPT blows/ft (or equivalent N)	USCS	SUMMARY OF SUBSURFACE CONDITIONS	Moisture Content (%)	Dry Density, (pcf)
0			Surface is grass and vegetation.		
			NATIVE: Silty SAND- light brown, dry, stiff, very fine to fine grained sand, some fine to coarse grained gravel.		
5	46	SP	SAND- light brown, slightly moist, very dense, fine to coarse grained sand, fine to coarse grained gravel, cobbles.	3	117

Total Depth: 6.5 feet due to refusal in cobbles.
No groundwater encountered during drilling.
No caving observed.
Boring backfilled with soil cuttings upon completion.

Ground elevation estimated with Google Earth.

This Boring Log should be evaluated in conjunction with the complete geotechnical report. This Boring Log represents conditions observed at the specific location and date indicated, it is not warranted to be representative of subsurface conditions at other locations and times.

PLATE 5

TGR GEOTECHNICAL, INC.

LOG OF EXPLORATORY BORING B-5/P-1

Sheet **1** of **1**

Project Number: **22-7455**
Project Name: **Colton Ave. and Wabash Ave., Redlands**
Date Drilled: **3/15/22 - 3/15/22**
Ground Elev: **1607**

Logged By: **RA**
Project Engineer: **SG**
Drill Type: **Hollow Stem**
Drive Wt & Drop: **140lbs / 30in**

■ Shelby Tube ☒ Standard Split Spoon ◉ No recovery
⧗ Modified California ▼ Water Table ATD

Elevation (ft)	Depth (ft)	Graphic Log	FIELD RESULTS				SUMMARY OF SUBSURFACE CONDITIONS	LAB RESULTS		
			Bulk Sample	Drive Sample	SPT blows/ft (or equivalent N)	Pocket Pen (tsf)	USCS		Moisture Content (%)	Dry Density, (pcf)	Other Tests
	0							Surface is grass and vegetation.			
1605								NATIVE: Silty <u>SAND</u>- light brown, dry, stiff, very fine to fine grained sand, some fine to coarse grained gravel.			
	5			⧗	50		SP	<u>SAND</u>- light brown, dry, very dense, fine grained sand, fine to coarse grained gravel, some silt.	2	113	-200= 5.7%
1600											
			☰				SP		2		-200= 10.2%
	10							Total Depth: 9 feet due to refusal in cobbles. No groundwater encountered during drilling. No caving observed. Boring utilized for percolation testing. Boring backfilled with soil cuttings upon completion. Ground elevation estimated with Google Earth.			
1595											

TGR GEOTECHNICAL, INC.

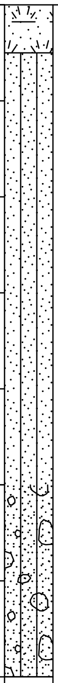
LOG OF EXPLORATORY BORING B-6

Sheet **1** of **1**

Project Number:	**22-7455**	Logged By:	**RA**
Project Name:	**Colton Ave. and Wabash Ave., Redlands**	Project Engineer:	**SG**
Date Drilled:	**3/15/22 - 3/15/22**	Drill Type:	**Hollow Stem**
Ground Elev:	**1611**	Drive Wt & Drop:	**140lbs / 30in**

■ Shelby Tube ☒ Standard Split Spoon ◉ No recovery

⧗ Modified California ▼ Water Table ATD



Elevation (ft)	Depth (ft)	FIELD RESULTS: Drive Sample	FIELD RESULTS: SPT blows/ft (or equivalent N)	FIELD RESULTS: Pocket Pen (tsf)	FIELD RESULTS: USCS	SUMMARY OF SUBSURFACE CONDITIONS	LAB RESULTS: Moisture Content (%)	LAB RESULTS: Dry Density, (pcf)	LAB RESULTS: Other Tests
						Surface is grass and vegetation.			
1610						NATIVE: Silty SAND- light brown, dry, stiff, very fine to fine grained sand, some fine to coarse grained gravel.			
						...Same as above, cobbles.			
	5	⧗				Gravelly SAND- grey brown, dry, very dense, fine to coarse grained sand, fine to coarse grained gravel, cobbles.			
1605			53		SPG		2	115	Consol
						Total Depth: 7 feet due to refusal in cobbles. No groundwater encountered during drilling. No caving observed. Boring backfilled with soil cuttings upon completion. Ground elevation estimated with Google Earth.			
	10								
1600									

This Boring Log should be evaluated in conjunction with the complete geotechnical report. This Boring Log represents conditions observed at the specific location and date indicated, it is not warranted to be representative of subsurface conditions at other locations and times.

PLATE 7

TGR GEOTECHNICAL, INC.

LOG OF EXPLORATORY BORING B-7

Sheet **1** of **1**

Project Number: **22-7455**
Project Name: **Colton Ave. and Wabash Ave., Redlands**
Date Drilled: **3/15/22 - 3/15/22**
Ground Elev: **1605**
Logged By: **RA**
Project Engineer: **SG**
Drill Type: **Hollow Stem**
Drive Wt & Drop: **140lbs / 30in**

■ Shelby Tube
☒ Standard Split Spoon
◉ No recovery
⧗ Modified California
▼ Water Table ATD

Elevation (ft)	Depth (ft)	Graphic Log	FIELD RESULTS					SUMMARY OF SUBSURFACE CONDITIONS	LAB RESULTS		
			Bulk Sample	Drive Sample	SPT blows/ft (or equivalent N)	Pocket Pen (tsf)	USCS		Moisture Content (%)	Dry Density, (pcf)	Other Tests
								Surface is grass and vegetation.			
								NATIVE: Silty <u>SAND</u>- light brown, dry, stiff, very fine to fine grained sand, some fine to coarse grained gravel.			
1600	5			⧗	59		SM	...Same as above, cobbles.	2	103	
								Total Depth: 7 feet due to refusal in cobbles. No groundwater encountered during drilling. No caving observed. Boring backfilled with soil cuttings upon completion. Ground elevation estimated with Google Earth.			
1595	10										

This Boring Log should be evaluated in conjunction with the complete geotechnical report. This Boring Log represents conditions observed at the specific location and date indicated, it is not warranted to be representative of subsurface conditions at other locations and times.

PLATE 8

TGR GEOTECHNICAL, INC.

LOG OF EXPLORATORY BORING B-8

Sheet **1** of **1**

Project Number:	**22-7455**	Logged By:	**RA**
Project Name:	**Colton Ave. and Wabash Ave., Redlands**	Project Engineer:	**SG**
Date Drilled:	**3/15/22 - 3/15/22**	Drill Type:	**Hollow Stem**
Ground Elev:	**1608**	Drive Wt & Drop:	**140lbs / 30in**

■ Shelby Tube
☒ Standard Split Spoon
◉ No recovery
⧗ Modified California
▼ Water Table ATD

Elevation (ft)	Depth (ft)	Graphic Log	FIELD RESULTS Bulk Sample	 Drive Sample	 SPT blows/ft (or equivalent N)	 Pocket Pen (tsf)	 USCS	SUMMARY OF SUBSURFACE CONDITIONS	LAB RESULTS Moisture Content (%)	 Dry Density, (pcf)	 Other Tests
	0		▬					Surface is grass and vegetation.			
			▬				SM	NATIVE: Silty SAND- light brown, dry, stiff, very fine to fine grained sand, some fine to coarse grained gravel.			Max, Shear
1605											
	5			⧗	14		SP	SAND- grey brown, dry, medium dense, fine to coarse grained sand, fine to coarse grained gravel, cobbles, some silt.	2	107	Consol
1600								Total Depth: 8 feet due to refusal in cobbles. No groundwater encountered during drilling. No caving observed. Boring backfilled with soil cuttings upon completion. Ground elevation estimated with Google Earth.			
	10										
1595											

This Boring Log should be evaluated in conjunction with the complete geotechnical report. This Boring Log represents conditions observed at the specific location and date indicated, it is not warranted to be representative of subsurface conditions at other locations and times.

PLATE 9

TGR GEOTECHNICAL, INC.

LOG OF EXPLORATORY BORING B-9

Sheet **1** of **1**

Project Number: **22-7455**
Project Name: **Colton Ave. and Wabash Ave., Redlands**
Date Drilled: **3/15/22 - 3/15/22**
Ground Elev: **1611**

Logged By: **RA**
Project Engineer: **SG**
Drill Type: **Hollow Stem**
Drive Wt & Drop: **140lbs / 30in**

■ Shelby Tube
☒ Standard Split Spoon
◉ No recovery
⧗ Modified California
▼ Water Table ATD

			FIELD RESULTS						LAB RESULTS		
Elevation (ft)	Depth (ft)	Graphic Log	Bulk Sample	Drive Sample	SPT blows/ft (or equivalent N)	Pocket Pen (tsf)	USCS	SUMMARY OF SUBSURFACE CONDITIONS	Moisture Content (%)	Dry Density, (pcf)	Other Tests
								Surface is grass and vegetation.			
1610								NATIVE: Silty SAND- light brown, dry, stiff, very fine to fine grained sand, some fine to coarse grained gravel.			
								Total Depth: 3 feet due to refusal in cobbles. No groundwater encountered during drilling. No caving observed. Boring backfilled with soil cuttings upon completion. Ground elevation estimated with Google Earth.			
	5										
1605											
	10										
1600											

This Boring Log should be evaluated in conjunction with the complete geotechnical report. This Boring Log represents conditions observed at the specific location and date indicated, it is not warranted to be representative of subsurface conditions at other locations and times.

PLATE 10

TGR GEOTECHNICAL, INC.

LOG OF EXPLORATORY BORING P-2

Sheet **1** of **1**

Project Number: **22-7455**
Project Name: **Colton Ave. and Wabash Ave., Redlands**
Date Drilled: **3/15/22 - 3/15/22**
Ground Elev: **1606**

Logged By: **RA**
Project Engineer: **SG**
Drill Type: **Hollow Stem**
Drive Wt & Drop: **140lbs / 30in**

■ Shelby Tube
☒ Standard Split Spoon
◉ No recovery
⧓ Modified California
▼ Water Table ATD

Elevation (ft)	Depth (ft)	Graphic Log	FIELD RESULTS					SUMMARY OF SUBSURFACE CONDITIONS	LAB RESULTS		
			Bulk Sample	Drive Sample	SPT blows/ft (or equivalent N)	Pocket Pen (tsf)	USCS		Moisture Content (%)	Dry Density, (pcf)	Other Tests
								Surface is grass and vegetation.			
1605								NATIVE: Silty <u>SAND</u>- light brown, dry, stiff, very fine to fine grained sand, some fine to coarse grained gravel.			
	5			⧓	16		SP	<u>SAND</u>- light brown, dry, medium dense, fine grained sand, fine to coarse grained gravel, cobbles.	2	117	-200= 10.3%
1600											
								Total Depth: 9 feet due to refusal in cobbles. No groundwater encountered during drilling. No caving observed. Boring utilized for percolation testing. Boring backfilled with soil cuttings upon completion. Ground elevation estimated with Google Earth.			
	10										
1595											

This Boring Log should be evaluated in conjunction with the complete geotechnical report. This Boring Log represents conditions observed at the specific location and date indicated, it is not warranted to be representative of subsurface conditions at other locations and times.

PLATE 11

TGR GEOTECHNICAL, INC.

APPENDIX C
LABORATORY TEST RESULTS

APPENDIX C

Laboratory Testing Procedures and Results

In-Situ Moisture and Dry Density Determination (ASTM D2216 and D7263): Moisture content and dry density determinations were performed on relatively undisturbed samples obtained from the test borings. The results of these tests are presented in the boring logs. Where applicable, only moisture content was determined from "undisturbed" or disturbed samples.

Maximum Density and Optimum Moisture Content (ASTM D1557): The maximum dry density and optimum moisture content of typical materials were determined in accordance with ASTM Test Method D1557. The results of these tests are presented in the table below:

Sample Location	Sample Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
B-8 @ 0-5 feet	Silty Sand	123.5	7.0

Direct Shear Strength (ASTM D3080): Direct shear test was performed on selected remolded samples, which were soaked for a minimum of 24 hours under a surcharge equal to the applied normal force during testing. After transfer of the sample to the shear box, and reloading the sample, pore pressures set up in the sample due to the transfer were allowed to dissipate for a period of approximately 1-hour prior to application of shearing force. The sample was tested under various normal loads, a motor-driven, strain-controlled, direct-shear testing apparatus at a strain rate of less than 0.001 to 0.5 inches per minute (depending upon the soil type). The test results are presented in the test data and in the table below:

Sample Location	Sample Description	Friction Angle (degrees)	Apparent Cohesion (psf)
B-8 @ 0-5 feet	Silty Sand (Remolded)	33	114

Consolidation Tests (ASTM D2435): Consolidation test were performed on selected, relatively undisturbed ring samples. Samples were placed in a consolidometer and loads were applied in geometric progression. The percent consolidation for each load cycle was recorded as the ratio of the amount of vertical compression to the original 1-inch height. The consolidation pressure curves are presented in the test data.

Expansion Potential (ASTM D4829): The expansion potential of selected materials was evaluated by the Expansion Index Test, ASTM D4829. Specimens are molded under a given compactive energy to approximately the optimum moisture content and approximately 50 percent saturation or approximately 90 percent relative compaction. The prepared 1-inch thick by 4-inch diameter specimens are loaded to an equivalent 144 psf surcharge and are inundated with tap water until volumetric equilibrium is reached. The results of these tests are presented in the table below:

Sample Location	Sample Description	Expansion Index	Expansion Potential
B-2 @ 0-5 feet	Silty Sand	0	Very Low

Soluble Sulfate (CAL 417A): The soluble sulfate content of selected sample was determined by standard geochemical methods. The test results are presented in the test data and in the table below:

Sample Location	Sample Description	Water Soluble Sulfate in Soil, (% by Weight)	Sulfate Content (ppm)	Exposure Class*
B-2 @ 0-5 feet	Silty Sand	0.0123	123	S0

* Based on the current version of ACI 318-14 Building Code, Table No. 19.3.1.1; Exposure Categories and Classes.

Corrosivity Tests (CAL 422, CAL 643 and CAL 747): Electrical conductivity, pH, and soluble chloride tests were conducted on representative samples and the results are provided in the test data and in the table below:

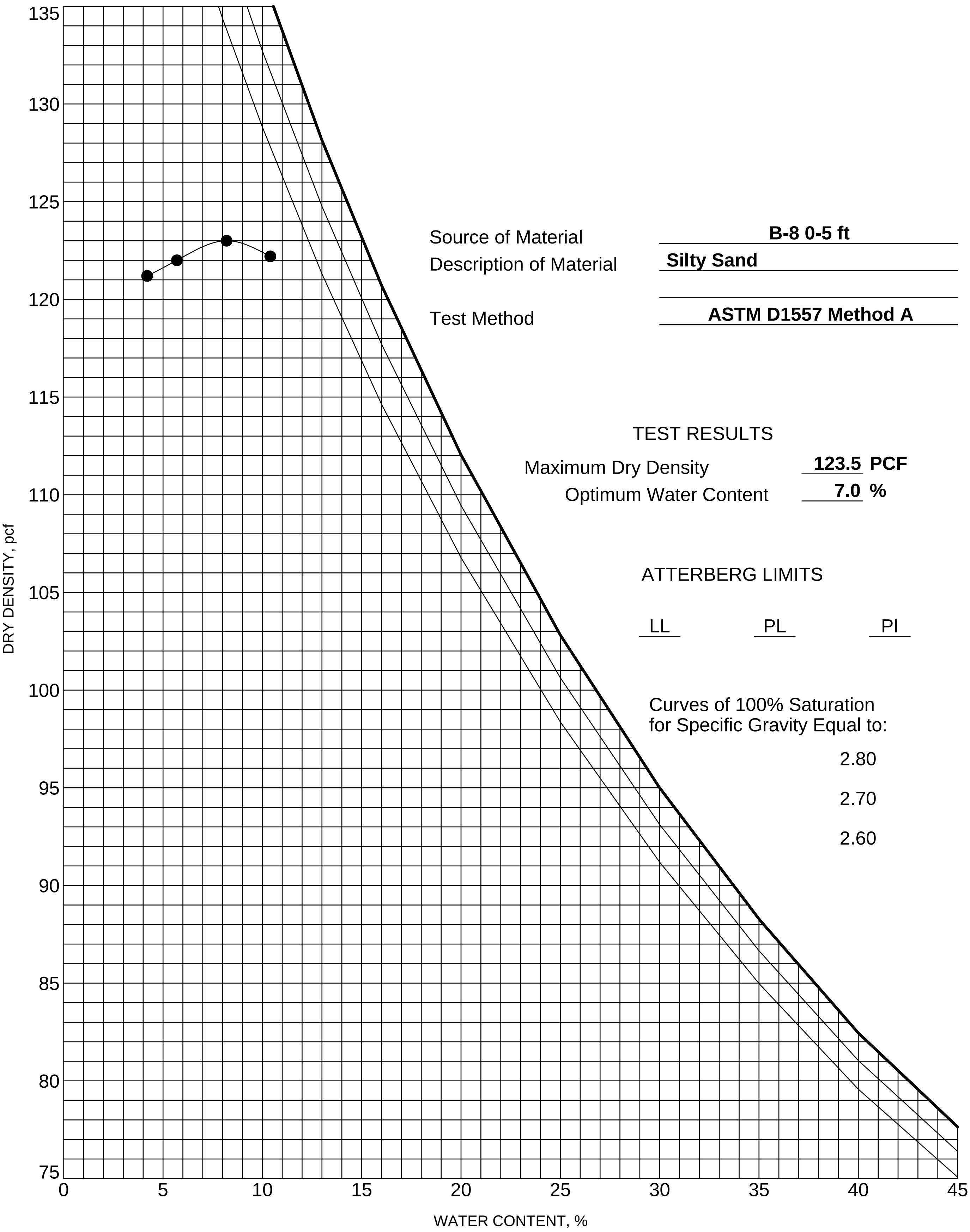
Sample Location	Sample Description	Soluble Chloride (CAL 422) (ppm)	Electrical Resistivity (CAL 643) (ohm-cm)	pH (CAL 747)	Potential Degree of Attack on Steel
B-2 @ 0-5 feet	Silty Sand	65	11,000	7.8	Mild

Passing No. 200 Sieve (ASTM D1140): Typical materials were washed over No. 200 sieve. The test results are presented in the boring logs and in the table below:

Sample Location	% Passing No. 200 Sieve
B-5/P-1 @ 5 feet	5.7
B-5/P-1 @ 8.5-9 feet	10.2
P-2 @ 5 feet	10.3

R-Value: The resistance “R”-Value was determined by the California Materials Method No. 301 for subgrade soils. One sample was prepared, and exudation pressure and “R”-Value determined. The graphically determined “R”-Value at exudation pressure of 300 psi is summarized in the table below:

Sample Location	Sample Description	R-Value
B-2 @ 0-5 feet	Silty Sand	73



Source of Material B-8 0-5 ft

Description of Material Silty Sand

Test Method ASTM D1557 Method A

TEST RESULTS

Maximum Dry Density 123.5 PCF

Optimum Water Content 7.0 %

ATTERBERG LIMITS

LL	PL	PI

Curves of 100% Saturation
for Specific Gravity Equal to:

2.80

2.70

2.60

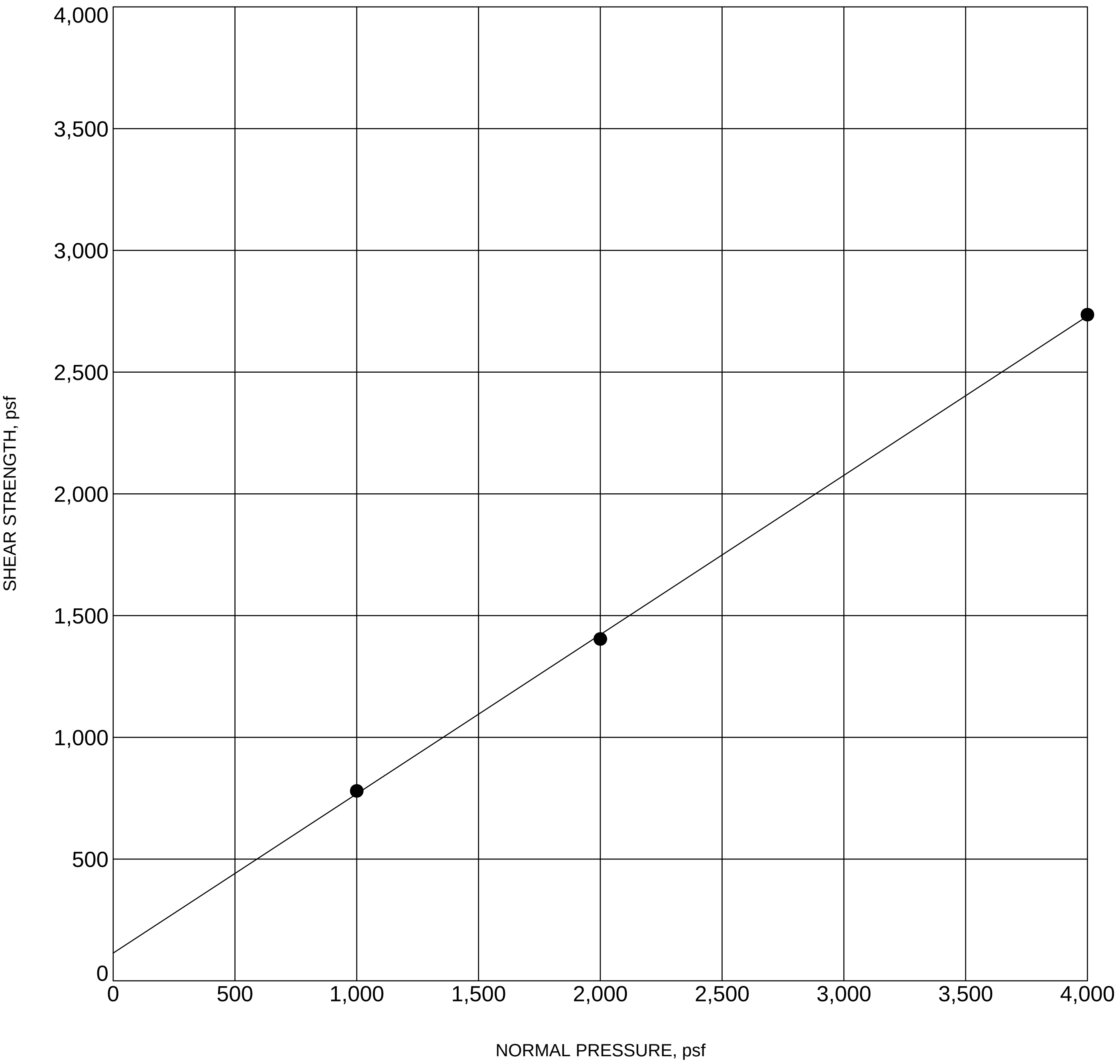
MOISTURE-DENSITY RELATIONSHIP

TGR GEOTECHNICAL, INC.

3037 S. Harbor Blvd
Santa Ana, CA 92704
Telephone: 714-641-7189
Fax: 714-641-7190

Project Number: 22-7455

Project Name: Colton Ave. and Wabash Ave., Redlands



	Specimen Identification		Classification	γ_d	MC%	c	ϕ
●	**B-8**	**0-5**	**Silty Sand - Remolded - 90% RC**	**111**	**7**	**114**	**33**

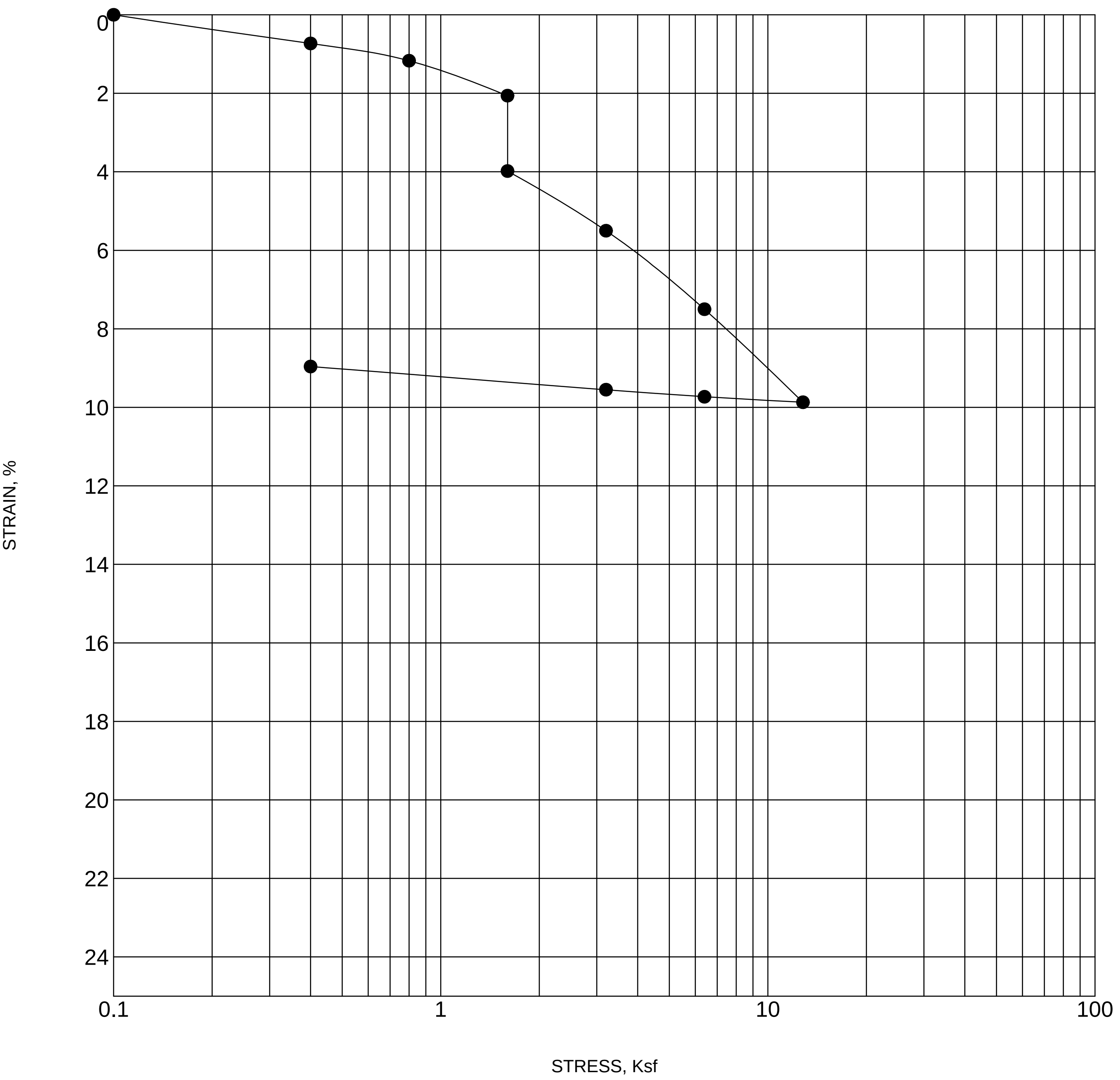
TGR GEOTECHNICAL, INC.

3037 S. Harbor Blvd
Santa Ana, CA 92704
Telephone: 714-641-7189
Fax: 714-641-7190

DIRECT SHEAR TEST

Project Number: 22-7455

Project Name: Colton Ave. and Wabash Ave., Redlands



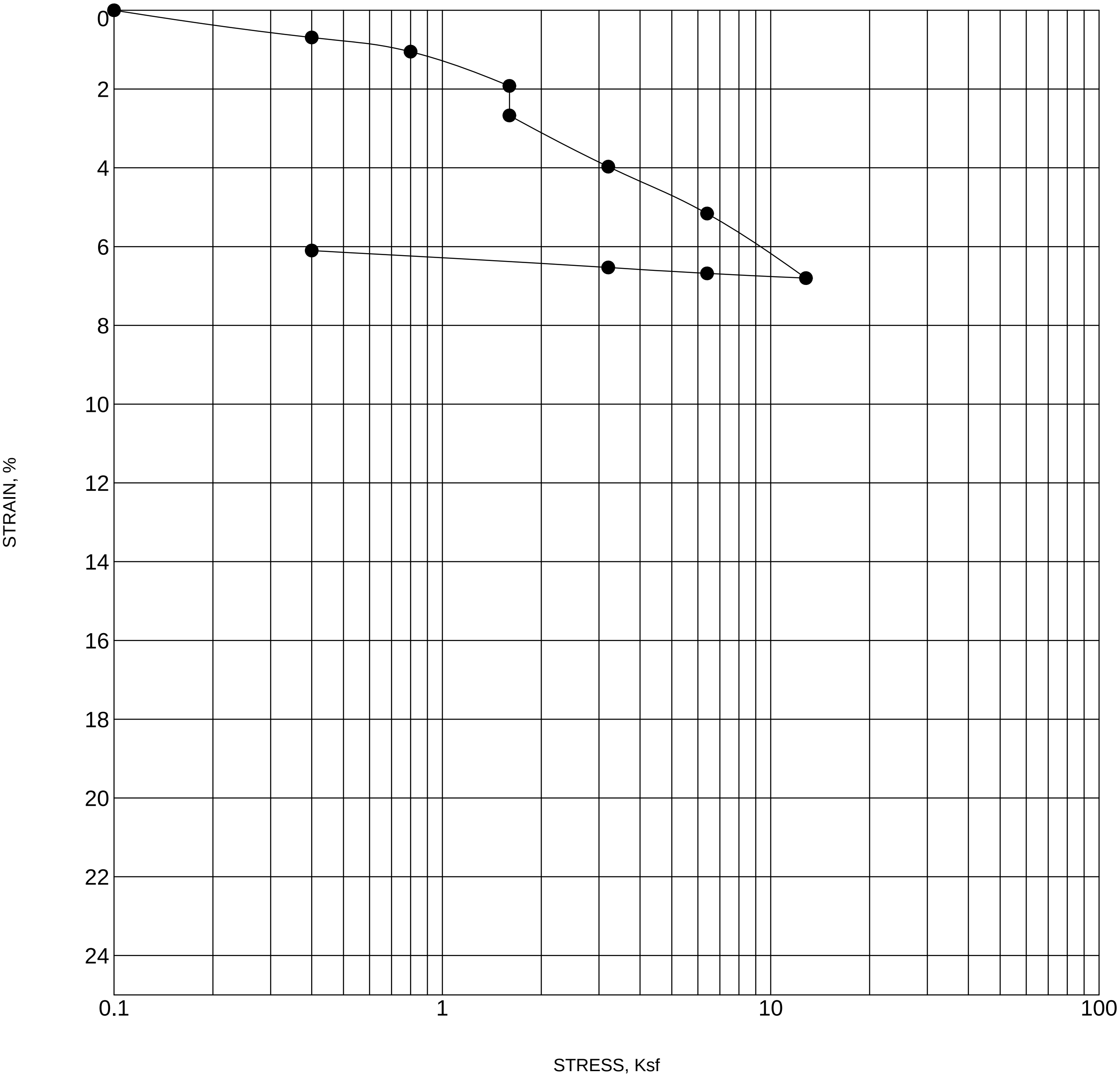
Specimen Identification			Classification	γ_d	MC%
●	**B-1**	**5.0**	**Sand**	**117**	**1**

TGR GEOTECHNICAL, INC.
3037 S. Harbor Blvd
Santa Ana, CA 92704
Telephone: 714-641-7189
Fax: 714-641-7190

CONSOLIDATION TEST

Project Number: 22-7455

Project Name: Colton Ave. and Wabash Ave., Redlands



	Specimen Identification		Classification	γ_d	MC%
●	**B-6**	**5.0**	**Gravelly Sand**	**115**	**2**

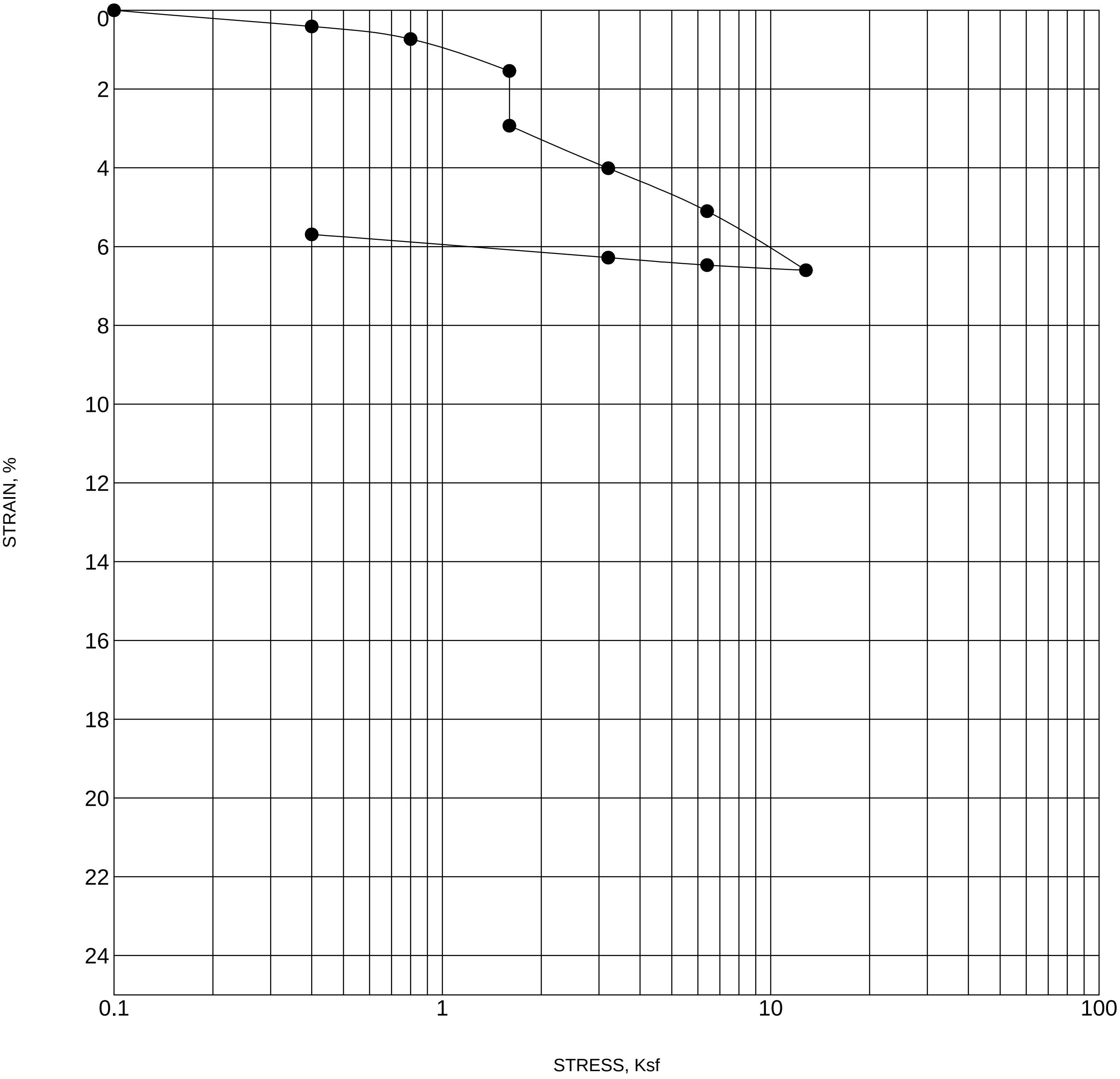
TGR GEOTECHNICAL, INC.

3037 S. Harbor Blvd
Santa Ana, CA 92704
Telephone: 714-641-7189
Fax: 714-641-7190

CONSOLIDATION TEST

Project Number: 22-7455

Project Name: Colton Ave. and Wabash Ave., Redlands



	Specimen Identification		Classification	γ_d	MC%
●	**B-8**	**5.0**		**107**	**2**

TGR GEOTECHNICAL, INC.

3037 S. Harbor Blvd
Santa Ana, CA 92704
Telephone: 714-641-7189
Fax: 714-641-7190

CONSOLIDATION TEST

Project Number: 22-7455

Project Name: Colton Ave. and Wabash Ave., Redlands

ANAHEIM TEST LAB, INC
196 Technology Dr., Unit D
Irvine, CA 92618
Phone (949) 336-6544

TO:

TGR GEOTECHNICAL
3037 S. HARBOR BLVD.
SANTA ANA, CA 92704

DATE: 3/31/2022

P.O. NO: VERBAL

LAB NO: C-5800

SPECIFICATION: CTM-643/417/422

MATERIAL: Soil

Project No.: 22-7455
Project: Colton - Redlands
Sample ID: B2 @ 0-5'

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
7.8	11,000	123	65

RESPECTFULLY SUBMITTED

WES BRIDGER LAB MANAGER

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

TO:

TGR GEOTECHNICAL
3037 S. HARBOR BLVD.
SANTA ANA, CA. 92704

DATE: 3/24/2022

P.O. NO.: VERBAL

LAB NO.: C-5801

SPECIFICATION: CTM- 301

MATERIAL: Brown, Silty Sand w. trace F. Gravel

Project No.: 22-7455
Project: Colton - Redlands
Sample ID: B2 @ 0-5'

ANALYTICAL REPORT

"R" VALUE

BY EXUDATION	BY EXPANSION
73	N/A

RESPECTFULLY SUBMITTED

WES BRIDGER LAB MANAGER

"R" VALUE CA 301

Client: TGR Geotechnical **ATL No.:** C 5801 **Date:** 3/24/2022

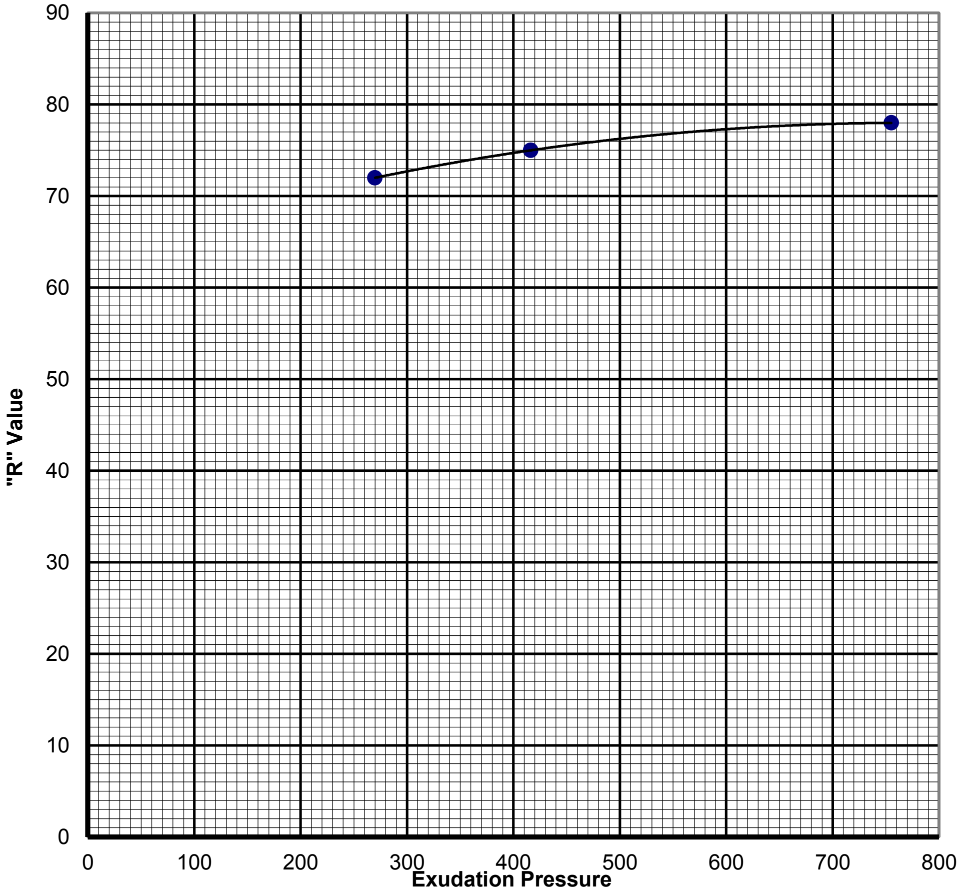
Client Reference No.: 22-7455

Sample: B2 @ 0-5' **Soil Type:** Brown, Silty Sand w. trace F. Gravel

.

TEST SPECIMEN		A	B	C	D
Compactor Air Pressure	psi	350	350	350	
Initial Moisture Content	%	1.7	1.7	1.7	
Moisture at Compaction	%	9.3	8.7	9.1	
Briquette Height	in.	2.52	2.44	2.48	
Dry Density	pcf	124.0	125.5	124.8	
EXUDATION PRESSURE	psi	270	755	416	
EXPANSION PRESSURE	psf	26	139	87	
Ph at 1000 pounds	psi	18	15	17	
Ph at 2000 pounds	psi	33	28	30	
Displacement	turns	3.74	3.33	3.62	
"R" Value		72	78	75	
CORRECTED "R" VALUE		72	78	75	

Final "R" Value	
BY EXUDATION: @ 300 psi	**73**
BY EXPANSION: TI = 5.0	**N/A**



APPENDIX D
SITE SEISMICITY AND DEAGGREGATED PARAMETERS

TABLE 1

SITE SPECIFIC GROUND MOTION ANALYSIS

22-7455 E. Colton Avenue and N. Wabash Avenue, Redlands

SA Period (sec)	Probabilistic Spectral Acceleration MCER (g)	Deterministic Spectral Acceleration (g)	Is Largest Deterministic Spectral Acceleration <1.5*Fa	Deterministic MCER	Site Specific MCER	2/3 of Site Specific MCER	80% Code Design	Site Specific Design Response Spectrum
	Rotated Maximum	Rotated Maximum 84th Percentile						
0	1.1374	0.7488	No	0.7488	0.7488	0.4992	0.4083	0.4992
0.1	1.8975	1.1212		1.1212	1.1212	0.7475	0.7055	0.7475
0.2	2.4849	1.5300		1.5300	1.5300	1.0200	1.0208	1.0208
0.3	2.8575	1.8442		1.8442	1.8442	1.2295	1.0208	1.2295
0.5	2.9634	2.0185		2.0185	2.0185	1.3457	1.0208	1.3457
0.75	2.6198	1.8138		1.8138	1.8138	1.2092	1.0208	1.2092
1	2.3673	1.6567		1.6567	1.6567	1.1045	1.0520	1.1045
2	1.4688	1.0058		1.0058	1.0058	0.6705	0.5260	0.6705
3	1.0626	0.7045		0.7045	0.7045	0.4697	0.3507	0.4697
4	0.8120	0.5124		0.5124	0.5124	0.3416	0.2630	0.3416
5	0.6435	0.3854		0.3854	0.3854	0.2569	0.2104	0.2569

Code Sds	1.276	Crs =	0.914	Code Ss =	1.914	**Site Specific SDS =**	**1.211**
Code Sd1	1.315	Cr1 =	0.891	Code S1 =	0.789	**Site Specific SD1 =**	**1.409**
To	0.21	Code Fa =	1	Sms =	1.914		
Ts	1.03	Code Fv =	2.5	Sm1 =	1.9725		
TL	8						
Input							

FIGURE 1

Site Specific Design Response Spectra

22-7455 E. Colton Avenue and N. Wabash Avenue, Redlands

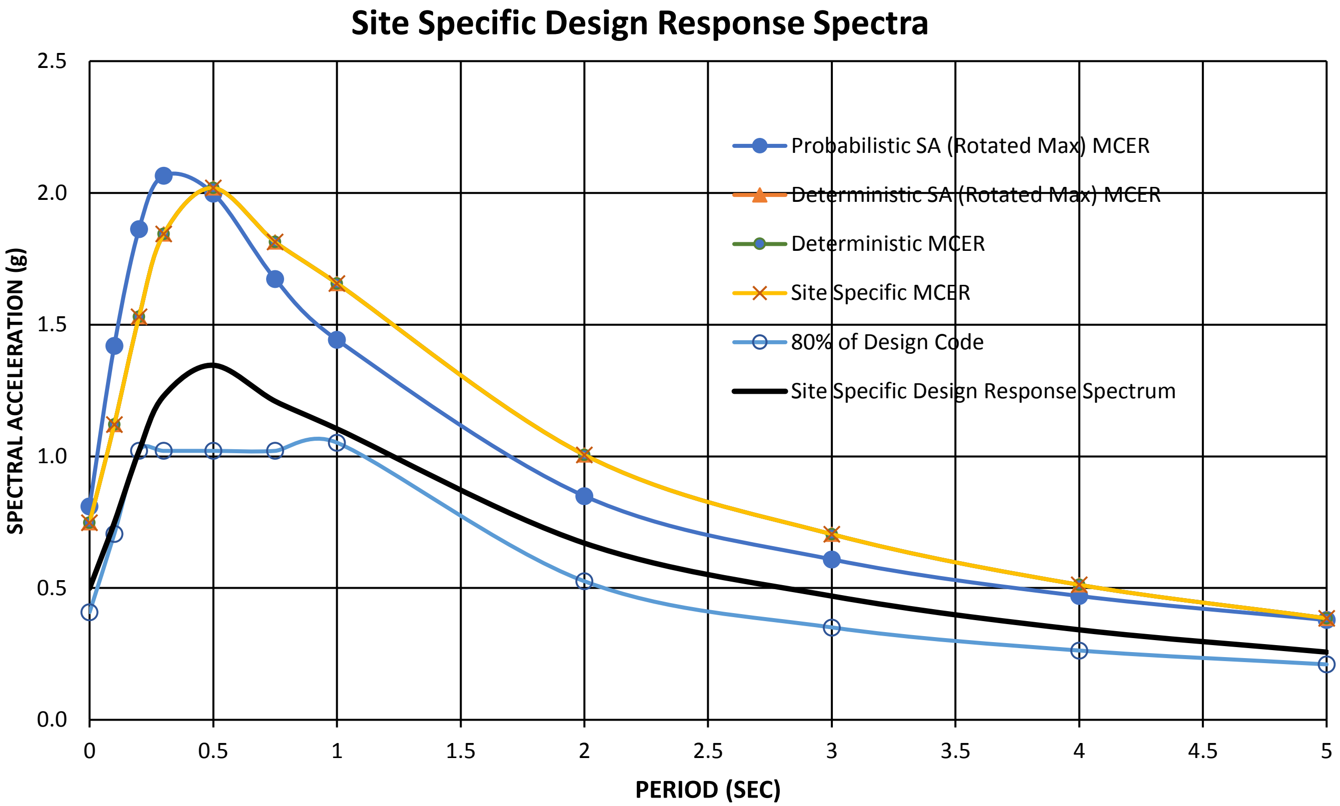


TABLE 2

Probabilistic Response Spectrum ASCE 7-16 Method 2

22-7455 E. Colton Avenue and N. Wabash Avenue, Redlands

Period (g)	UHGM (g)	RTGM (g)	Max Dir Scale factor	Max Dir RTGM (g)
0	1.059	1.034	1.1	1.137
0.1	1.740	1.725	1.1	1.898
0.2	2.276	2.259	1.1	2.485
0.3	2.611	2.540	1.125	2.858
0.5	2.694	2.522	1.175	2.963
0.75	2.326	2.117	1.2375	2.620
1	2.026	1.821	1.3	2.367
2	1.226	1.088	1.35	1.469
3	0.863	0.759	1.4	1.063
4	0.640	0.560	1.45	0.812
5	0.489	0.429	1.5	0.644

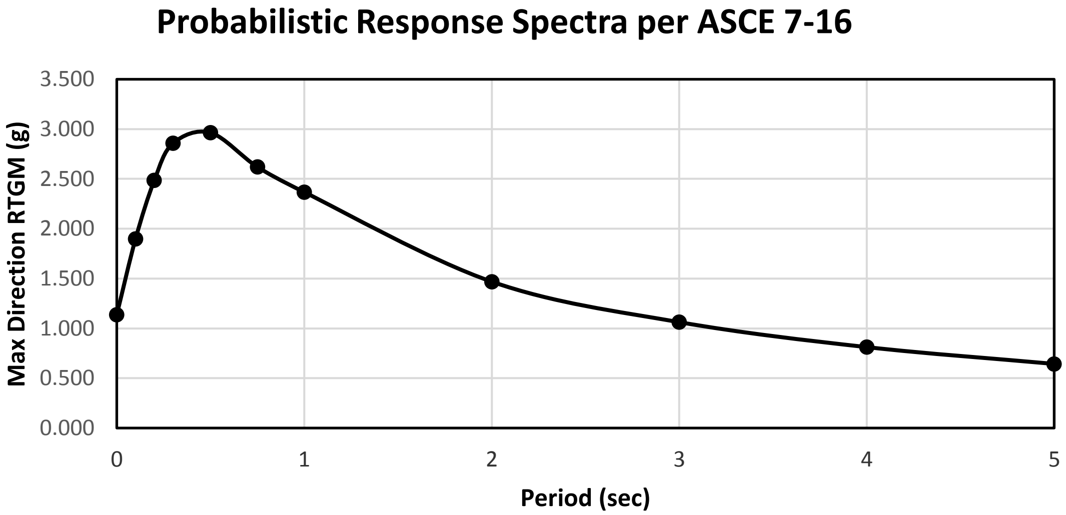
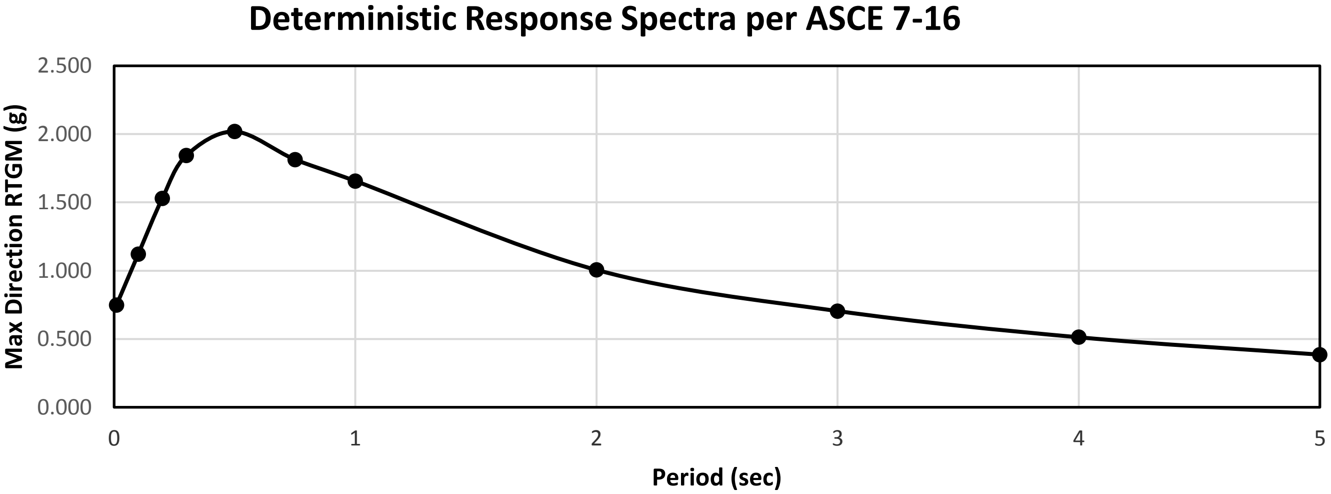


TABLE 3

Deterministic Response Spectrum ASCE 7-16 - San Andreas (San Bernardino S)

22-7455 E. Colton Avenue and N. Wabash Avenue, Redlands

Period (g)	84th-Percentile Spectral Acceleration (g)	Max Dir Scale factor	Max Dir Deterministic SA (g)
0.01	0.681	1.1	0.749
0.1	1.019	1.1	1.121
0.2	1.391	1.1	1.530
0.3	1.639	1.125	1.844
0.5	1.718	1.175	2.019
0.75	1.466	1.2375	1.814
1	1.274	1.3	1.657
2	0.745	1.35	1.006
3	0.503	1.4	0.704
4	0.353	1.45	0.512
5	0.257	1.5	0.385



PACIFIC EARTHQUAKE ENGINEERING RESEARCH CENTER

WEIGHTED AVERAGE of 2014 NGA WEST-2 GMPEs

Last updated: 04 14 15

by Emel Seyhan, PhD, PEER & UCLA -- email: emel.seyhan@gmail.com, peer_center@berkeley.edu

San Andreas (San Bernardino S) fault

This excel file will be updated as necessary on the PEER website to fix any typos or other errors. Please check the website frequently for new versions at: http://peer.berkeley.edu/ngawest2/databases/

Legend	Pre-defined option	Main input variable	Calculated variable	Input var. flag	Internal variable

GMPE averaging	Geometric	Weighted average of the natural logarithm of the spectral values

GMPEs	ASK14	BSSA14	CB14	CY14	I14
Weight	0.25	0.25	0.25	0.25	0

# of std. dev.	1	
Damping ratio (%)	5	Modification factors are calculated in Sheet DSF

ASK14 Abrahamson & Silva & Kamai 2014 NGA West-2 Model
BSSA14 Boore & Stewart & Seyhan & Atkinson 2014 NGA West-2 Model
CB14 Campbell & Bozorgnia 2014 NGA West-2 Model
CY14 Chiou & Youngs 2014 NGA West-2 Model
I14 Idriss 2014 NGA West-2 Model

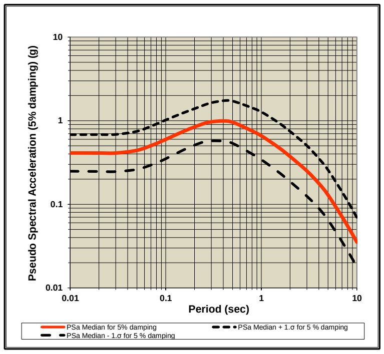
Input variables		Errors and warnings
M_w	7.47	
R_{RUP} (km)	5.81	
R_{JB} (km)	5.81	
R_X (km)	5.81	
Ry0 (km)	999	If unknown use 999
V_{S30} (m/sec)	260	
U (BSSA13)	1	1: Unspecified fault mech.
F_{RV}	0	1: reverse fault
F_{NM}	0	1: normal fault
F_{HW}	0	1: hanging wall side
Dip (deg)	90	
Z_{TOR} (km)	999	If unknown use 999
Z_{HYP} (km)	7.48	If unknown use 999
$Z_{1.0}$ (km)	999	If unknown use 999
$Z_{2.5}$ (km)	999	If unknown use 999
W (km)	11.52	If unknown use 999
Vs30Flag	measured	Choose options for V_{s30} from the list
F_{AS}	no	Aftershock effect is not applicable.
Region	California	Choose region from the list

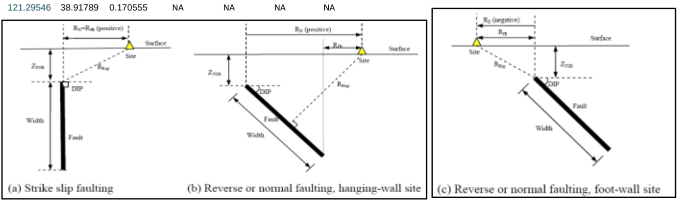
Calculated Variables/Flags

ΔDPP	0	Always 0 for median calcs.
PGA_r (g)	0.347	
Z_{BOT} (km) (CB14)	15	Enter for default W calcs
SS	1	auto calculated
$V_{s30Flag}$	1	measured
F_{AS}	0	Aftershock effect is not applicable.
Region	0	California
Option for Sa value	1	Weighted average of the natural logarithm of the spectral values

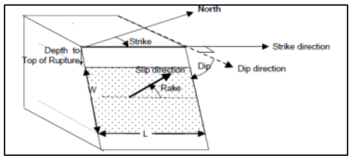
RotD50 Horizontal Component of PGA, PGV and *IMs*

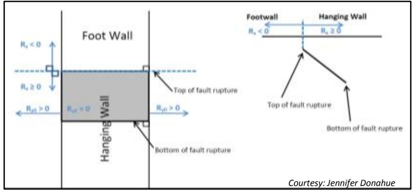
GMP	T (s)	Baseline: 5% Damping				User defined: 5% Damping			
		PSa Median for 5% damping	PSa Median + 1.σ for 5% damping	PSa Median - 1.σ for 5% damping	S_d Median for 5% damping	PSa Median for 5% damping	PSa Median + 1.σ for 5 % damping	PSa Median - 1.σ for 5 % damping	Sd Median for 5 % damping
PSa (g), S_d (cm)	0.01	0.4113299	0.6807387	0.248542	0.001021	0.41133	0.6807387	0.248542	0.001021
	0.02	0.4106372	0.6825476	0.247049	0.004077	0.410637	0.6825476	0.247049	0.004077
	0.03	0.4109585	0.6858812	0.246233	0.009181	0.410548	0.6851953	0.245987	0.009172
	0.05	0.4420571	0.7465188	0.261768	0.027434	0.442057	0.7465188	0.261768	0.027434
	0.075	0.5175297	0.8840625	0.302962	0.072264	0.518565	0.8858306	0.303567	0.072409
	0.1	0.5962087	1.016328	0.349754	0.148001	0.597997	1.019377	0.350803	0.148445
	0.15	0.735682	1.2287286	0.440478	0.410903	0.737153	1.2311861	0.441359	0.411725
	0.2	0.8403639	1.3881876	0.508729	0.834437	0.842045	1.390964	0.509747	0.836106
	0.25	0.9204193	1.5271272	0.554749	1.428013	0.924101	1.5332357	0.556968	1.433725
	0.3	0.9690804	1.6377524	0.573418	2.165054	0.970049	1.6393901	0.573991	2.167219
	0.4	0.9912308	1.7296257	0.568064	3.936961	0.993213	1.733085	0.5692	3.944835
	0.5	0.9592081	1.7162152	0.53611	5.952771	0.960167	1.7179314	0.536646	5.958724
	0.75	0.7823989	1.4657097	0.417646	10.92489	0.782399	1.4657097	0.417646	10.92489
	1	0.6608729	1.2757199	0.342358	16.4053	0.660212	1.2744442	0.342016	16.3889
	1.5	0.4828805	0.9530981	0.244648	26.97047	0.483363	0.9540512	0.244893	26.99744
	2	0.3740868	0.7465504	0.18745	37.14486	0.373339	0.7450573	0.187075	37.07057
	3	0.2510145	0.5037736	0.125073	56.07997	0.250764	0.5032698	0.124948	56.02389
	4	0.1779982	0.3537962	0.089553	70.69715	0.17782	0.3534424	0.089463	70.62645
	5	0.1293917	0.2577257	0.064961	80.29949	0.129004	0.2569525	0.064766	80.0586
	7.5	0.062502	0.1241545	0.031465	87.27359	0.062377	0.1239062	0.031402	87.09905
	10	0.0354445	0.0697478	0.018012	87.98629	0.035303	0.0694688	0.01794	87.63435
PGA (g)	0	0.4089755	0.6763637	0.247294	0.001015	0.41133	0.6807387	0.248542	0.001021
PGV (cm/s)	-1	68.706356	121.29546	38.91789	0.170555	NA	NA	NA	NA





(a) Strike slip faulting (b) Reverse or normal faulting, hanging-wall site (c) Reverse or normal faulting, foot-wall site





Courtesy: Jennifer Donahue

Definition of Parameters

Damping ratio = Viscous damping ratio (%) See Sanaz et al. (2012) PEER Report
PSA = Pseudo-absolute acceleration response spectrum (g)
PGA = Peak ground acceleration (g)
PGV = Peak ground velocity (cm/s)
S_d = Relative displacement response spectrum (cm)
M_w = Moment magnitude
R_{RUP} = Closest distance to coseismic rupture (km), used in ASK13, CB13 and CY13. See Figures a, b and c for illustation
R_{JB} = Closest distance to surface projection of coseismic rupture (km). See Figures a, b and c for illustation
R_X = Horizontal distance from top of rupture measured perpendicular to fault strike (km). See Figures a, b and c for illustation
R_{y0} = The horizontal distance off the end of the rupture measured parallel to strike (km)
V_{S30} = The average shear-wave velocity (m/s) over a subsurface depth of 30 m
U = Unspecified-mechanism factor: 1 for unspecified; 0 otherwise
F_{RV} = Reverse-faulting factor: 0 for strike slip, normal, normal-oblique; 1 for reverse, reverse-oblique and thrust
F_{NM} = Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique, thrust and normal-oblique; 1 for normal
F_{HW} = Hanging-wall factor: 1 for site on down-dip side of top of rupture; 0 otherwise
Dip = Average dip of rupture plane (degrees)
Z_{TOR} = Depth to top of coseismic rupture (km)
Z_{HYP} = Hypocentral depth from the earthquake
$Z_{1.0}$ = Depth to Vs=1 km/sec
$Z_{2.5}$ = Depth to Vs=2.5 km/sec
W = Fault rupture width (km)
$V_{s30flag}$ = 1 for measured, 0 for inferred Vs30
F_{AS} = 0 for mainshock; 1 for aftershock
Region = Specific regions considered in the models, Click on Region to see codes
ΔDPP = Directivity term, direct point parameter; uses 0 for median predictions
PGA_r (g) = Peak ground acceleration on rock (g), this specific cell is updated in the cell for BSSA14 and CB14, for others it is taken account for in the macros
Z_{BOT} (km) = The depth to the bottom of the seismogenic crust
Z_{BOR} (km) = The depth to the bottom of the rupture plane
SS = 1 for strike slip, automatically updated in the cell

Input variables with defaults (If entered 999 as input):

Red colored value: The value is used in the code when input is unknown

DEFAULTs	USER defined	ASK14	BSSA14	CB14	CY14	I14
W (km)	11.52	▨	▨	15.000	▨	▨
$Z_{1.0}$ (km)	999.000	0.475	▨	▨	0.485	▨
$\delta Z_{1.0}$ (km)	0.000	▨	0.000	▨	▨	▨
$Z_{2.5}$ (V_{S30}=1100)(km)	999.000	▨	▨	0.398	▨	▨
$Z_{2.5}$ (V_{S30})(km)	999.000	▨	▨	2.070	▨	▨
Z_{hyp} (km)	7.48	▨	▨	10.227	▨	▨
Z_{tor} (km)	999.00	▨	▨	0.000	0.000	▨
Z_{BOR} (km)	-	▨	▨	15.000	▨	▨

ACKNOWLEDGEMENTS

CEA CALIFORNIA EARTHQUAKE AUTHORITY
Caltrans
PG&E

Nick Gregor, Bechtel
Silvia Mazzoni, Consultant

All NGA West-2 participants are acknowledged for their constructive comments and feedback.

PACIFIC EARTHQUAKE ENGINEERING RESEARCH CENTER

WEIGHTED AVERAGE of 2014 NGA WEST-2 GMPEs
Last updated: 04 14 15

by Emel Seyhan, PhD, PEER & UCLA -- email: emel.seyhan@gmail.com, peer_center@berkeley.edu

San Jacinto (San Jacinto Valley) fault

This excel file will be updated as necessary on the PEER website to fix any typos or other errors. Please check the website frequently for new versions at: http://peer.berkeley.edu/ngawest2/databases/

Legend	Pre-defined option	Main input variable	Calculated variable	Input var. flag	Internal variable

GMPE averaging	Geometric

Weighted average of the natural logarithm of the spectral values

GMPEs	**ASK14**	**BSSA14**	**CB14**	**CY14**	**I14**
Weight	0.25	0.25	0.25	0.25	0

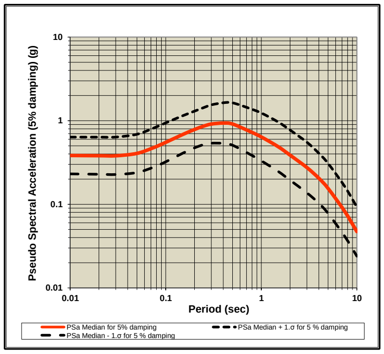
# of std. dev.	1
Damping ratio (%)	5

Modification factors are calculated in Sheet DSF

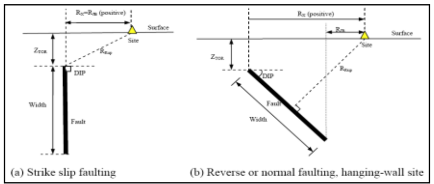
ASK14	Abrahamson & Silva & Kamai 2014 NGA West-2 Model
BSSA14	Boore & Stewart & Seyhan & Atkinson 2014 NGA West-2 Model
CB14	Campbell & Bozorgnia 2014 NGA West-2 Model
CY14	Chiou & Youngs 2014 NGA West-2 Model
I14	Idriss 2014 NGA West-2 Model

RotD50 Horizontal Component of PGA, PGV and *IMs*

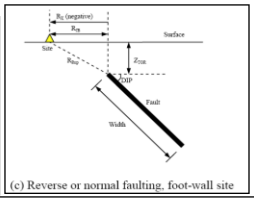
GMP	*T* (s)	Baseline: 5% Damping				User defined: 5% Damping			
		PSa Median for 5% damping	PSa Median + 1.σ for 5% damping	PSa Median - 1.σ for 5% damping	S_d Median for 5% damping	PSa Median for 5% damping	PSa Median + 1.σ for 5 % damping	PSa Median - 1.σ for 5 % damping	Sd Median for 5 % damping
PSa (g), S_d (cm)	**0.01**	0.3835155	0.6366977	0.231011	0.000952	0.383515	0.6366977	0.231011	0.000952
	0.02	0.3813894	0.6359577	0.228723	0.003787	0.381389	0.6359577	0.228723	0.003787
	0.03	0.3809259	0.6378317	0.227497	0.00851	0.380545	0.6371939	0.227269	0.008502
	0.05	0.4063868	0.6883907	0.239908	0.02522	0.406387	0.6883907	0.239908	0.02522
	0.075	0.4756873	0.8148786	0.277684	0.066422	0.477114	0.8173232	0.278517	0.066621
	0.1	0.5492158	0.9389254	0.321259	0.136336	0.550863	0.9417422	0.322223	0.136745
	0.15	0.6806703	1.1405579	0.406215	0.380177	0.682712	1.1439796	0.407434	0.381317
	0.2	0.7829452	1.297868	0.472315	0.777424	0.784511	1.3004638	0.47326	0.778978
	0.25	0.8592518	1.4306566	0.516066	1.333113	0.86183	1.4349486	0.517615	1.337112
	0.3	0.9100874	1.543215	0.53671	2.033255	0.911908	1.5463014	0.537784	2.037322
	0.4	0.9345762	1.6349049	0.534241	3.711941	0.936445	1.6381747	0.535309	3.719365
	0.5	0.9116648	1.6344062	0.508523	5.657721	0.912576	1.6360407	0.509031	5.663379
	0.75	0.7493947	1.4054866	0.399571	10.46404	0.749395	1.4054866	0.399571	10.46404
	1	0.6401862	1.2366468	0.331411	15.89178	0.639546	1.2354102	0.33108	15.87589
	1.5	0.4887143	0.964879	0.247535	27.29631	0.489203	0.9658439	0.247783	27.32361
	2	0.3883103	0.7749882	0.194564	38.55717	0.387534	0.7734382	0.194175	38.48006
	3	0.2755349	0.5529874	0.13729	61.55814	0.275259	0.5524344	0.137152	61.49658
	4	0.205369	0.4081994	0.103323	81.56825	0.205164	0.4077912	0.10322	81.48668
	5	0.1553114	0.3093533	0.077974	96.38508	0.155001	0.3087346	0.077818	96.19231
	7.5	0.0819255	0.1627376	0.041243	114.3953	0.081598	0.1620866	0.041078	113.9377
	10	0.0473547	0.0931848	0.024065	117.5518	0.047165	0.092812	0.023968	117.0816
PGA (g)	**0**	0.3813813	0.632705	0.229889	0.000947	0.383515	0.6366977	0.231011	0.000952
PGV (cm/s)	**-1**	68.879089	121.68825	38.98757	0.170983	NA	NA	NA	NA



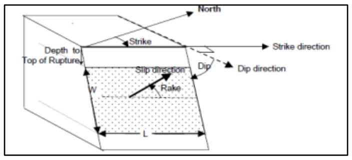
Input variables	**Errors and warnings**
M_w	
8.01	
R_{RUP} (km)	
9.81	
R_{JB} (km)	
9.81	
R_X (km)	
9.81	
Ry0 (km)	*If unknown use 999*
999	
V_{S30} (m/sec)	
260	
U (BSSA13)	*1: Unspecified fault mech.*
1	
F_{RV}	*1: reverse fault*
0	
F_{NM}	*1: normal fault*
0	
F_{HW}	*1: hanging wall side*
0	
Dip (deg)	
90	
Z_{TOR} (km)	*If unknown use 999*
999	
Z_{HYP} (km)	*If unknown use 999*
10.27	
$Z_{1.0}$ (km)	*If unknown use 999*
999	
$Z_{2.5}$ (km)	*If unknown use 999*
999	
W (km)	*If unknown use 999*
15.81	
Vs30Flag	
measured	*Choose options for V_{s30} from the list*
F_{AS}	
no	*Aftershock effect is not applicable.*
Region	
California	*Choose region from the list*

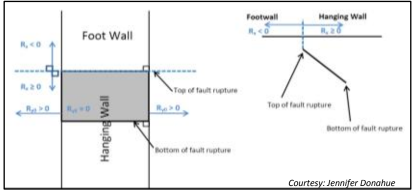


(a) Strike slip faulting (b) Reverse or normal faulting, hanging-wall site



(c) Reverse or normal faulting, foot-wall site





Courtesy: Jennifer Donahue

Definition of Parameters

Damping ratio = Viscous damping ratio (%) See Sanaz et al. (2012) PEER Report
PSA = Pseudo-absolute acceleration response spectrum (g)
PGA = Peak ground acceleration (g)
PGV = Peak ground velocity (cm/s)
S_d = Relative displacement response spectrum (cm)
M_w = Moment magnitude
R_{RUP} = Closest distance to coseismic rupture (km), used in ASK13, CB13 and CY13. See Figures a, b and c for illustation
R_{JB} = Closest distance to surface projection of coseismic rupture (km). See Figures a, b and c for illustation
R_X = Horizontal distance from top of rupture measured perpendicular to fault strike (km). See Figures a, b and c for illustation
R_{Y0} = The horizontal distance off the end of the rupture measured parallel to strike (km)
V_{S30} = The average shear-wave velocity (m/s) over a subsurface depth of 30 m
U = Unspecified-mechanism factor: 1 for unspecified; 0 otherwise
F_{RV} = Reverse-faulting factor: 0 for strike slip, normal, normal-oblique; 1 for reverse, reverse-oblique and thrust
F_{NM} = Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique, thrust and normal-oblique; 1 for normal
F_{HW} = Hanging-wall factor: 1 for site on down-dip side of top of rupture; 0 otherwise
Dip = Average dip of rupture plane (degrees)
Z_{TOR} = Depth to top of coseismic rupture (km)
Z_{HYP} = Hypocentral depth from the earthquake
$Z_{1.0}$ = Depth to Vs=1 km/sec
$Z_{2.5}$ = Depth to Vs=2.5 km/sec
W = Fault rupture width (km)
$V_{s30flag}$ = 1 for measured, 0 for inferred Vs30
F_{AS} = 0 for mainshock; 1 for aftershock
Region = Specific regions considered in the models, Click on Region to see codes
ΔDPP = Directivity term, direct point parameter; uses 0 for median predictions
PGA_r (g) = Peak ground acceleration on rock (g), this specific cell is updated in the cell for BSSA14 and CB14, for others it is taken account for in the macros
Z_{BOT} (km) = The depth to the bottom of the seismogenic crust
Z_{BOR} (km) = The depth to the bottom of the rupture plane
SS = 1 for strike slip, automatically updated in the cell

Calculated Variables/Flags	
ΔDPP	*Always 0 for median calcs.*
0	
PGA_r (g)	
0.318	
Z_{BOT} (km) (CB14)	*Enter for default W calcs*
15	
SS	
1	*auto calculated*
$V_{s30Flag}$	
1	*measured*
F_{AS}	
0	*Aftershock effect is not applicable.*
Region	
0	*California*
Option for Sa value	
1	*Weighted average of the natural logarithm of the spectral values*

Input variables with defaults (If entered 999 as input):

Red colored value: The value is used in the code when input is unknown

DEFAULTs	**USER defined**	**ASK14**	**BSSA14**	**CB14**	**CY14**	**I14**
W (km)	15.81	▨	▨	15.000	▨	▨
$Z_{1.0}$ (km)	999.000	0.475	▨	▨	0.485	▨
$\delta Z_{1.0}$ (km)	0.000	▨	0.000	▨	▨	▨
$Z_{2.5}$ (V_{S30}=1100)(km)	999.000	▨	▨	0.398	▨	▨
$Z_{2.5}$ (V_{S30})(km)	999.000	▨	▨	2.070	▨	▨
Z_{hyp} (km)	10.27	▨	▨	10.227	▨	▨
Z_{tor} (km)	999.00	▨	▨	0.000	0.000	▨
Z_{BOR} (km)	-	▨	▨	15.000	▨	▨

ACKNOWLEDGEMENTS

Nick Gregor, Bechtel
Silvia Mazzoni, Consultant

All NGA West-2 participants are acknowledged for their constructive comments and feedback.

E. Colton Avenue and N. Wabash Avenue, Redlands

Latitude, Longitude: 34.0638, -117.1400

Date	3/22/2022, 12:45:04 PM
Design Code Reference Document	ASCE7-16
Risk Category	III
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.914	MCE_R ground motion. (for 0.2 second period)
S_1	0.789	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.914	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.276	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.819	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.901	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	2.587	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	2.831	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.914	Factored deterministic acceleration value. (0.2 second)
S1RT	1.018	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	1.143	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.789	Factored deterministic acceleration value. (1.0 second)
PGAd	0.819	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.914	Mapped value of the risk coefficient at short periods
C_{R1}	0.891	Mapped value of the risk coefficient at a period of 1 s

DISCLAIMER

U.S. Geological Survey - Earthquake Hazards Program

Unified Hazard Tool

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

^ Input

Edition

Dynamic: Conterminous U.S. 2014 (upd…

Latitude

Decimal degrees

34.0638

Longitude

Decimal degrees, negative values for western longitudes

-117.14

Site Class

259 m/s (Site class D)

Spectral Period

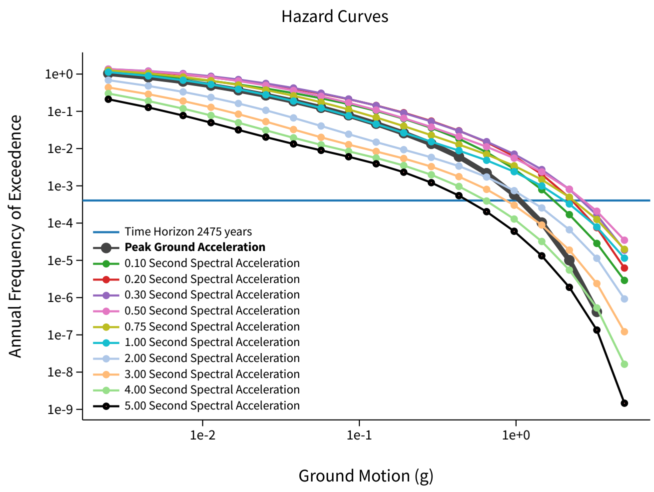
Peak Ground Acceleration

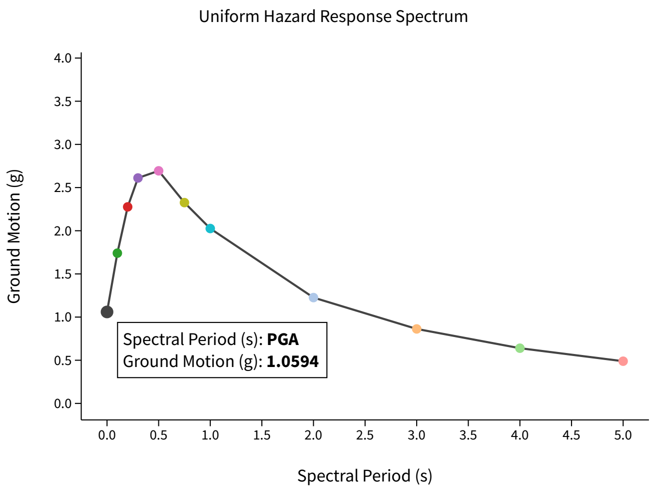
Time Horizon

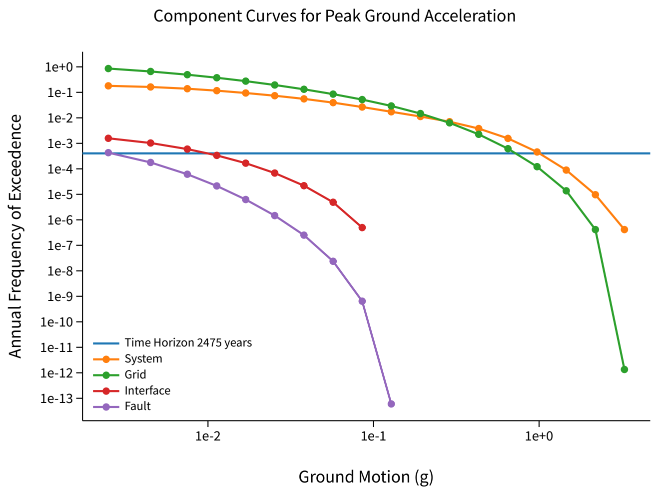
Return period in years

2475

Hazard Curve


Hazard Curves


Uniform Hazard Response Spectrum

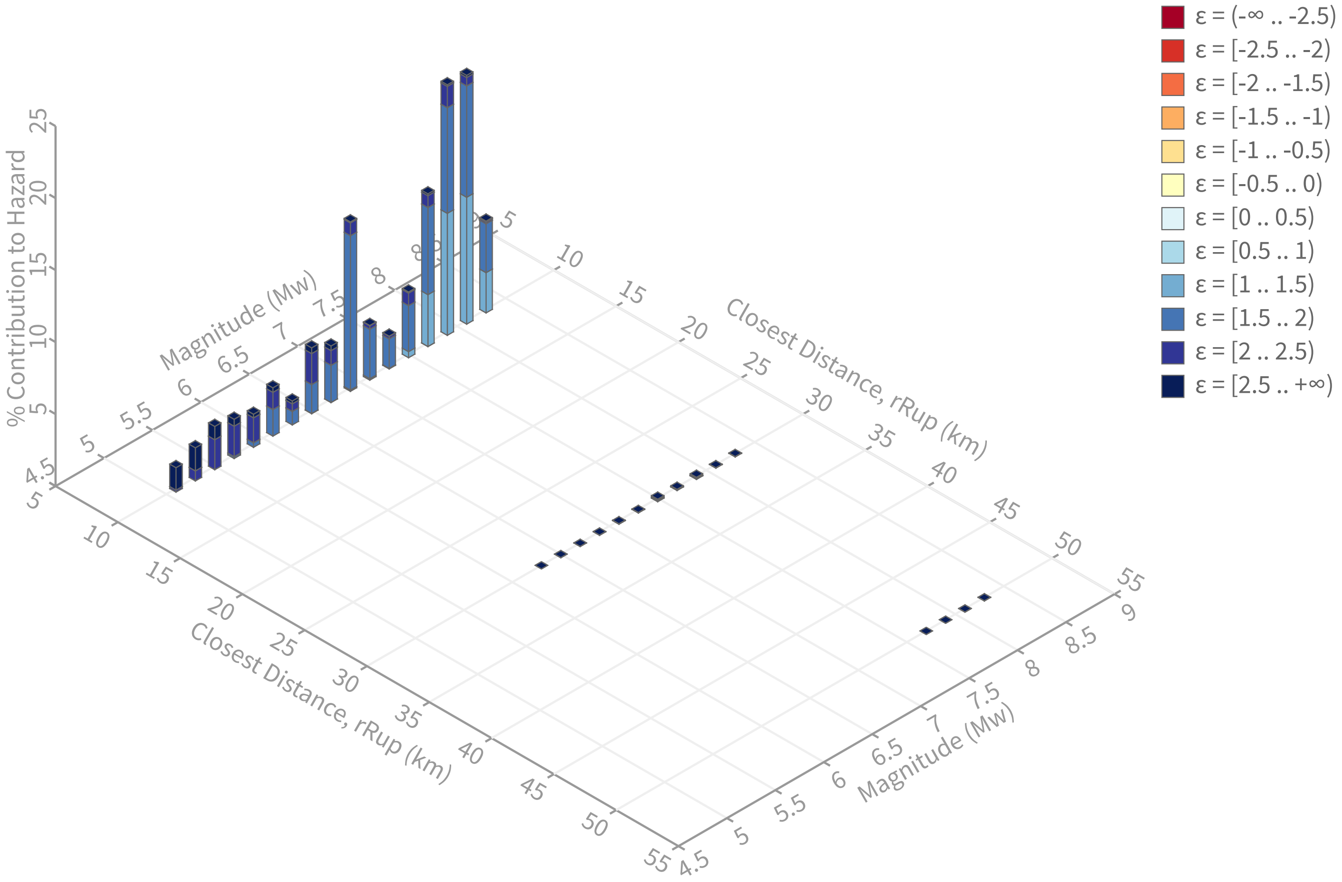

Component Curves for Peak Ground Acceleration

View Raw Data

^ Deaggregation

Component

Total ⌄



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 1.0593831 g

Recovered targets

Return period: 3321.3976 yrs
Exceedance rate: 0.00030107808 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.03 %

Mean (over all sources)

m: 7.27
r: 7.32 km
ε_0: 1.8 σ

Mode (largest m-r bin)

m: 7.91
r: 7.19 km
ε_0: 1.6 σ
Contribution: 17.39 %

Mode (largest m-r-ε_0 bin)

m: 6.84
r: 5.81 km
ε_0: 1.81 σ
Contribution: 10.71 %

Discretization

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

Epsilon keys

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

Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
UC33brAvg_FM31	System							41.52
San Andreas (San Bernardino S) [1]		5.81	7.46	1.67	117.117°W	34.111°N	22.38	26.23
San Jacinto (San Jacinto Valley) rev [0]		9.81	8.02	1.78	117.215°W	34.002°N	225.08	7.41
San Andreas (North Branch Mill Creek) [1]		7.08	7.94	1.40	117.111°W	34.123°N	21.83	3.54
San Andreas (San Bernardino S) [2]		6.03	6.79	1.89	117.099°W	34.104°N	39.89	1.39
UC33brAvg_FM32	System							41.39
San Andreas (San Bernardino S) [1]		5.81	7.47	1.66	117.117°W	34.111°N	22.38	26.35
San Jacinto (San Jacinto Valley) rev [0]		9.81	8.01	1.78	117.215°W	34.002°N	225.08	7.39
San Andreas (North Branch Mill Creek) [1]		7.08	7.95	1.40	117.111°W	34.123°N	21.83	3.64
San Andreas (San Bernardino S) [2]		6.03	6.81	1.88	117.099°W	34.104°N	39.89	1.23
UC33brAvg_FM31 (opt)	Grid							8.54
PointSourceFinite: -117.140, 34.122		8.07	5.70	2.26	117.140°W	34.122°N	0.00	1.70
PointSourceFinite: -117.140, 34.122		8.07	5.70	2.26	117.140°W	34.122°N	0.00	1.70
PointSourceFinite: -117.140, 34.113		7.45	5.65	2.20	117.140°W	34.113°N	0.00	1.32
PointSourceFinite: -117.140, 34.113		7.45	5.65	2.20	117.140°W	34.113°N	0.00	1.32
UC33brAvg_FM32 (opt)	Grid							8.54
PointSourceFinite: -117.140, 34.122		8.07	5.70	2.26	117.140°W	34.122°N	0.00	1.70
PointSourceFinite: -117.140, 34.122		8.07	5.70	2.26	117.140°W	34.122°N	0.00	1.70
PointSourceFinite: -117.140, 34.113		7.45	5.65	2.20	117.140°W	34.113°N	0.00	1.32
PointSourceFinite: -117.140, 34.113		7.45	5.65	2.20	117.140°W	34.113°N	0.00	1.32

APPENDIX E
STANDARD GRADING GUIDELINES

STANDARD GRADING SPECIFICATIONS

These specifications present the usual and minimum requirements for grading operations performed under the observation and testing of TGR Geotechnical, Inc.

No deviation from these specifications will be allowed, except where specifically superseded in the Preliminary Geotechnical Investigation report, or in other written communication signed by the Soils Engineer or Engineering Geologist.

1.0 GENERAL

- The Soils Engineer and Engineering Geologist are the Owner's or Builder's representatives on the project. For the purpose of these specifications, observation and testing by the Soils Engineer includes that observation and testing performed by any person or persons employed by, and responsible to, the licensed Geotechnical Engineer or Geologist signing the grading report.

- All clearing, site preparation or earthwork performed on the project shall be conducted by the Contractor under the observation of the Geotechnical Engineer.

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

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

- A final report will be issued by the Geotechnical Engineer and Engineering Geologist attesting to the Contractor's conformance with these specifications.

2.0 SITE PREPARATION

- All vegetation and deleterious material such as rubbish shall be disposed of off-site. The removal must be concluded prior to placing fill.

- The Civil Engineer shall locate all houses, sheds, sewage disposal systems, large trees or structures on the site, or on the grading plan to the best of his knowledge prior to preparing the ground surface.

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

- After the ground surface to receive fill has been cleared, it shall be scarified, disced or bladed by the Contractor until it is uniform and free from ruts, hollows, hummocks or other uneven features which may prevent uniform compaction.

  The scarified ground surface shall then be brought to optimum moisture content, mixed as required, and compacted as specified. If the scarified zone is greater than twelve inches in depth, the excess shall be removed and placed in lifts restricted to six inches. Prior to placing fill, the ground surface to receive fill shall be inspected, tested and approved by the Geotechnical Engineer.

- Any underground structures such as cesspools, cisterns, mining shafts, tunnels, septic tanks, wells, pipe lines or others not located prior to grading are to be removed or treated in a manner prescribed by the Geotechnical Engineer.

3.0 COMPACTED FILLS

- Any material imported or excavated on the property may be utilized in the fill, provided each material has been determined to be suitable by the Geotechnical Engineer. Roots, tree branches and other matter missed during clearing shall be removed from the fill as directed by the Geotechnical Engineer.
- Rock fragments less than six inches in diameter may be utilized in the fill, provided:

  - They are not placed in concentrated pockets.
  - There is a sufficient percentage of fine-grained material to surround the rocks.
  - The distribution of the rocks is observed by the Geotechnical Engineer.

- Rocks greater than six inches in diameter shall be taken off-site, or placed in accordance with the recommendations of the Geotechnical Engineer in areas designated as suitable for rock disposal. Details for rock disposal such as location, moisture control, percentage of the rock placed, etc., will be referred to in the "Conclusions and Recommendations" section of the Geotechnical Report, if applicable.

  If rocks greater than six inches in diameter were not anticipated in the Preliminary Geotechnical report, rock disposal recommendations may not have been made in the "Conclusions and Recommendations" section. In this case, the Contractor shall notify the Geotechnical Engineer if rocks greater than six inches in diameter are encountered. The Geotechnical Engineer will then prepare a rock disposal recommendation or request that such rocks be taken off-site.

- Material that is spongy, subject to decay, or otherwise considered unsuitable shall not be used in the compacted fill.

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

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

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

- Each layer shall be compacted to 90 percent of the maximum dry density in compliance with the testing method specified by the controlling governmental agency; (in general, ASTM D1557 will be used.)

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

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

- The key for side hill fills shall be a minimum of 15 feet within bedrock or firm materials, unless otherwise specified in the Preliminary report. (See details)

- Drainage terraces and subdrainage devices shall be constructed in compliance with the ordinances of the controlling governmental agency, or with the recommendation of the Geotechnical Engineer and Engineer Geologist.

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

The Contractor shall prepare a written detailed description of the method or methods he will employ to obtain the required slope compaction. Such documents shall be submitted to the Geotechnical Engineer for review and comments prior to the start of grading.

If a method other than overbuilding and cutting back to the compacted core is to be employed, slope tests will be made by the Geotechnical Engineer during construction of the slopes to determine if the required compaction is being achieved. Where failing tests occur or other field problems arise, the contractor will be notified by the Geotechnical Engineer.

If the method of achieving the required slope compaction selected by the Contractor fails to produce the necessary results, the Contractor shall rework or rebuild such slopes until the required degree of compaction is obtained, at no additional cost to the Owner or Geotechnical Engineer.

- All fill slopes should be planted or protected from erosion by methods specified in the preliminary report or by means approved by the governing authorities.

- Fill-over-cut slopes shall be properly keyed through topsoil, colluvium or creep material into rock or firm materials; and the transition shall be stripped of all soil prior to placing fill. (See detail)

4.0 CUT SLOPES

- The Engineering Geologist shall inspect all cut slopes excavated in rock, lithified or formation material at vertical intervals not exceeding ten feet.

- If any conditions not anticipated in the preliminary report such as perched water, seepage, lenticular or confined strata of a potentially adverse nature, unfavorably inclined bedding, joints or fault planes are encountered during grading, these conditions shall be analyzed by the Engineering Geologist and Geotechnical Engineer; and recommendations shall be made to treat these problems.

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

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

- Drainage terraces shall be constructed in compliance with the ordinances of controlling governmental agencies, or with the recommendations of the Geotechnical Engineer or Engineering Geologist.

5.0 GRADING CONTROL

- Inspection of the fill placement shall be provided by the Geotechnical Engineer during the progress of grading.

- In general, density tests should be made at intervals not exceeding two feet of fill height or every 500 cubic yards of fill placed. This criteria will vary depending on soil conditions and the size of the job. In any event, an adequate number of field density tests shall be made to verify that the required compaction of being achieved.
- Density tests should be made on the surface material to receive fill as required by the Geotechnical Engineer.

- All cleanout, processed ground to receive fill, key excavations, subdrains and rock disposal must be inspected and approved by the Geotechnical Engineer (and often by the governing authorities) prior to placing any fill. It shall be the Contractor’s responsibility to notify the Geotechnical Engineer and governing authorities when such areas are ready for inspection.

6.0 CONSTRUCTION CONSIDERATIONS

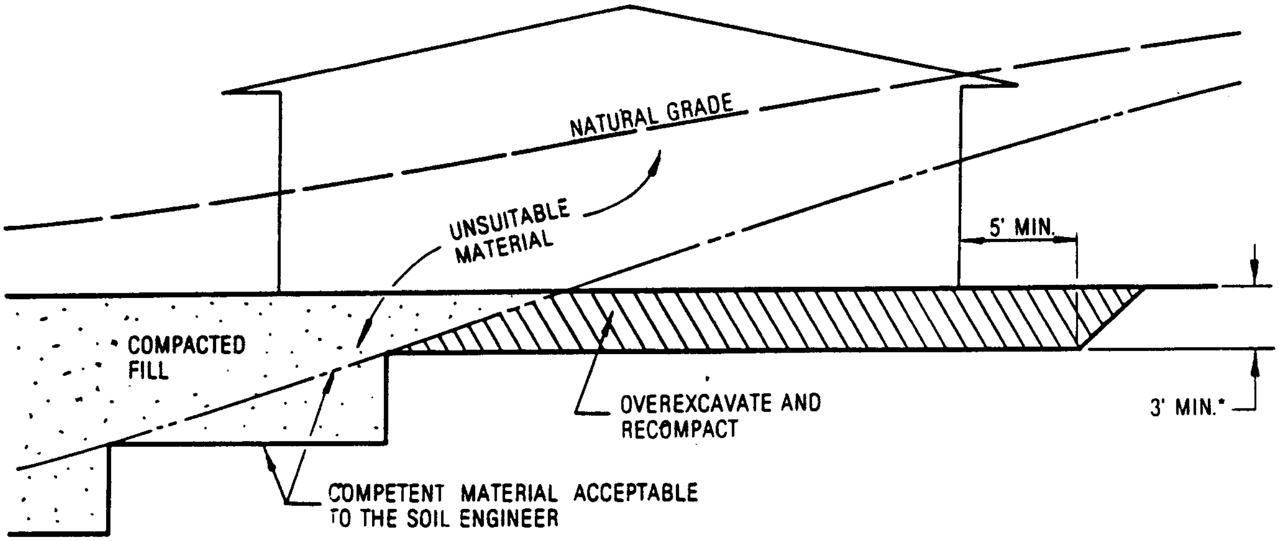
- Erosion control measures, when necessary, shall be provided by the Contractor during grading and prior to the completion and construction of permanent drainage controls.

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

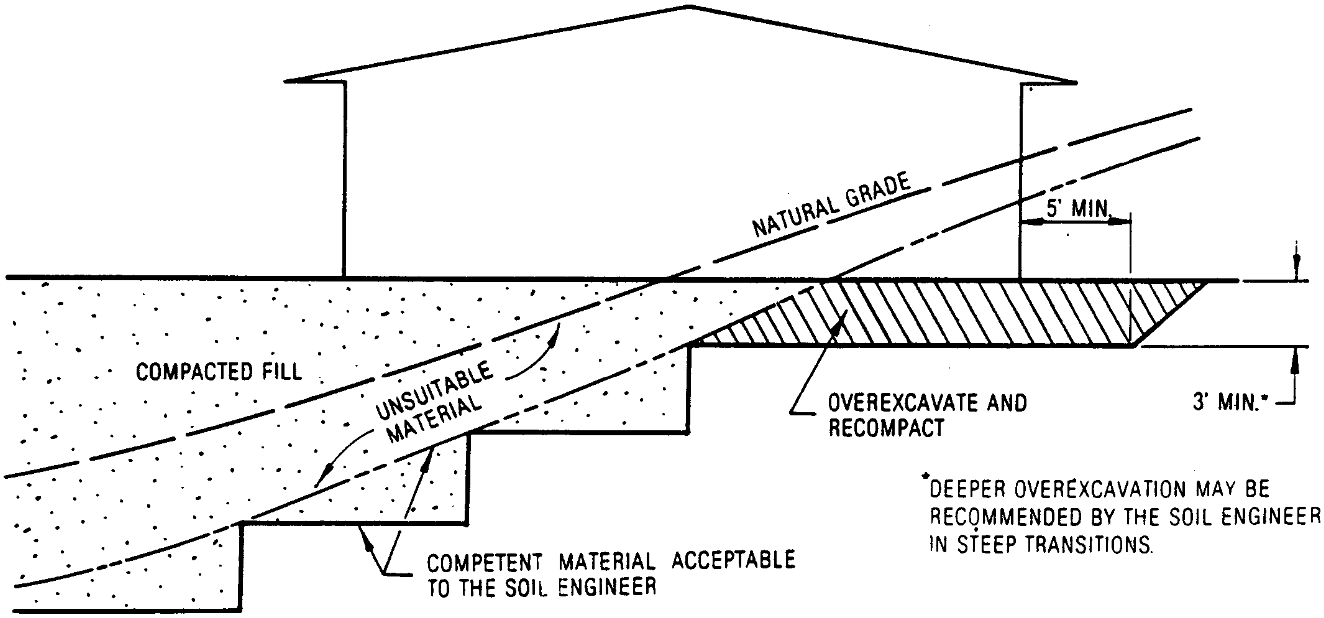
- Care shall be taken by the Contractor during final grading to preserve any berms, drainage terraces, interceptor swales, or other devices of a permanent nature on or adjacent to the property.

TYPICAL OVEREXCAVATION OF DAYLIGHT LINE

CUT LOT

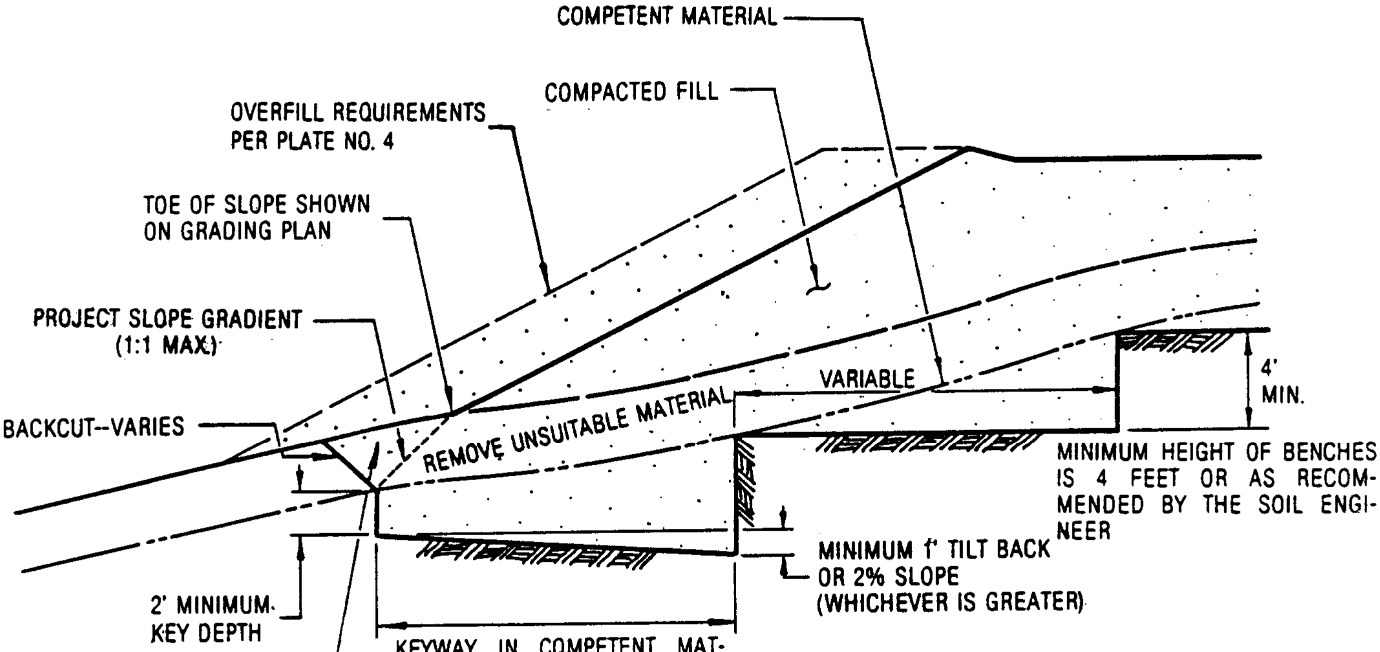


CUT FILL LOT (TRANSITION)



*DEEPER OVEREXCAVATION MAY BE RECOMMENDED BY THE SOIL ENGINEER IN STEEP TRANSITIONS.

TYPICAL FILL OVER NATURAL SLOPE

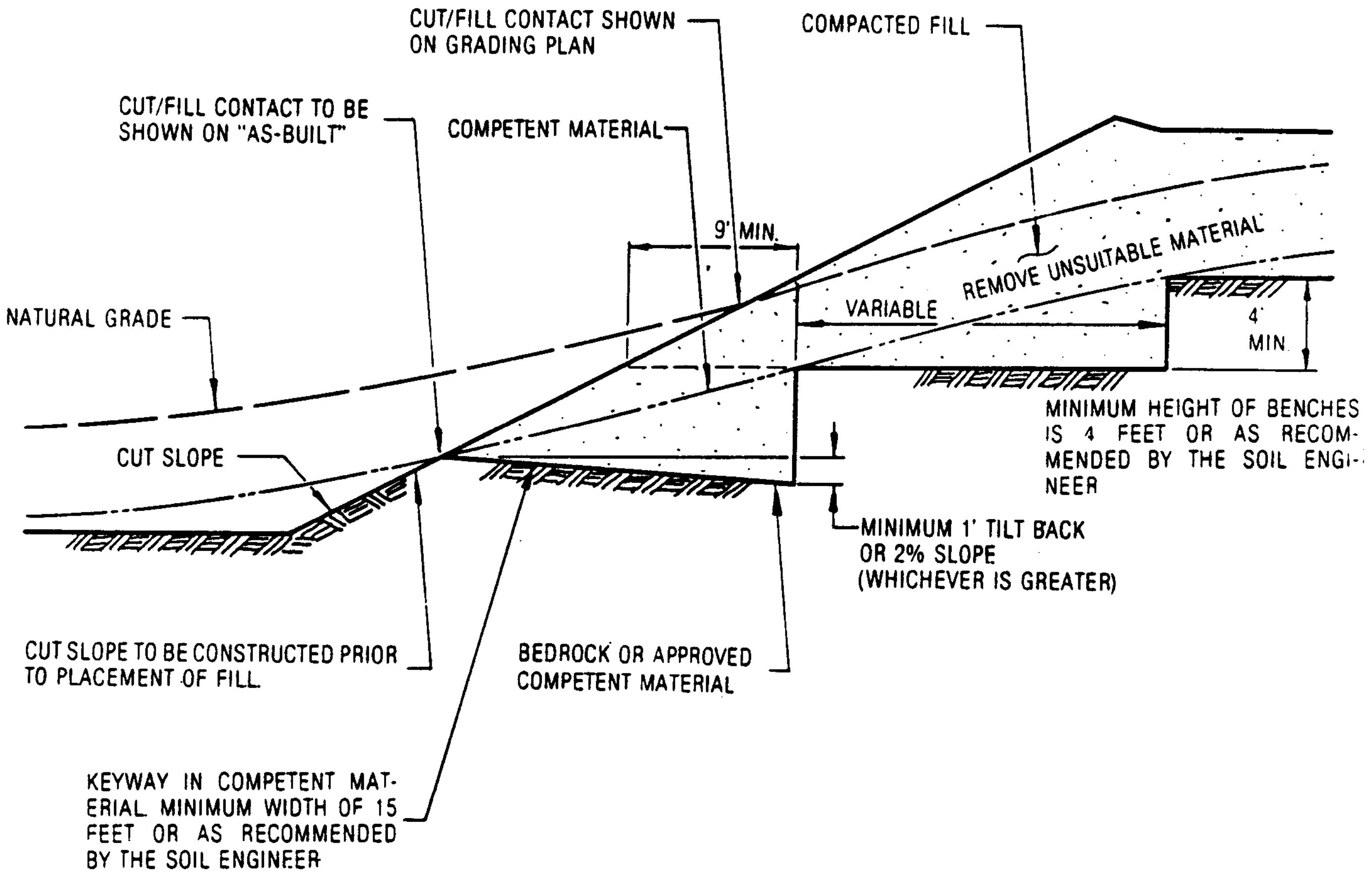


MINIMUM HEIGHT OF BENCHES IS 4 FEET OR AS RECOMMENDED BY THE SOIL ENGINEER

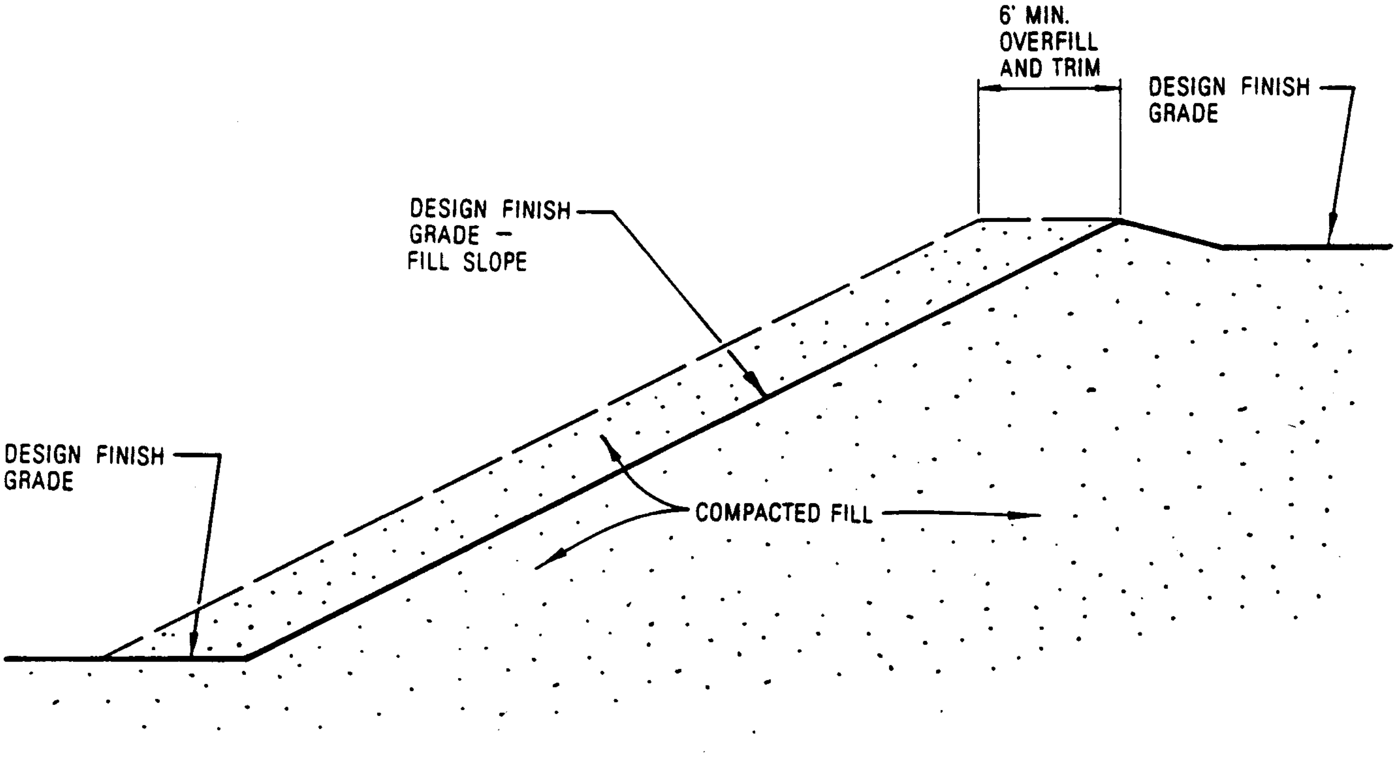
KEYWAY IN COMPETENT MATERIAL. MINIMUM WIDTH OF 15 FEET OR AS RECOMMENDED BY THE SOIL ENGINEER. KEYWAY MAY NOT BE REQUIRED IF FILL SLOPE IS LESS THAN 5' IN HEIGHT. AS RECOMMENDED BY THE SOIL ENGINEER.

NOTE:
BENCHING SHALL BE REQUIRED WHEN NATURAL SLOPES ARE EQUAL TO OR STEEPER THAN 5:1 OR WHEN RECOMMENDED BY THE SOIL ENGINEER.

TYPICAL FILL-OVER-CUT SLOPE



TYPICAL FILL SLOPE CONSTRUCTION



NOTES:

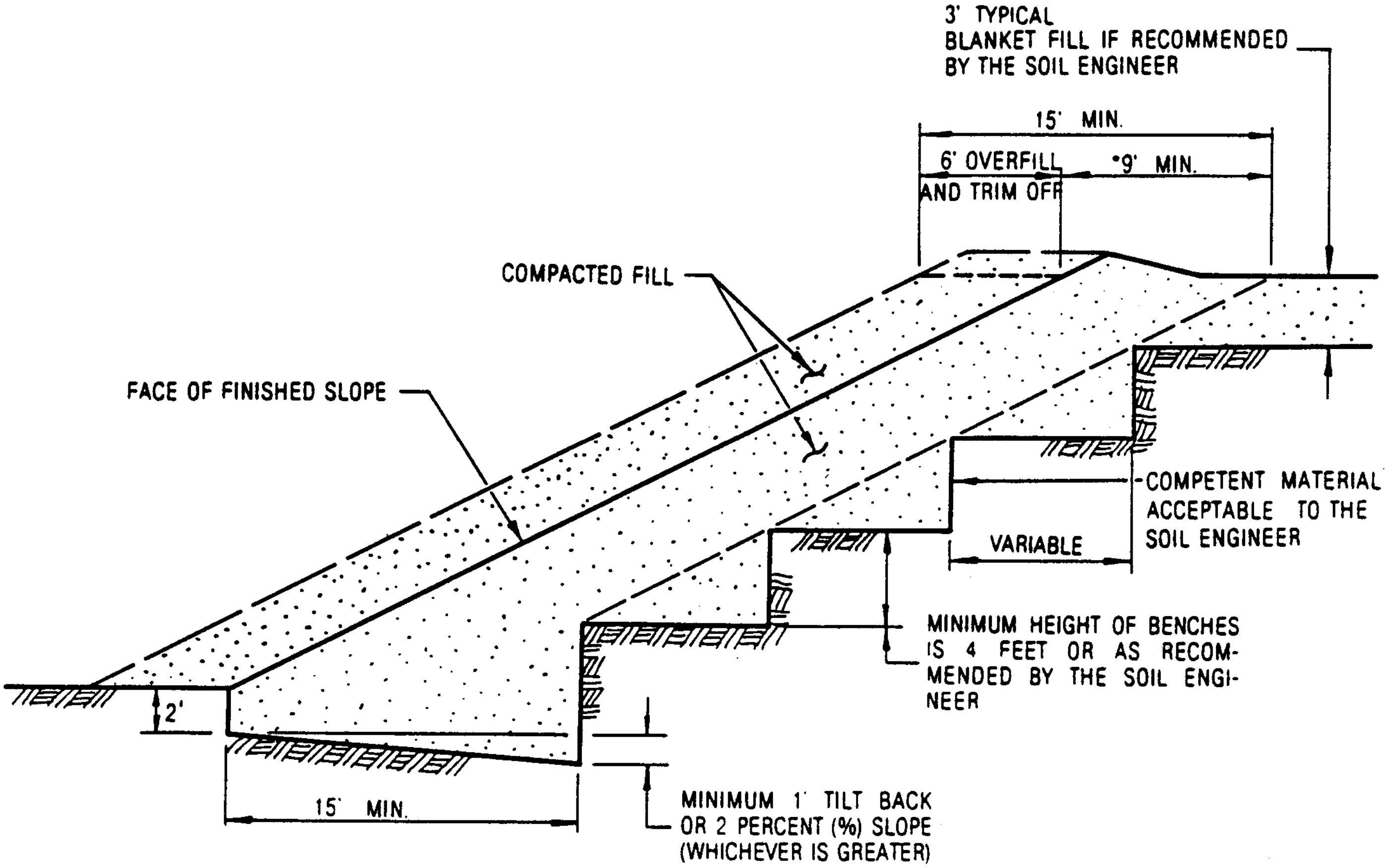
1. ALL FILL SLOPES, INCLUDING BUTTRESS AND STABILIZATION FILLS, SHALL BE OVERFILLED A MINIMUM OF SIX FEET HORIZONTALLY WITH COMPACTED FILL AND TRIMMED TO THE DESIGN FINISH GRADE.

   EXCEPTIONS:

   A. FILL SLOPE OVER CUT SLOPE.
   B. FILL SLOPE ADJACENT TO EXISTING IMPROVEMENTS.

2. THE EXCEPTIONS ABOVE WHICH DO NOT HAVE THE 6 FOOT SLOPE OVERFILL AND TRIM SHALL BE COMPACTED AS STATED IN THE PROJECT SPECIFICATIONS.

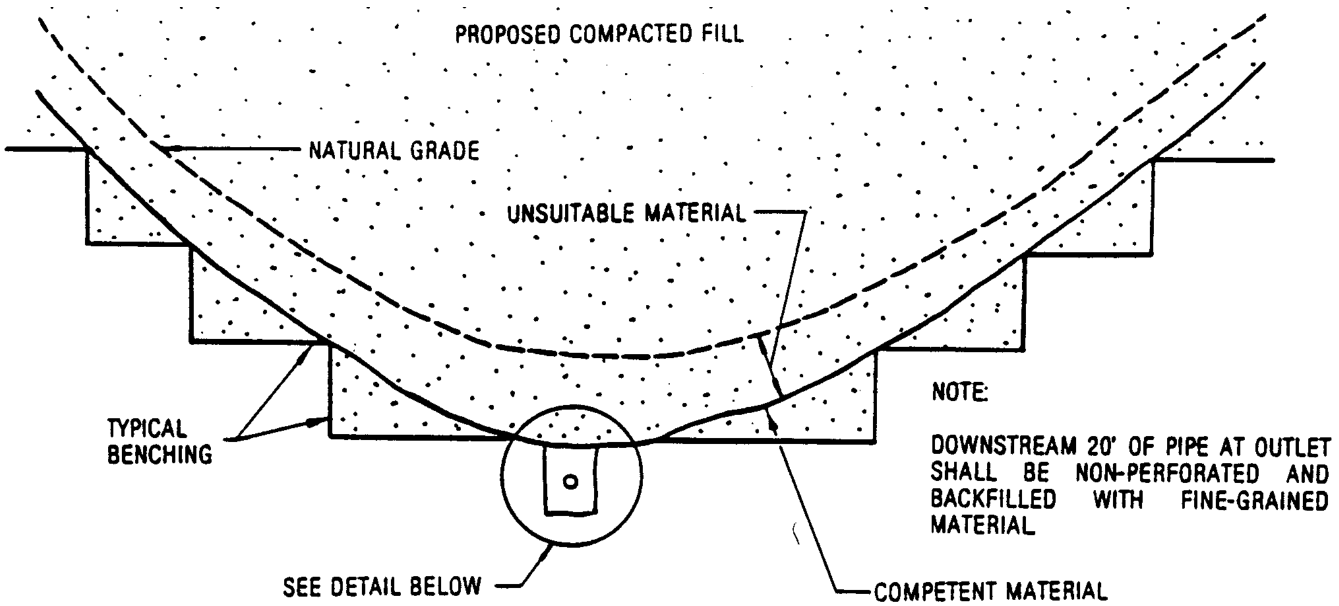
TYPICAL STABILIZATION FILL



NOTE:
SEE PLATE 6 FOR TYPICAL SUBDRAIN DETAILS FOR STABILIZATION FILLS. IF RECOMMENDED BY THE SOIL ENGINEER.

*GREATER THAN 9' IF RECOMMENDED BY THE SOIL ENGINEER: 15' WHERE NO 6' OVERFILL

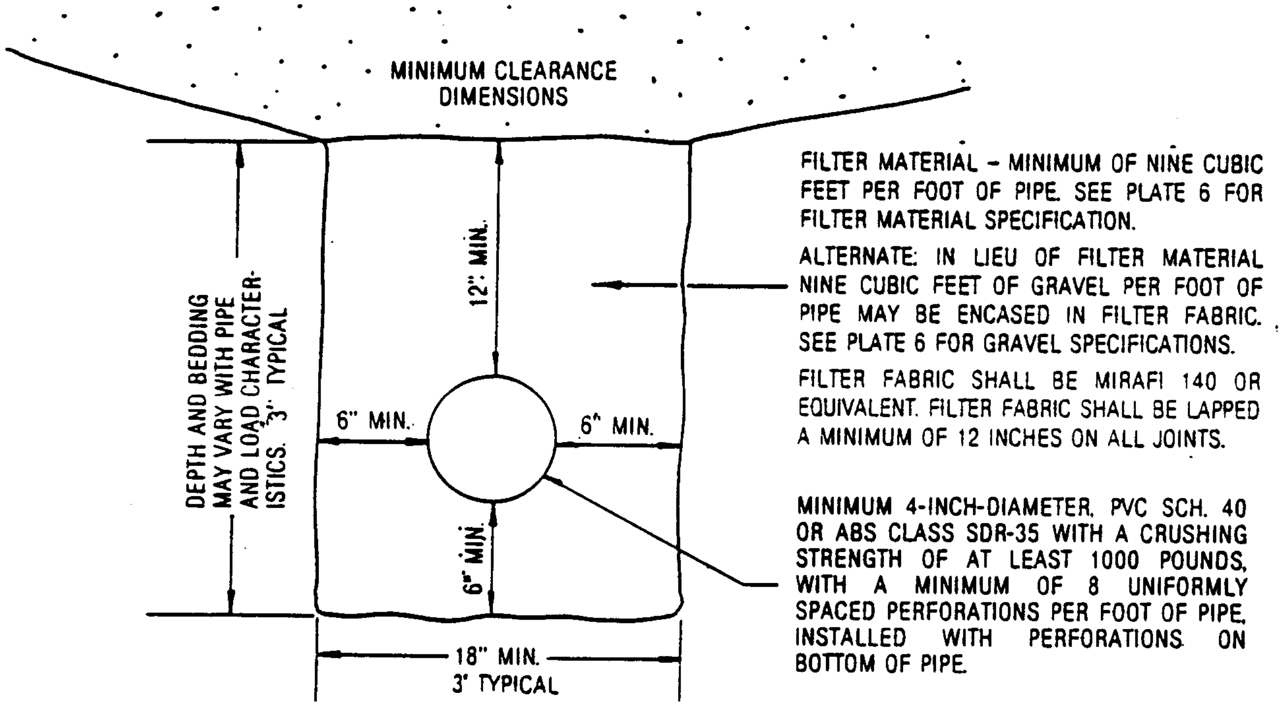
TYPICAL CANYON SUBDRAIN



NOTE:

DOWNSTREAM 20' OF PIPE AT OUTLET SHALL BE NON-PERFORATED AND BACKFILLED WITH FINE-GRAINED MATERIAL

NOTES:
PIPE SHALL BE A MINIMUM OF 4 INCHES DIAMETER AND RUNS OF 500 FEET OR MORE USE 6-INCH DIAMETER PIPE, OR AS RECOMMENDED BY THE SOIL ENGINEER



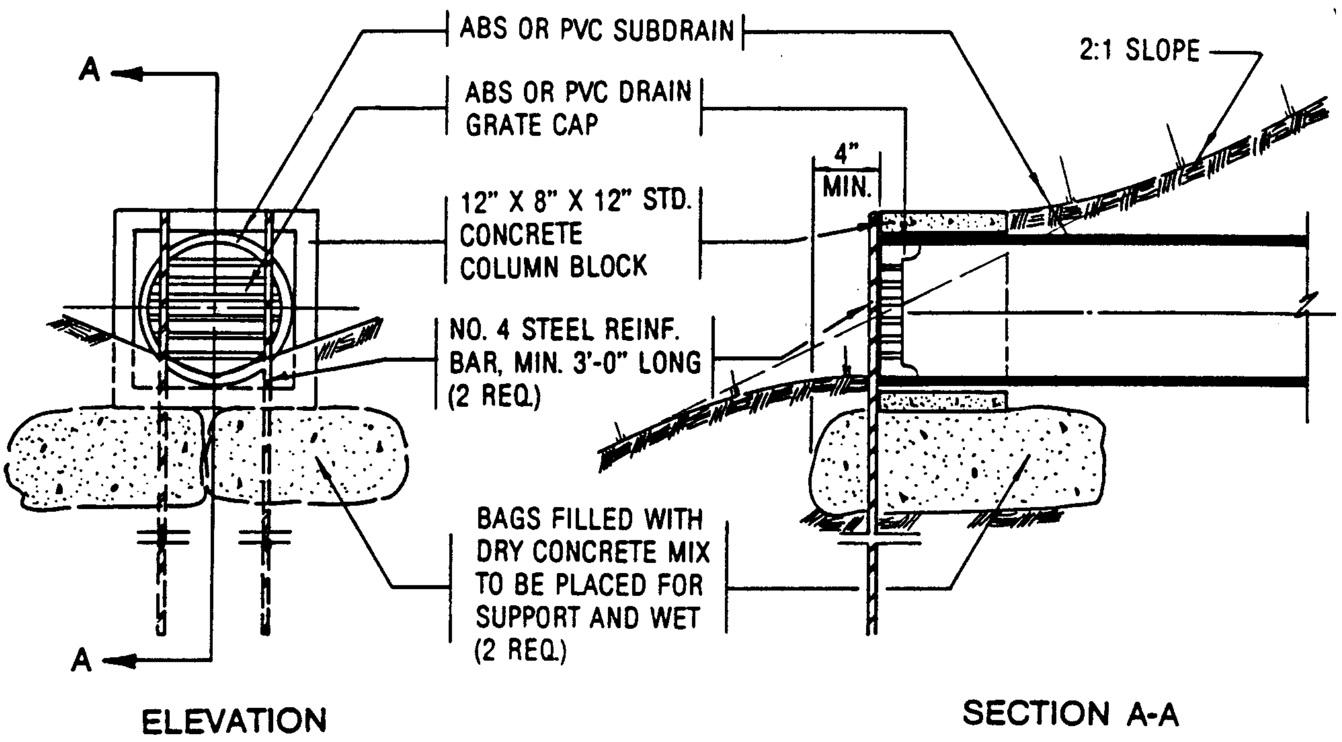
FILTER MATERIAL – MINIMUM OF NINE CUBIC FEET PER FOOT OF PIPE. SEE PLATE 6 FOR FILTER MATERIAL SPECIFICATION.

ALTERNATE: IN LIEU OF FILTER MATERIAL NINE CUBIC FEET OF GRAVEL PER FOOT OF PIPE MAY BE ENCASED IN FILTER FABRIC. SEE PLATE 6 FOR GRAVEL SPECIFICATIONS.

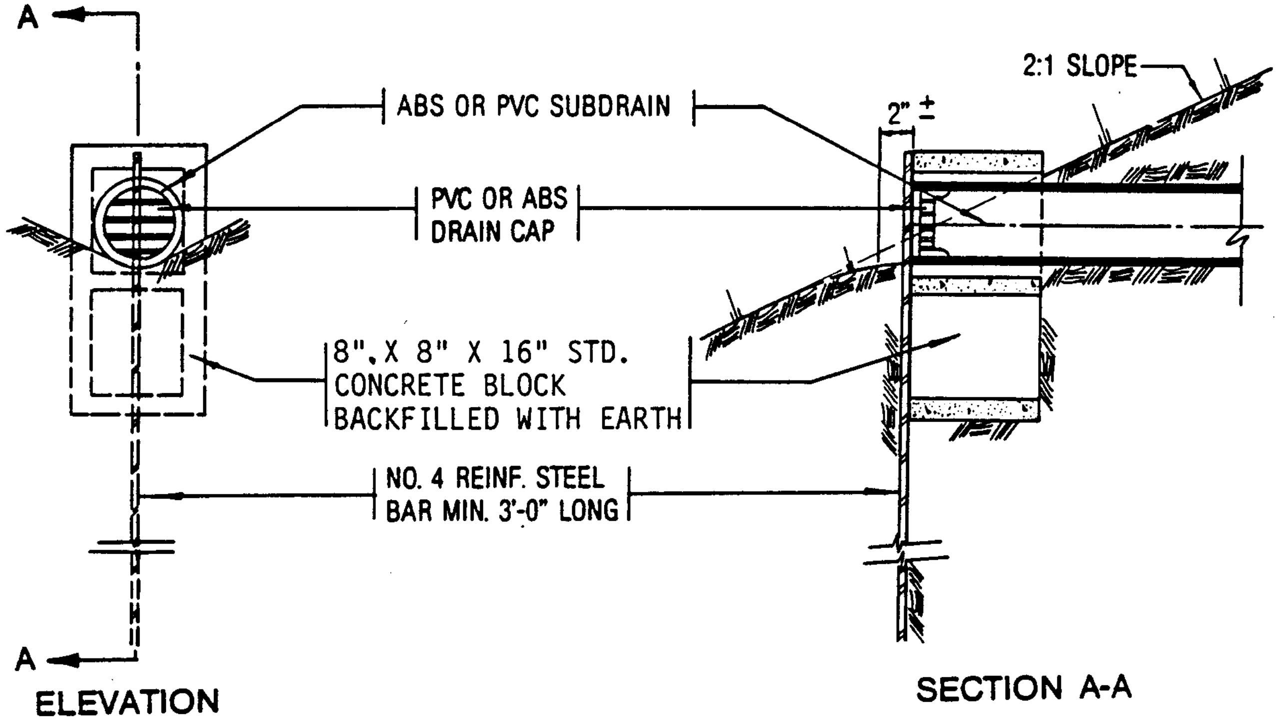
FILTER FABRIC SHALL BE MIRAFI 140 OR EQUIVALENT. FILTER FABRIC SHALL BE LAPPED A MINIMUM OF 12 INCHES ON ALL JOINTS.

MINIMUM 4-INCH-DIAMETER, PVC SCH. 40 OR ABS CLASS SDR-35 WITH A CRUSHING STRENGTH OF AT LEAST 1000 POUNDS, WITH A MINIMUM OF 8 UNIFORMLY SPACED PERFORATIONS PER FOOT OF PIPE, INSTALLED WITH PERFORATIONS ON BOTTOM OF PIPE.

SUBDRAIN OUTLET MARKER

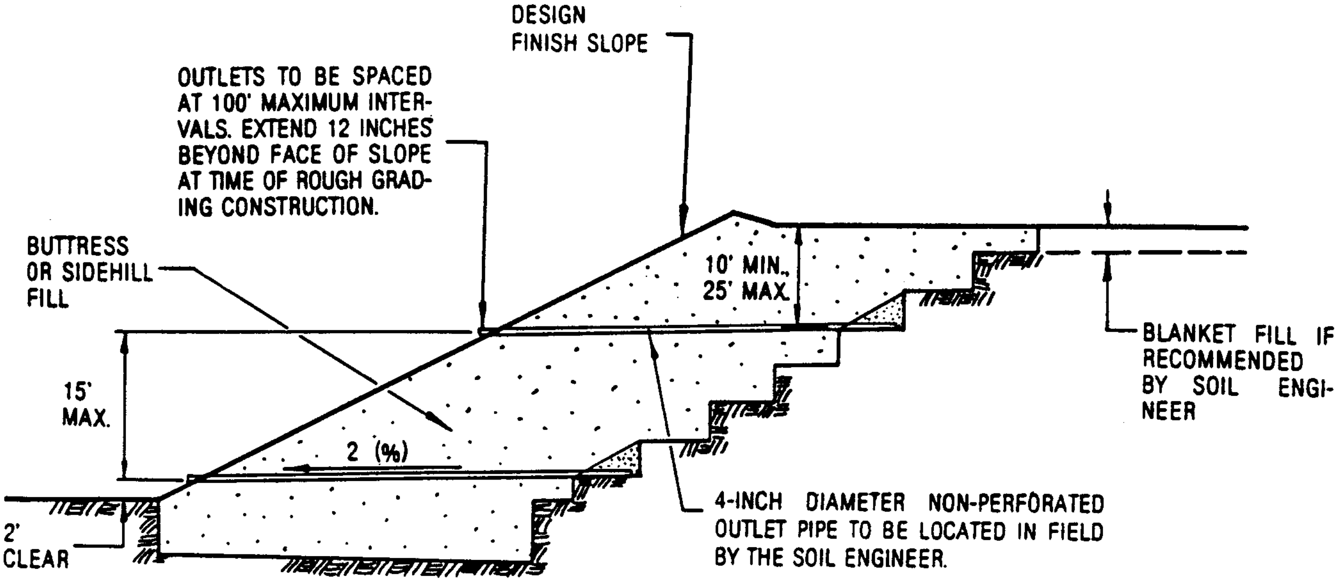


SUBDRAIN OUTLET MARKER FOR 6" AND 8" PIPES



SUBDRAIN OUTLET MARKER – 4" PIPE

TYPICAL STABILIZATION AND BUTTRESS FILL SUBDRAIN



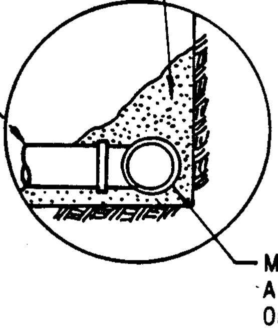
FILTER MATERIAL" TO MEET FOLLOWING SPECIFI-
;ATION OR APPROVED EQUIVALENT: (CONFORMS TO
:MA STD. PLAN 323)

SIEVE SIZE	PERCENTAGE PASSING
1"	100
3/4"	90-100
3/8"	40-100
NO. 4	25-40
NO. 8	18-33
NO. 30	5-15
NO. 50	0-7
NO. 200	0-3

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

SIEVE SIZE	MAXIMUM PERCENTAGE PASSING
1½"	100
NO. 4	50
NO. 200	8

SAND EQUIVALENT = MINIMUM OF 50



OUTLET PIPE TO BE CONNECTED TO SUBDRAIN PIPE WITH TEE OR ELBOW

FILTER MATERIAL - MINIMUM OF FIVE CUBIC FEET PER FOOT OF PIPE. SEE ABOVE FOR FILTER MATERIAL SPECIFICATION.

ALTERNATIVE: IN LIEU OF FILTER MATERIAL. FIVE CUBIC FEET OF GRAVEL PER FOOT OF PIPE MAY BE ENCASED IN FILTER FABRIC. SEE ABOVE FOR GRAVEL SPECIFICATION.

FILTER FABRIC SHALL BE MIRAFI 140 OR EQUIVALENT. FILTER FABRIC SHALL BE LAPPED A MINIMUM OF 12 INCHES ON ALL JOINTS.

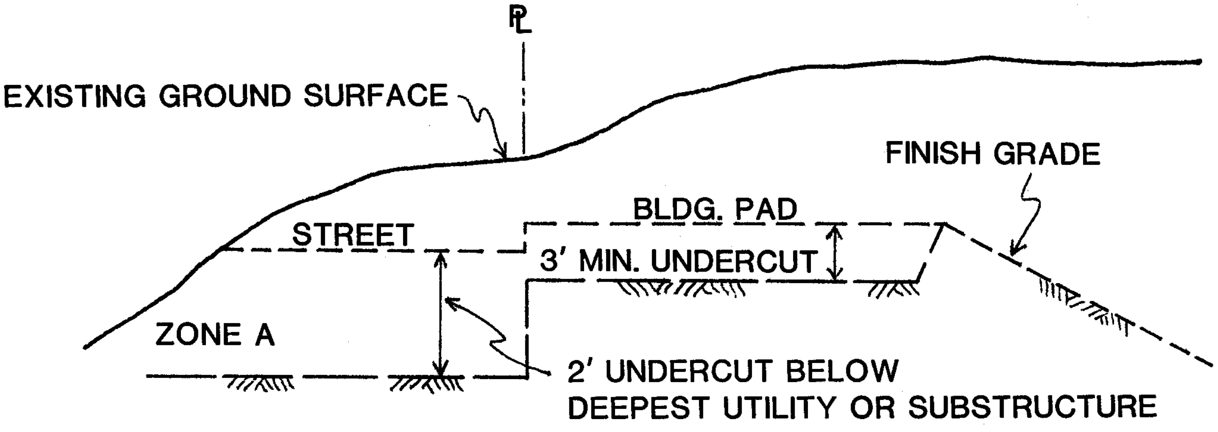
MINIMUM 4-INCH DIAMETER PVC SCH 40 OR ABS CLASS SDR 35 WITH A CRUSHING STRENGTH OF AT LEASE 1,000 POUNDS, WITH A MINIMUM OF 8 UNIFORMLY SPACED PERFORATIONS PER FOOT OF PIPE INSTALLED WITH PERFORATIONS ON BOTTOM OF PIPE. PROVIDE CAP AT UPSTREAM END OF PIPE. SLOPE AT 2 PERCENT TO OUTLET PIPE.

NOTES:

1. TRENCH FOR OUTLET PIPES TO BE BACKFILLED WITH ON-SITE SOIL.

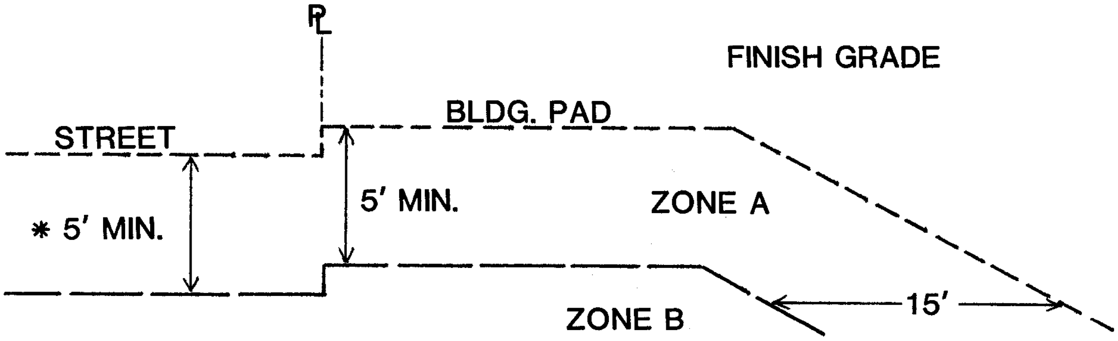
TYPICAL CUT AND FILL GRADING DETAILS

TYPICAL GRADING WITHIN PROPOSED DEEP BEDROCK CUT AREAS



NO SCALE

TYPICAL GRADING WITHIN PROPOSED FILL AREAS



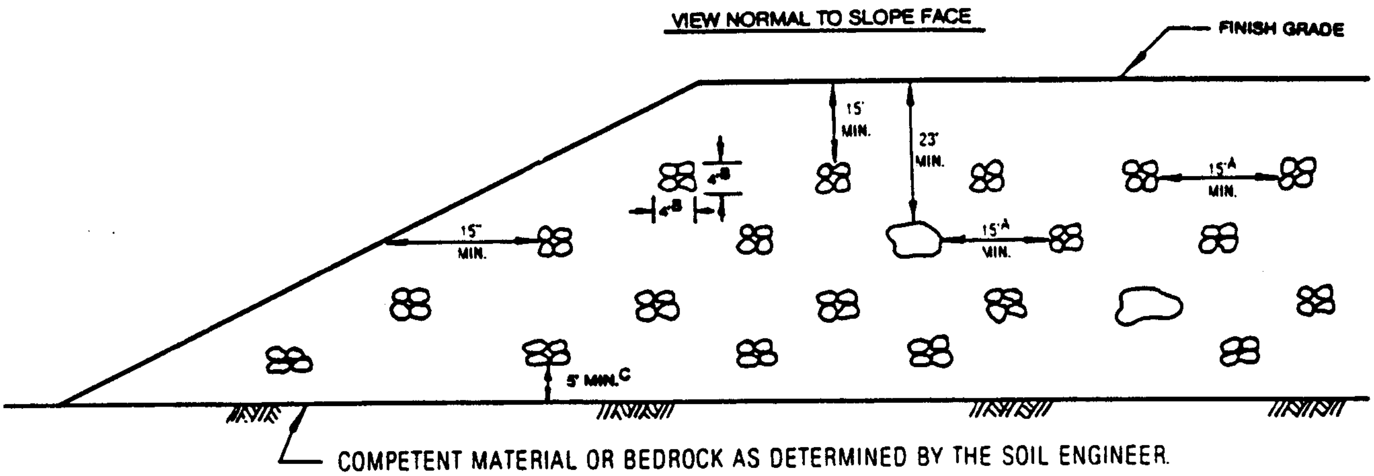
LEGEND

ZONE A ......"SOIL" FILL PLACED IN ACCORDANCE WITH THE RECOMMENDATIONS PRESENTED IN SECTION 11.2.3 OF THIS REPORT

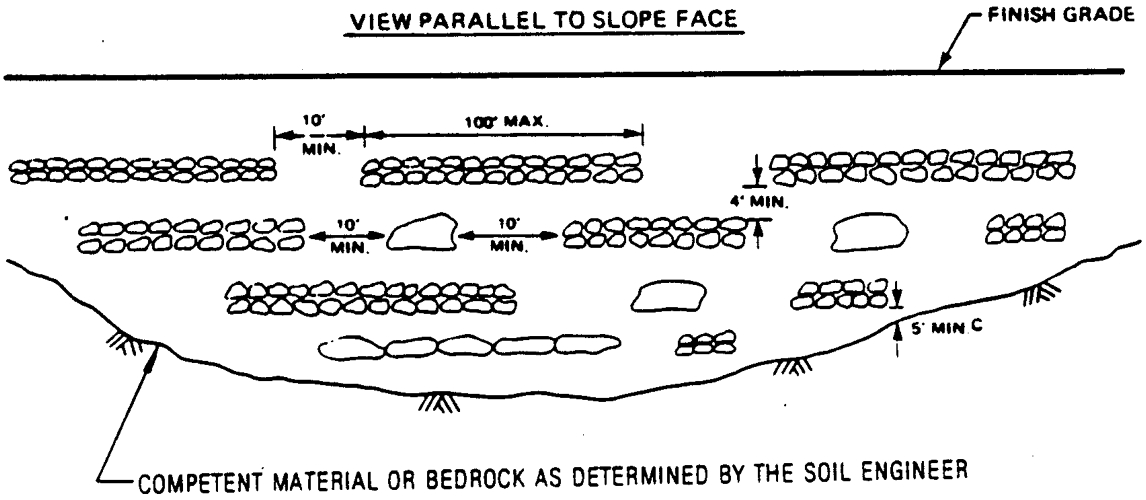
ZONE B ......"SOIL-ROCK" AND/OR "ROCK" FILL PLACED IN ACCORDANCE WITH THE RECOMMENDATIONS PRESENTED IN SECTION 11.2.3 OF THIS REPORT

* 5′ OR 1′ BELOW DEEPEST UTILITY, WHICHEVER IS GREATER

TYPICAL OVERSIZE ROCK DISPOSAL – “SOIL-ROCK” FILL



NOTE:
ORIENTATION OF WINDROWS MAY VARY BUT SHALL BE AS RECOMMENDED BY SOIL ENGINEER.



NOTES:

A. ONE EQUIPMENT WIDTH OR A MINIMUM OF 15 FEET.

B. HEIGHT AND WIDTH MAY VARY DEPENDING ON ROCK SIZE AND TYPE OF EQUIPMENT.

C. IF APPROVED BY THE SOIL ENGINEER, WINDROWS MAY BE PLACED DIRECTLY ON COMPETENT MATERIALS OR BEDROCK PROVIDING ADEQUATE SPACE IS AVAILABLE FOR COMPACTION.

D. VOIDS IN WINDROW TO BE FILLED BY FLOODING GRANULAR SOIL INTO PLACE. GRANULAR SOIL SHALL MEAN ANY SOIL WHICH HAS A UNIFIED SOIL CLASSIFICATION SYSTEM (UBC 29-1) DESIGNATION OF SM, SP, SW, GM, GP, OR GW.

E. AFTER FILL BETWEEN WINDROWS IS PLACED AND COMPACTED WITH THE LIFT OF FILL COVERING WINDROW, WINDROW SHALL BE PROOF-ROLLED WITH D-9 DOZER OR EQUIVALENT.

F. OVERSIZED ROCK IS DEFINED AS LARGER THAN 12" IN SIZE.