

APPENDIX D:
GEOTECHNICAL ENGINEERING REPORT



October 17, 2022
W.O. 7783

RW
1111 Bayside Drive, #150
Corona Del Mar, California 92625

Attention: Mr. Chandler Clark

Subject: Geotechnical Engineering Report, Proposed Warehouse Buildings, APN 0254-041-04, 2245 West Valley Boulevard, Colton, California

As requested, GeoSoils Consultants, Inc. (GSC) has prepared a geotechnical engineering report on the subject property. The purpose of this investigation is to provide geotechnical engineering recommendations for site grading and foundations. The report presents the results of our research, subsurface exploration, laboratory testing, site reconnaissance, and provides geotechnical engineering recommendations for site development. Development of the site is considered feasible from a geotechnical engineering prospective, provided the recommendations presented herein are incorporated into the design and implemented during site development.

We appreciate this opportunity to be of service to you. If you have any questions regarding this report, or if we may be of further assistance to you, please do not hesitate to contact us.

Very truly yours,

GEOSOILS CONSULTANTS, INC.

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cc: (1) Addressee

MDN 23184

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cc: (1) Addressee

1.0 INTRODUCTION

The purpose of this investigation is to determine the geotechnical conditions on the site and their impact on proposed development, and to provide geotechnical engineering data and recommendations to aid in development of the subject site. The following sections provide a summary of the geotechnical engineering conditions on the site, and recommendations for site grading and stabilization, fill placement, and foundations.

This report has been prepared in accordance with generally accepted geotechnical engineering practices in the City of Colton at the time it was prepared. The report presents a brief description of the site, the geotechnical engineering characteristics of the area, the seismicity of the area, an engineering analysis of the site characteristics, conclusions, and recommendations to develop the site.

Opinions presented in this report are based on an inspection of the site, a review of the regional geologic maps and seismic hazard reports, site exploration, laboratory testing and our general knowledge of the geologic and soils engineering conditions in the site area. The opinions presented have been arrived at through the exercise of the generally understood standard of care for our profession and standard of engineering practice for the City of Colton, as we understand it.

1.1 Scope of Services

Our scope of services included the following:

- Site reconnaissance.
- Review of regional geologic maps and seismic hazard reports.
- Excavated, sampled, and logged four (4) hollow stem auger borings at the locations shown on Plate 1, Site Plan.
- Infiltration testing in additional two (2) hollow stem borings.
- Laboratory testing.
- Review of the grading plans.
- Engineering analyses.
- Preparation of this report.

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1.2 Limitations

The findings and recommendations of this report were prepared in accordance with generally accepted professional geotechnical engineering principles and practice for the City of Colton at this time. We make no other warranty, either express or implied. The conclusions and recommendations contained in this report are based on-site conditions disclosed in our site inspection. However, soil/rock conditions can vary significantly between borings; therefore, further refinements of our recommendations contained herein may be necessary due to changes in the building plans or what is encountered during site grading.

The recommendations provided in this report are applicable for preliminary development planning for the referenced site provided that surface water will be kept from infiltrating into the subgrade adjacent to the foundation systems. This may include, but not be limited to rainwater, roof water, landscape water and/or leaky plumbing. The site will be fine graded at the completion of construction to include positive drainage away from the structures and roof water will be collected via gutters, downspouts, and transported to the street in buried drainpipes. Property buyers should be cautioned against constructing open draining planters adjacent to the structure or obstructing the yard drainage in any way.

Since our investigation was based on the site conditions observed and engineering analyses, the conclusions and recommendations contained herein are professional opinions. Further, these opinions have been derived in accordance with standard engineering practices, and no warranty is expressed or implied.

1.3 Site Description

The subject site is located within the City of Colton, 2245 West Valley Boulevard, California (Figure 1). The site is currently occupied by a 198,640 SF warehouse, and parking lots, on a previously graded lot. Existing site conditions are shown on Plate

1. The site is on the north side of W. Valley Boulevard, approximately 0.1 miles east of Riverside Ave.

1.4 Proposed Development

Proposed development of the site will consist of razing existing structures and construction of four (4) 50,000± square foot warehouses.

2.0 FIELD EXPLORATION

Four (4) hollow stem auger borings to the maximum depth of 51 feet, were excavated on the site at the locations shown on Plate 1. Two (2) additional borings were excavated to a maximum depth of 15 feet and were used for Deep Percolation Testing. Soil samples were obtained with a California ring sampler. The hollow stem auger borings used the standard 140 lb. hammer with a 30-inch drop.

A representative from our firm continuously observed the borings, logged the subsurface conditions, and collected representative soil samples. All samples were stored in watertight containers and later transported to our laboratory for further visual examination and testing, as deemed necessary. After the borings were completed, the borings were backfilled with soil cuttings.

The enclosed Boring Logs (Plates B-1 to B-4) describes the vertical sequence of soils and materials encountered in the borings, based primarily on our field classifications and supported by our subsequent laboratory examination and testing.

2.1 Infiltration Testing

The infiltration testing consisted of excavating two, 8-inch diameter borings to a maximum depth of 15 feet, with the infiltration zone from 10 to 15 feet (Plates B-5 to B-6). After drilling the borings and installing the pipe, the 2-inch diameter pipe was then surrounded by well-graded sand and capped with bentonite at 9 feet below existing grade. The pipe was filled with water and pre-soaked overnight. The test results and conclusions can be seen below.

No groundwater was encountered during the excavations. Historical high groundwater map from the “Contour map showing minimum depth to ground water, San Bernardino valley and vicinity, California, 1973-1983” report notes the groundwater to be approximately 150-feet below the ground surface.

Reduction factors applied to the raw percolation rate are as follows:

REDUCTION FACTORS	
Factor Category: Suitability Assessment	S _A = 1.75
Factor Category: Design	S _B = 1.75

Combined Safety Factor, S_{TOT} = S_A X S_B = 3.063

Design Percolation Rate = Measured Percolation Rate/S_{TOT}

The borings were presoaked prior to the infiltration testing. The results are included below and shown on Plates P-1 and P-2.

PERCOLATION TEST RESULTS			
Test Location	Average Field Rate (inch/hour)	Reduction* Factor	Design Rate (inch/hour)
B-5	3.48	3.063	1.14
B-6	2.73	3.063	0.89
*Reduction factor was determined by County of San Bernardino Guidelines for Infiltration Rate Evaluation Protocol and Factor of Safety Recommendations, Worksheet H, dated May 19, 2011.			

As seen in the table above, the adjusted infiltration rate can be assumed at 0.89 inch per hour. The design infiltration rate exceeds the minimum required rate of 0.3 inch per hour.

3.0 LABORATORY TESTING

3.1 Soil Classification

Soil materials encountered within the property were classified and described in accordance with the Unified Soil Classification System and in general accordance with the current version of Test Method ASTM D 2488. The assigned group symbols are presented in the exploration logs, Appendix A.

3.2 In Situ Moisture Content and Dry Unit Weight

In-place moisture content and dry unit weight of selected, relatively undisturbed ring soil samples were determined in accordance with the current version of the Test Method ASTM D 2435 and Test Method ASTM D2216, respectively. Once the dry unit weights had been determined, in-place densities of underlying soil profile were estimated. In those cases where ring samples were obtained, the moisture content and dry unit weights are presented on Boring Logs, Appendix A.

3.3 Expansive Soil

Expansion index testing was performed on selected bulk samples of the on-site soils in accordance with the current version of Test Method ASTM D4829-07. The test results are presented in Plate EI-1. Additional testing will be performed at the completion of grading. The test results indicate an expansion index of 0 (very low range).

3.4 Compaction Tests

One compaction test was performed to determine moisture density relationships of the typical surficial soils encountered on the site. The laboratory standard used was in accordance with ASTM Test Designation D-1557-12. The results are presented below in Table 1 and in Plate MDD-1.

TABLE 1 COMPACTION TEST RESULTS			
Sample	Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
B-1 @ 0-5'	Brown slightly silty, very fine to fine SAND	116.0	9.5

3.5 Sulfate Tests

Samples of the near surface soil were sent to an independent outside laboratory to determine the sulfate content of the soil. The results are included in Appendix B.

3.6 Consolidation Tests

Five (5) consolidation tests were performed on selected ring samples. The samples were inundated at an approximate load of one ton per square foot to monitor the hydro consolidation. Loads were applied to the samples in several increments in geometric progression and resulting deformations were recorded at selected time intervals. Results of the consolidation tests are presented on Plates C-1 to C-5.

4.0 EARTH MATERIALS

Fill

Fill was not encountered in any of the borings but may still be on site in the form of utility trench backfill, etc. If fill is encountered during site grading, it must be removed to competent alluvium.

Alluvium (Qs/Qa)

Alluvium underlies the site which consists of gray brown to medium brown, sandy SILT to gravelly SAND that is moderately moist to moist. The alluvium has a dense to very dense consistency which makes it not susceptible to seismic settlement.

4.1 Groundwater

Groundwater was not encountered to the maximum depth of approximately 51 feet below existing ground surface during our subsurface evaluation. Historic high groundwater is approximately 150 feet below current grade per the “Contour map showing minimum depth to ground water, San Bernardino valley and vicinity, California, 1973-1983” report by Scott E. Carson and Jonathan C Matti (1986).

5.0 FAULTING AND SEISMICITY

The project site is not located within an Alquist-Priolo Earthquake Fault Zone and there are no active faults on or adjacent to the property (Figure 3). Although there are no faults on or adjacent to the property, there are faults near the site that can cause moderate to intense

ground shaking during the lifetime of the proposed development. The closest active fault is the San Jacinto Fault approximately three miles away. Therefore, earthquake resistant design is recommended.

Earthquake Characterization: Earthquakes are characterized by magnitude, which is a quantitative measure of the earthquake strength, based on strain energy released during a seismic event. The magnitude of an earthquake is constant for any given site and is independent of the site in question.

Earthquake Intensity: The intensity of an earthquake at a random site is not constant and is subject to variations. The intensity is an indirect measurement of ground motion at a particular site and is affected by the earthquake magnitude, the distance between the site and the hypocenter (the location on the fault at depth where the energy is released), and the geologic conditions between the site and the hypocenter. Intensity, which is often measured by the Mercalli scale, generally increases with increasing magnitude and decreases with increasing distance from the hypocenter. Topography may also affect the intensity of an earthquake from one site to another. Topographic effects such as steep sided ridges or slopes may result in a higher intensity than sites located in relatively flat-lying areas.

5.1 2019 California Building Code (CBC) Seismic Design Criteria

The 2019 CBC (California Building Code) seismic coefficient criteria are provided in Table 2 for structural design consideration. Under the Earthquake Design Regulations of Chapter 16, Section 1613 of the CBC 2019, the following coefficients apply for the proposed structures at the site¹. Site Class D should be used for the site. The following seismic data is presented for preliminary design purposes. Ground motion parameters based on the Mapped Risk-Targeted Maximum Considered Earthquake (MCE_r) were determined and adhere to requirements discussed in ASCE 7-16 referenced by the 2019 California Building Code. The parameters include 5% critical damping for 0.2- and 1.0-second time periods. A summary of parameters is provided in the table below for a Site Class D designation. These values may only be used when the value of the

seismic response coefficient C_s satisfies equations 12.8-2, 12.8-3, or 12.8-4 of the ASCE 7-16 Standard.

TABLE 2 SEISMIC PARAMETERS	
Description	Value
Mapped Response (0.2 second), S_s	1.885
Mapped Spectral Response (1.0 second), S_1	0.742
Short Period Site Coefficient, F_a	1.2
1-second Period Site Coefficient, F_v	1.7
Adjusted Maximum Considered Earthquake Spectral Response (0.2 second), S_{MS}	2.262
Adjusted Maximum Considered Earthquake Spectral Response (1.0 second), S_{M1}	1.261
5-percent Damped Design Spectral Response (0.2 second), S_{DS}	1.508
5-percent Damped Design Spectral Response (1.0 second), S_{D1}	0.841
Maximum Considered Earthquake Geometric Mean Peak Ground Acceleration, PGA_M	0.955
Site Coordinates: Latitude: 34.071445°, Longitude: -117.3669725°	

Conformance to the above criteria for seismic excitation does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a maximum level earthquake occurs. The primary goal of seismic design is to protect life and not to avoid all damage, since such design may be economically prohibitive. Following a major earthquake, a building may be damaged beyond repair, yet not collapse.

5.2 Secondary Earthquake Effects

Ground shaking produced during an earthquake can result in a number of potentially damaging phenomena classified as secondary earthquake effects. These secondary effects include ground rupture, landslides, seiches and tsunamis, seismically induced settlement, and liquefaction. Descriptions of each of these phenomena and how it could potentially affect the proposed site are described as follows:

5.2.1 Ground Rupture

Ground rupture occurs when movement on a fault breaks the ground surface and usually occurs along pre-existing fault traces where zones of weakness already exist. The State has established Earthquake Fault Zones for the purpose of mitigating the hazard of fault rupture by prohibiting the location of most human occupancy structures across the traces of active faults.

Earthquake fault zones are regulatory zones that encompass surface traces of active faults with a potential for future surface fault rupture. The site is not located within a State established Earthquake Fault Zone and there are no known active faults within the limits of the property (Figure 3); therefore, the ground rupture hazard potential for the site is considered remote.

5.2.2 Landsliding

Landslides are slope failures that occur where the horizontal seismic forces act to induce soil and/or bedrock failures. The most common effect is reactivation or movement on a pre-existing landslide. Typically, existing slides that are stable under static conditions (i.e., factor-of-safety above one) become unstable and move during strong ground shaking. The site is flat and not subject to landslides.

5.2.3 Seiches and Tsunamis

A seiche is the resonant oscillation of a body of water, typically a lake or swimming pool caused by earthquake shaking (waves). The hazard exists where water can be splashed out of the body of water and impact nearby structures. No bodies of constant water are near the site, therefore, the hazards associated with seiches are considered low.

Tsunamis are seismic sea waves generated by undersea earthquakes or landslides. When the ocean floor is offset or tilted during an earthquake, a set of waves are generated similar to the concentric waves caused by an object dropped in water.

Tsunamis can have wavelengths of up to 120 miles and travel as fast as 500 miles per hour across hundreds of miles of deep Ocean. Upon reaching shallow coastal waters, the once two-foot high wave can become up to 50 feet in height causing great devastation to structures within reach. Tsunamis can

generate seiches as well. Due to the distance of the site relative to the ocean, seiches and tsunamis are not considered a hazard to the site.

5.2.4 Liquefaction and Dry Sand Settlement Analysis

Dry sand settlement can occur during moderate and large earthquakes when loose, natural or fill sandy soils are densified and settle, often unevenly across a site. In order for dry sand settlement to occur, the following four factors are required: 1) Relatively dry soil or soil situated above the groundwater table; 2) undrained loading (strong ground shaking), such as by earthquake; 3) contractive soil response during shear loading, which is often the case for a soil which is initially in a loose or uncompacted state; and 4) susceptible soil type; such as clean, uniformly graded sands. As discussed above, competent alluvium underlies the compacted fill which will be used to support the structures. Therefore, seismic settlement is not considered to be a hazard to the site.

Liquefaction is a soil softening dynamic response, by which an increase in the excess pore water pressure results in partial to full loss of soil shear strength and post-liquefaction dissipation of this pore water pressure results in ground settlement shortly after the earthquake. In order for liquefaction to occur, the following four factors are required: 1) saturated soil or soil situated below the groundwater table; 2) undrained loading (strong ground shaking), such as by earthquake; 3) contractive soil response during shear loading, which is often the case for a soil which is initially in a loose or uncompacted state; and 4) susceptible soil type; such as clean, uniformly graded sands, non-plastic silts, or gravels.

Historic high groundwater is mapped at a depth of approximately 150 feet below existing grade as discussed in section 4.4 above. The alluvial soils encountered in our borings were generally found to be dense to very dense and generally not susceptible to liquefaction.

6.0 CONCLUSIONS

The development of the subject site is considered feasible from a geologic and geotechnical engineering viewpoint, provided that the recommendations presented in this report are followed during grading.

7.0 RECOMMENDATIONS

7.1 Removals

Removals shall extend a minimum of 5 (five) feet below existing ground surface or proposed grades in building areas, whichever is lower in elevation, and shall include all existing fill and alluvium. Removals shall extend a minimum of 5 feet beyond the building footprint or equal to the depth of removal, whichever is greater. Deeper removals may be required if soft or dry soil conditions are observed during grading. For the parking lot(s), removals shall extend a minimum of 2 (two) feet below existing ground surface or proposed grades, whether is lower in elevation, and shall include all existing fill and alluvium. Recommendations for removals may be modified if the current site development plan changes (See Plate 1). Preparation of areas to receive fill and fill placement shall be performed as discussed under “*Grading section*”.

7.2 Foundation Recommendations

The following recommendations are provided for preliminary design purposes and the final expansion index should be determined following grading. In our opinion, conventional footings with slab-on-grade should be used to support the proposed structures.

Foundations should be designed for very low-expansive soil conditions. The proposed improvements should be founded into compacted fill. Under no circumstances should foundations be cast atop loose, soft, or slough, debris, existing artificial fill, topsoil, or surfaces covered by standing water. Prior to placing concrete in a foundation excavation, an inspection should be made by our representative to ensure that the foundation’s subgrade is free of loose and disturbed soils and is embedded in the

recommended material. We offer the following site-specific recommendations and comments for purposes of foundation design and construction. The on-site materials have a negligible sulfate content (see Plate L-1).

7.2.1 Footings

The proposed structures may be supported on footings with slab-on-ground. Exterior isolated pad footings may need to be connected to adjacent footings via tie beams at the discretion of the project structural engineer.

Subgrade Preparation

All conventional footings should be constructed on firm, unyielding certified compacted fill. All compacted fill should be compacted to at least 90 percent of the Modified Proctor maximum laboratory density, as determined by ASTM D-1557-12 compaction method. Pre-moistening of all areas to receive concrete is recommended. The moisture content of the subgrade soils should be equal to or slightly greater than optimum moisture and verified by the Geotechnical Engineer to a depth of 18 inches below adjacent grade within 24 hours of concrete placement. Footing's subgrades shall be prepared in accordance with the *Grading* section of this report.

Bearing Capacity

Continuous and isolated buildings footings should have a width of at least 12 and depth of 18 inches. New footings should extend at least 18 inches below exterior grade and, at least 6 inches below the bottom of concrete slabs-on-grade. In areas where removals can not extend beyond the building footprint due to property line, footings should extend at least 30 inches below exterior grade. Exterior isolated pad footings intended for support of roof overhangs such as decks, patio covers and similar construction should be a minimum of 24 inches square and founded at a minimum depth of 18 inches below the lowest adjacent final grade. Footings with at least above minimum dimensions

may be designed for a preliminary allowable bearing pressure of 2,000 pounds per square foot (psf) for dead plus live loads, with a one-third increase allowed when considering additional short-term wind or seismic loading. The allowable bearing value may be increased by 200 pounds per square foot per foot increase in depth or width to a maximum of 4,000 psf. The weight of the footings may be neglected for design purposes. All footings located adjacent to utility lines should be embedded below a 1:1 plane extending up from the bottom edge of the utility trench.

Settlement

The footings should be designed based on a very low-expansive soils condition. Thirty-year post-construction differential settlement due to static loads is not expected to exceed about 0.5-inch over 30 feet span for the proposed improvements supported on footings, provided that the foundations are designed and constructed as recommended.

The spread footings should be designed to accommodate a seismic total and differential settlement of approximately 1 and 0.5 inch over a horizontal span of 30 feet, respectively. In our opinion the project structural engineer should determine how the provided static and dynamic settlement estimates be combined for purposes of structural design.

Lateral Capacity

Lateral loads may be resisted by friction between the bottom of the footings and the supporting subgrade, and by passive soil pressure acting against the footings cast neat in foundation excavations or backfilled with properly compacted structural fill. A coefficient of friction of 0.4 may be assumed for design for footings supported on compacted fill. We recommend an equivalent fluid pressure of 250 pounds per cubic foot for passive soil resistance and not to exceed 2,500 pounds per cubic foot, where appropriate. The upper foot of

passive soil resistance should be neglected where soil adjacent to the footing is not covered and protected by a concrete slab or pavement. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third.

General Structural Design

We recommend that foundations be reinforced with a minimum two, No. 4 rebar both top and bottom, to provide structural continuity and to permit spanning of local irregularities.

7.2.2 Foundation General Recommendations

The above parameters are applicable provided structures have gutters and downspouts and positive drainage is maintained away from structures. Therefore, it is important that information regarding drainage and site maintenance be passed on to future owners.

The above recommendations assume, and GeoSoils Consultants, Inc. strongly recommends, that surface water will be kept from infiltrating into the subgrade adjacent to the building foundation system. This may include, but not be limited to rainwater, roof water, landscape water and/or leaky plumbing. The lots are to be fine graded at the completion of construction to include positive drainage away from the structure and roof water will be collected via gutters, downspouts, and transported to the street in buried drainpipes. Buyers should be cautioned against constructing open draining planters adjacent to the houses or obstructing the yard drainage in any way.

- Utility trenches beneath the slabs should be backfilled with compacted native soil materials, free of rocks.

- Standard City of Colton structural setback guidelines are applicable, except where superseded by specific recommendations by the Project Geologist and Geotechnical Engineer.

7.3 Interior Slabs

General

Conventional interior slabs should be at least 4 inches thick and reinforced with a minimum No. 4 bars at 16 inches on center each way. In moisture sensitive areas, a minimum 10 mil Visqueen should be sandwiched between two, one-inch sand layers.

7.4 Retaining Walls

Proposed truck loading docks may require retaining walls. All grading and construction phases associated with retaining wall construction, including backcut excavations, footing trenches, installation of the subdrainage systems, and placement of backfill should be observed and tested by a representative of the project geotechnical consultant.

Allowable Bearing Values

Proposed retaining walls should be supported on spread footings using the design criteria recommended previously for building footings.

Active and At-Rest Earth Pressures

The equivalent fluid pressures recommended are based on the assumption of a uniform backfill and no build-up of hydrostatic pressure behind the wall. To prevent the build-up of lateral soil pressures in excess of the recommended design pressures, over compaction of the fill behind the wall should be avoided. This can be accomplished by placement of the backfill above a 45-degree plane projected upward from the base of the wall, in lifts not exceeding eight inches in loose depth, and compacting with a hand-operated or small, self-propelled vibrating plates.

All recommendations for active lateral earth pressures contained herein assume that the anticipated retaining structures are in tight contact with the fill soil or shoring system that they are supposed to support. Cantilever (yielding) retaining wall that are free-standing to the point that they can yield at the top at least 0.1 percent of the height of the wall, should be designed for active pressure. Retaining walls that are not free to deflect or rotate (such as any basement retaining walls) or are expected to yield at the top of the wall to less than 1/1000 the height of the wall must be designed for at rest pressure.

On-Site Soils Used for Backfill

The on-site earth materials are considered to have a very low expansion potential and are suitable for use as backfill. If these materials are used as backfill, active and at rest earth pressures equivalent to fluids having density of 30 and 45 pcf, respectively, should be used for design. The above values are for retaining walls that have been supplied with a proper subdrain.

Earthquake Loads on Retaining Walls

The seismic load increment is determined by the following equations for walls backfilled with engineering fill:

Cantilever (yielding) walls with level backfill:

$$\Delta P_{ae} = 3/8\gamma H^2 k_h$$

H = Height of wall

$$K_h = 1/3PGA_m = 1/3(0.955) = 0.318$$

$\gamma = 120$ pcf

$$\Delta P_{ae} = 3/8*(120 \text{ pcf})(0.318)H^2 = 14.33H^2$$

The combined static and seismic load can be expressed using Seed and Whitman's (1970)² notation where Pa is the static load and ΔP_{ae} is the seismic load increment:

$$P_{ae} = P_a + \Delta P_{ae}$$

The resultant force (static and seismic loading) acts 1/3H from the base of the wall (Lew et al. 2010)³. Considering the difference between the on-site soil and imported engineering fill for backfill is not significant we propose the same seismic design values for retaining walls backfilled with on-site soils.

Surcharge Load

Wherever walls will be subjected to surcharge loads within a 45 degree projection upward from the wall bottom, such as from adjacent foundations, vehicle loads, or construction, the walls should be designed for an additional uniform lateral pressure equal to one-half of the surcharge loads for yielding walls and one-third of the surcharge load for none-yielding walls. A more accurate surcharge load can be provided, if requested.

Subdrainage

Perforated pipe and gravel subdrains should be installed behind all retaining walls that retains more than 2 feet to prevent entrapment of water in the backfill. Perforated pipe should consist of 4-inch-minimum diameter PVC Schedule 40, or SDR-35, with the perforations laid down. The pipe should be encased in a 1-foot-wide column of ¾-inch to 1½-inch open-graded gravel.

We note that any wall should be constructed with weep holes near the bottom, on five-foot centers or with perforated drainpipe in a gravel envelope at the bottom and behind the wall.

Waterproofing

The portions of retaining walls supporting backfill should be coated with an approved waterproofing compound or covered with a similar material to inhibit infiltration of moisture through the walls.

Wall Backfill

Where the onsite soils materials or imported sand (with a Sand Equivalent of 30 or greater) are used as backfill behind the proposed retaining walls, the backfill materials should be placed in approximately 6- to 8-inch-thick maximum lifts, watered as necessary to achieve above optimum moisture conditions, and then mechanically compacted in place to a minimum relative compaction of 90 percent. Flooding or jetting of the backfill materials should be avoided. A representative of the project geotechnical consultant should observe the backfill procedures and test the wall backfill to verify adequate compaction.

To reduce the potential for the direct infiltration of surface water into the backfill, imported sand, gravel, or rock backfill should be capped with at least 12 inches of on-site soil. Filter fabric such as Mirafi 140N or equivalent, should be placed between the soil and the imported gravel or rock to prevent fines from penetrating into the backfill.

7.5 Grading

The grading will involve the removing and recompacting of existing near surface material and any material disturbed during the demolition process. We offer the following recommendations and construction considerations concerning earthwork grading at the site.

7.5.1 General

Monitoring: We recommend that all earthwork (i.e., clearing, site preparation, fill placement, etc.) should be conducted with engineering control under observation and testing by the Geotechnical Engineer and in accordance with the requirements within the *Grading* section of this report.

Job Site Safety: At all times, safety should have precedence over production work. If an unsafe job condition is observed, it should be brought to the attention of the grading contractor or the developer's representative. Once this

condition is noted, it should be corrected as soon as possible, or work related to the unsafe condition should be terminated.

The contractor for the project should realize that services provided by GSC do not include supervision or direction of the actual work performed by the contractor, his employees, or agents. GSC will use accepted geotechnical engineering and testing procedures; however, our testing and observations will not relieve the contractor of his primary responsibility to produce a completed project conforming to the project plans and specifications. Furthermore, our firm will not be responsible for job or site safety on this project, as this is the responsibility of the contractor.

7.5.2 Site Preparation

Existing Structure Location: The General Contractor should locate all surface and subsurface structures on the site or on the approved grading plan prior to preparing the ground.

Existing Structure Removal: Any underground structures (e.g., septic tanks, wells, pipelines, foundations, utilities, etc.) that have not been located prior to grading should be removed or treated in a manner recommended by the Geotechnical Engineer.

Clearing and Stripping: The construction areas should be cleared and stripped of all vegetation, trees, bushes, sod, topsoil, artificial fill, debris, asphalt, concrete, and other deleterious material prior to fill placement.

Removals: Please refer to the *Removals* section of this report for specific recommendations for removals.

Subgrade Preparation: We recommend that the subgrade for those areas receiving any fill be prepared by scarifying the upper 12 inches and moisture conditioning, as required to obtain at least optimum moisture, but not greater

than 120 percent of optimum. The scarified areas shall be compacted to at least 90 percent of the maximum laboratory density, as determined by ASTM D-1557-12 compaction method. All areas to receive fill should be observed by the Geotechnical Engineer prior to fill placement.

Subgrade Verification and Compaction Testing: Regardless of material or location, all fill material should be placed over properly compacted subgrades in accordance with this section. The condition of all subgrades shall be verified by the Geotechnical Engineer before fill placement or earthwork grading begins. Earthwork monitoring and field density testing shall be performed during grading to provide a basis for opinions concerning the degree of soil compaction attained. The Contractor should be responsible for notifying the Geotechnical Engineer when such areas are ready for inspection. Inspection of the subgrade may also be required by the controlling governmental agency within the respective jurisdictions. Density tests should also be made on the prepared subgrade to receive fill, unless the areas are underlain by dense alluvium, as required by the Geotechnical Engineer.

7.5.3 Fill Placement

Laboratory Testing: Representative samples of materials to be utilized as compacted fill should be analyzed in a laboratory to determine their physical properties. If any material other than that previously tested is encountered during grading, the appropriate analysis of this material should be conducted.

On-Site Fill Material: The on-site soils, in our opinion, are adequate for re-use in controlled fills provided the soils do not contain any organic matter, debris, and that over-sized rocks are buried in accordance with the recommendations under *Rock Fragments*.

Rock Fragments: The alluvium on the site should be free of oversized rocks. Rocks greater than 6 inches in diameter should be taken off site or placed in accordance with the recommendations of the Geotechnical Engineer. Rocks shall not be placed in concentrated pockets, shall be surrounded with fine grained material, and the distribution of the rocks shall be supervised by the Geotechnical Engineer. A sufficient amount of fine-grained material shall be placed around the rocks to prevent nesting and to fill all void space. An adequate amount of water will be required to force fines into any open voids.

Fill Placement: Approved on-site material shall be evenly placed, watered, processed, and compacted in controlled horizontal layers not exceeding eight inches in loose thickness, and each layer should be thoroughly compacted with approved equipment. The fill should be placed and compacted in horizontal layers, unless otherwise recommended by the Geotechnical Engineer.

Compaction Criteria - Shallow Fills: For fills less than 40 feet in vertical thickness, each layer shall be compacted to at least 90 percent of the maximum laboratory density for material used as determined by ASTM D-1557-12. The field density shall be determined by the ASTM D-1556-07 method or equivalent. Where moisture content of the fill or density testing yields compaction results less than 90 percent, additional compaction effort and/or moisture conditioning, as necessary, shall be performed, until the fill material is in accordance with the requirements of the Geotechnical Engineer.

Fill Material - Moisture Content: All fill material placed must be moisture conditioned, as required to obtain at least optimum moisture, but not greater than 120 percent. If excessive moisture in the fill results in failing results or an unacceptable “pumping” condition, then the fill should be allowed to dry until the moisture content is within the necessary range to meet the required compaction requirements or reworked until acceptable conditions are obtained.

Density Testing Intervals: In general, density tests should be conducted at minimum intervals of 2 feet of fill height or every 500 to 1,000 cubic yards. Due to the variability that can occur in fill placement and different fill material characteristics, a higher number of density tests may be warranted to verify that the required compaction is being achieved.

Grading Control: Earthwork monitoring and field density testing shall be performed by the Geotechnical Engineer during grading to provide a basis for opinions concerning the degree of soil compaction attained. The Contractor should receive a copy of the Geotechnical Engineer's *Daily Field Engineering Report* which will indicate the results of field density tests for that day. Where failing tests occur or other field problems arise, the Contractor shall be notified of such conditions by written communication from the Geotechnical Engineer in the form of a conference memorandum, to avoid any misunderstanding arising from oral communication.

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

7.5.4 Construction Considerations

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

Compaction Equipment: It is also the Contractor's responsibility to have suitable and sufficient compaction equipment on the project site to handle the amount of fill being placed and the type of fill material to be compacted. If necessary, excavation equipment should be shut down to permit completion of compaction in accordance with the recommendations contained herein.

Sufficient watering devices/equipment should also be provided by the Contractor to achieve optimum moisture content in the fill material.

Final Grading Considerations: Care should 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.

7.5.5 Earthwork Adjustment Factors

The following table presents shrinkage factors as based on laboratory testing of the alluvium.

TABLE 3 EARTHWORK ADJUSTMENT FACTORS	
Material Type	Adjustment Factor
Alluvium	3 to 8 (bulking)

7.5.6 Temporary Excavation

Where the necessary space is available, temporary unsurcharged embankments may be sloped back without shoring. The slope should not be cut steeper than the following gradient:

TABLE 4 TEMPORARY EXCAVATION SLOPE	
Height	Temporary Gradient (Horizontal:Vertical)
0 - 5'	Near Vertical
above 5'	1:1

In areas where soils with little or no binder are encountered, shoring or flatter excavation slopes shall be made. These recommended temporary excavation slopes do not preclude local raveling or sloughing.

All applicable requirements of the California Construction and General Industry Safety Orders, the Occupational Safety and Health Act, and the Construction Safety Act should be met.

Where sloped embankments are used, the top of the slope should be barricaded to prevent equipment and heavy storage loads within five feet of the top of the slope. If the temporary construction embankments are to be maintained for long periods, berms should be constructed along the top of the slope to prevent runoff water from eroding the slope faces. The soils exposed in the temporary backcut slopes during excavation should be observed by our personnel so that modifications of the slopes can be made if variations in the soil conditions occur. The temporary excavation slopes should be supported within three weeks.

Slot cuts may be required for grading adjacent to property lines. It is recommended the grading be performed using ABC slot cutting. Each individual slot cut should not exceed 8 feet in horizontal length of 5 feet in vertical height. No adjoining slot cuts should be opened at the same time. All vertical cuts should be maintained in a moist, but not saturated condition.

7.5.7 Excavation Observation

All footing and other excavations should be observed by an Engineering Geologist or Geotechnical Engineering prior to placement of any steel to verify that the proper foundation material has been encountered. The City Inspector should also observe the excavation.

7.5.8 Utility Trenching and Backfill

Utility Trenching: Open excavations and excavations that are shored shall conform to all applicable Federal, State, and local regulations.

Backfill Placement: Approved on-site or imported fill material shall be evenly placed, watered, processed, and compacted in controlled horizontal layers not exceeding eight inches in loose thickness, and each layer should be thoroughly compacted with approved equipment. All fill material should be moisture

conditioned, as required to obtain at least optimum moisture, but not greater than 120 percent of optimum moisture content. The fill should be placed and compacted on a horizontal plane, unless otherwise recommended by the Geotechnical Engineer.

As an alternative to on-site or imported fill material, for shallow trenches where pipe or utility lines may be damaged by mechanical compaction equipment, such as under building floor slabs, imported clean sand having a sand equivalent (SE) value of 30 or greater may be utilized. The sand backfill materials should be watered to achieve near optimum moisture conditions and then tamped into place. No specific relative compaction will be required; however, observation, probing, and if deemed necessary, testing should be performed by a representative of the project geotechnical consultant to verify an adequate degree of compaction.

Backfill Compaction Criteria: Each layer of utility trench backfill shall be compacted to at least 90 percent of the maximum laboratory density determined by ASTM D-1557-12. The field density shall be determined by the ASTM D-1556-07 method or equivalent. Where moisture content of the fill or density testing yields compaction results less than 90 percent, additional compaction effort and/or moisture conditioning, as necessary, shall be performed, until the compaction criteria is reached.

Exterior Trenches Adjacent to Footings: Exterior trenches, paralleling a footing and extending below a 1H:1V plane projected from the outside bottom edge of the footing, should be compacted to 90 percent of the laboratory standard. Sand backfill, unless it is similar to the in-place fill, should not be allowed in the trench backfill areas. Density testing, along with probing, should be accomplished to verify the desired results.

Pipe Bedding: We recommend that a minimum of 6 inches of bedding material should be placed in the bottom of the utility trench. All bedding materials shall extend at least 4 inches above the top of utilities which require protection during subsequent trench backfilling. All trenches shall be wide enough to allow for compaction around the haunches of the pipe.

Groundwater Migration: Backfilled utility trenches may act as French drains to some extent, and considerable groundwater flow along utility bedding and backfill should be expected. Wherever buried utilities, or structures which they may intersect, could be adversely affected by such drainage, provisions shall be made to collect groundwater migrating along the trench lines. These situations include where buried utilities enter buildings, particularly where they enter below grade mechanical rooms, and where buried utilities enter junction boxes or switching stations that are intended to remain dry. Mitigation measures include, but are not limited to, placement of perforated drain pipes below and continuous with bedding materials, and placement of seepage barriers such as lean mix concrete or controlled density fill (CDF).

8.0 CLOSURE

We appreciate this opportunity to be of continued service to you. If you have any questions regarding the content of this report or any other aspects of the project, please do not hesitate to contact us.

October 17, 2022
W.O. 7783

REFERENCES

Dibblee, T.W. and Minch, J.A., 2004, Geologic Map of the San Bernardino North/north ½ of San Bernardino South quadrangles, San Bernardino and Riverside County, California

¹ California Building Code (CBC), 2019, California Code of Regulations, Title 24, Part 2, Volume I and II.

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APPENDIX A

FIELD EXPLORATION and INFILTRATION RESULTS

² Seed, H.B. and Whitman, R.V.. (1970). "Design of Earth Retaining Structures for Dynamic Loads." ASCE Specialty Conference, Lateral Stresses in the Ground and Design of Earth Retaining Structures. Cornell University, Ithaca, NY., 103-147.

³ Lew, M., Sitar, N. and Al Atik, L. (2010). "Seismic Earth Pressures: Fact or Fiction." Proceedings of the Earth Retention Conference 3, Geo-Institute of ASCE, Bellevue, WA.

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APPENDIX B
LABORATORY TEST RESULTS

MDN 23184

**GEOTECHNICAL ENGINEERING REPORT
PROPOSED WAREHOUSE BUILDINGS,**

APN 0254-041-04
2245 West Valley Boulevard
Colton, California

for

RW

October 17, 2022

W.O. 7783

MDN 23184