Trenton Hansen, Ph.D.
Superintendent
4850 Pedley Road, Jurupa Valley, CA 92509 T (951) 360-4100

Date: September 21, 2022

Re: 22-23-03MO – JUSD Storage Facility – Addendum No. 3

TO ALL BIDDERS:

The following changes, omissions, and/or additions to the Bid Documents and/or Project Manual and/or Drawings shall apply to proposals made for and to the execution of the various parts of the work affected thereby, and all other conditions shall remain the same. All parties of interest shall take careful note of the addendum so that the proper allowances may be made in strict accordance with the Addendum.

Bidder shall acknowledge receipt of this addendum in the space provided on the Bid Form. Failure to do so may subject Bidder disqualification.

In case of conflict between Drawings, bid documents and this addendum, this addendum shall govern.

ITEM #1 Q: What is the start date for the project?
A: The anticipated start date is October 31, 2022.

ITEM #2 Q: The Foundation Plan S1-1.1 indicates a grade beam callout of 8/SD1.11. This detail does not exist. Please provide.
A; See attached revised S1-1.1

ITEM #3 Q: Detail 1 on SD1.1 notes "See Plan" for size of footings. However, the Foundation plan S1-1.1 does not show any dimensions. Please provide dimensions.
A: See attached revised S1-1.1

Item #4 Q: Is a soils report available for the project?
A: Yes, please see attached.

Item # 5 Q: Some exterior wall details indicate Densglass between the metal framing and the metal wall siding, but some details don't. What is the intent?
A: No Densglass, this is a pre-fabricated metal building, see attached revised sheet A1-8.1

Item # 6 Q: RE: Drawing C.-2.1
Per site walk all (E) Relocatable classrooms and (E) containers are to be relocated by district.
A: Correct, all existing relocatable & containers are to be relocated by the District

Item #7 Q: Re: A1-7.1/ A1-8.1 17/18
Please confirm HT CP 5/8" 6WP is plywood & Gypsum to be finished and painted.
A: High Impact GWB need no paint.
B. Height as indicated on bldg. section A1-7.1, See attached sheet
C. On detail 18/A1-8.1 height of plywood is indicated, See attached sheet

Item #8 Q: Re: Add #2 Fire Suppression HVAC
Will both F.S/ HVAC be a Bid, or will AIA provide drawings?
A: Will be bid

Item #9 Q: Re: Insulated Metal Roof – Drawing A1-5.1
General Note +4 Provide detail
A: Insulation must be provided by the metal building manufacturer

Item #10 Q: Re: Fire alarm/ security
Please provide current F/A and security contractor the services your facility.
A: Vision Security Systems, Moreno Valley, CA

Item #11 Q: Will bid be extended?
A: No extension to the bid deadline.

GEOTECHNICAL EVALUATION
PROPOSED JURUPA UNIFIED SCHOOL DISTRICT STORAGE PROJECT
JURUPA VALLEY HIGH SCHOOL
10551 BELLEGRAVE AVENUE
JURUPA VALLEY, RIVERSIDE COUNTY, CALIFORNIA

PREPARED FOR

JURUPA UNIFIED SCHOOL DISTRICT
3612 MISSION INN AVENUE
RIVERSIDE, CALIFORNIA 92501

PREPARED BY

GEOTEK, INC.
1548 NORTH MAPLE STREET
CORONA, CALIFORNIA 92878

PROJECT NO. 3050-CR **FEBRUARY 17, 2022**

GEOTEK

GeoTek, Inc.
1548 North Maple Street, Corona, California 92880
(951) 710-1160 Office (951) 710-1167 Fax www.geotekusa.com

February 17, 2022
Project No. 3050-CR

Jurupa Valley High School
4850 Pedley Road
Jurupa, California 92501

Attention: Mr. Jeffrey Lewis, Director of Purchasing

Subject: **Geotechnical Evaluation**
Proposed Jurupa Unified School District Storage Project
Jurupa Valley High School
10551 Bellegrave Avenue
Jurupa Valley, Riverside County, California

Dear Mr. Lewis:

GeoTek, Inc. (GeoTek) is pleased to provide the results of this Geotechnical Evaluation for the proposed Jurupa Unified School District storage project that will be constructed on the campus of Jurupa Valley High School, located at 10551 Bellegrave Avenue, in the City of Jurupa Valley, Riverside County, California. This report presents a discussion of GeoTek's evaluation and provides preliminary geotechnical recommendations for site preparation, foundation design, infiltration rates and construction of the proposed site improvements.

Based upon review and evaluation, site development appears feasible from a geotechnical viewpoint provided that the recommendations included in this report are incorporated into the design and construction phases of site development.

The opportunity to be of service is sincerely appreciated. If you have any questions, please do not hesitate to contact GeoTek.

Respectfully submitted,
GeoTek, Inc.

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1. PURPOSE AND SCOPE OF SERVICES

The purpose of this study was to evaluate the geotechnical conditions for the proposed Jurupa Unified School District storage project on the Jurupa Valley High School campus, with respect to currently proposed improvements, as outlined in the proposal P-0101422-CR dated January 4, 2022. Services provided for this study included the following:

- Research and review of available geologic and geotechnical data and general information pertinent to the site,
- A site reconnaissance,
- Excavation of two (2) exploratory borings for the geotechnical portion of the evaluation to depths of about 21.5 feet and 51.5 feet below existing grades at the boring locations,
- Collection of soil samples from the test borings,
- Laboratory testing of selected soil samples,
- Review and evaluation of site seismicity, and;
- Compilation of this geotechnical report which presents preliminary recommendations for site development.

The intent of this report is to aid in the evaluation of the site for future proposed development from a geotechnical perspective. The professional opinions and geotechnical information contained in this report may need to be updated based upon review of the final site development plans. These plans should be provided to GeoTek, Inc. for review when available.

2. SITE DESCRIPTION AND PROPOSED DEVELOPMENT

2.1 SITE DESCRIPTION

Jurupa Valley High School is located at 10551 Bellegrave Avenue in the City of Jurupa Valley, Riverside County, California. The high school campus contains numerous permanent buildings and educational facilities, play and athletic fields, parking facilities and hardscaping and landscaping improvements. Access to the school campus is available from Cantu-Galleano Ranch Road, Etiwanda Avenue and Bellegrave Avenue, all paved improved streets located adjacent to the northern, western, and southern boundaries of the campus, respectively. A sports park asphalt concrete access road is located adjacent to the eastern boundary of the campus. The campus has a topographic high in the approximate center of the site sloping downward to the north and south with approximately 30 feet of elevation differential across the campus

The project site is located in the northeast portion of the campus. The project site is situated at approximate 34.003° North Latitude and approximately -117.520 West Longitude. The project site is located in a concrete paved storage yard containing relocatable classroom buildings and storage containers. While a private utility scan indicated no underground utilities within the proposed building area, underground utilities may be present in the project area.

The project site is situated at an elevation of approximately 740 feet above mean sea level (amsl). The project site generally slopes downward to the south/southwest at a gradient of less than two percent. Total site relief across the project site is less than 5 feet. The project site location is presented on Figure 1, Site Location and General Topography Map.

2.2 PROPOSED DEVELOPMENT

Plans are to construct an approximate 7,500 square foot prefabricated metal building to be used for data/file storage. The proposed building is anticipated to be supported by conventional shallow foundations and slab-on-grade floor system. Estimated maximum structural loads are anticipated to be 1,500 plf for continuous foundations and 25 kips for columns loads. As part of the site development, new hardscaping is anticipated to be constructed.

Since the campus has already been graded it is anticipated that minimal cuts and fills will be required for site development, and major slopes and retaining walls are not proposed.

If the site development differs from the information provided in this report, the recommendations should be subject to further review and evaluation by GeoTek. Final site development plans should be reviewed by GeoTek when they become available.

3. FIELD EXPLORATION AND LABORATORY TESTING

3.1 FIELD EXPLORATION

The field exploration for this project was conducted on January 26, 2022. For the geotechnical portion of the investigation, two (2) test borings were excavated with a hollow-stem auger drill rig to depths of 21.5 feet and 51.5 feet below the existing ground surface (concrete) at the boring locations. The approximate boring locations are indicated on Figure 4, Boring Location Map.

The exploration logs show subsurface conditions at the dates and locations indicated and may not be representative of other locations and times. The stratification lines presented on the logs represent the approximate boundaries between soil types, and the transitions may be gradual.

In the geotechnical borings, relatively undisturbed soil samples were recovered at various intervals with a California sampler. The California sampler is a 2.9-inch outside diameter, 2.5-inch inside diameter, split barrel sampler lined with brass rings. The sampler was 18 inches long. The sampler conformed to the requirements of ASTM D 3550. A 140-pound automatic trip hammer was utilized, dropping 30 inches for each blow. The relatively undisturbed samples, together with bulk samples of representative soil types, were returned to the laboratory for testing and evaluation.

In Boring B-1 standard penetration tests (SPT) were performed with a 2.0-inch outside diameter, 1.5-inch inside diameter, split-barrel sampler. The sampler was 18 inches long. The inside diameter of the sampler shoe was 1.4 inches. The sampler was unlined. The sampler conformed to the requirements of ASTM D 1586. A 140-pound automatic trip hammer was utilized, dropping 30 inches for each blow. The sampler penetration test data are presented on the Log for Boring for Boring B-1.

3.2 LABORATORY TESTING

Laboratory testing was performed on selected soil samples obtained during the field exploration. The purpose of the laboratory testing was to confirm the field classification of the soils encountered and to evaluate the physical properties of the soils for use in engineering design and analysis.

Included in the laboratory testing were moisture-density determinations on relatively undisturbed samples. An optimum moisture content-maximum dry density relationship was established for a typical soil type so that the relative compaction of the subsoils could be determined. Collapse testing was performed on selected samples to evaluate the compressibility characteristics of the soils. Expansion index testing was performed on a selected sample to evaluate the expansion potential of the on-site soils. Chemical testing comprised of pH, soluble sulfate, chloride and resistivity testing was conducted on a selected sample. The moisture-density data are presented on the exploration logs. The maximum density, collapse, expansion index and chemical test data are presented in Appendix B.

4. GEOLOGIC AND SOILS CONDITIONS

4.1 REGIONAL SETTING

The subject property is situated in the Peninsular Ranges geomorphic province. The Peninsular Ranges province is one of the largest geomorphic units in western North America. It extends approximately 975 miles south of the Transverse Ranges geomorphic province to the tip of Baja California. This province varies in width from about 30 to 100 miles. It is bounded on the west by the Pacific Ocean, on the south by the Gulf of California and on the east by the Colorado Desert Province.

The Peninsular Ranges are essentially a series of northwest-southeast oriented fault blocks. Several major fault zones are found in this province. The Elsinore Fault zone and the San Jacinto Fault zone trend northwest-southeast and are found near the middle of the province. The San Andreas Fault zone borders the northeasterly margin of the province.

More specific to the subject property, the site is located southwest of the San Jacinto Fault zone, southeast of the Cucamonga Fault zone and northeast of the Chino Fault zone. In general, the site is underlain by alluvium derived from the mountains located to the north. A Geologic Map of the area is included in Figure 2, and a Regional Fault Map (Morton and Miller, 2006) is presented on Figure 3.

The Elsinore Fault is located approximately 10 miles to the southwest of the site and the San Jacinto Fault is located approximately 11 miles to the northeast. A potential earthquake with a mean magnitude (MCE) of 7.0 may result from these faults. These are the known faults that would create the most significant earthshaking event. No faults are shown in the immediate site vicinity on maps reviewed for the area.

More specific to the subject property, the site is located in an area geologically mapped to be underlain by older alluvium (Dibblee, T.W. and Minch, J.A., 2004). No faults are shown in the immediate site vicinity on the maps reviewed for the area (see Figure 3).

4.2 GENERAL SOIL/GEOLOGIC CONDITIONS

A brief description of the lithologic units encountered on the site is presented in the following sections. Based on the field exploration and observations, the site is generally underlain by artificial fill soils overlying alluvium. A Geologic Map is presented on Figure 2. Geologic Cross Sections are presented on Figures 5 and 6.

4.2.1 Artificial Fill

Concrete pavement was present at the surface of both borings and within the proposed building areas. Artificial fill, consisting of compact dense silty sands (SM soil type based upon the Unified Soil Classification System), was encountered to depths of about 5 below existing grades in both borings. Deeper or other deposits of fill may be present in areas of the site that were not explored. This fill was probably placed during the initial grading of the high school campus.

4.2.2 Alluvium

Alluvial deposits, consisting of interbedded medium dense to very dense sands, silty sands and clayey sands (SP, SM and SC soil types), were encountered below the artificial fill to the maximum depth explored (51.5 feet) in both test borings.

Based on the laboratory test results, the near surface soils have a "very low" expansion potential (ASTM D 4829). Based on the laboratory test results, the near surface soils have a soluble sulfate content of less than 0.1 percent (ASTM D 4327). Based upon the collapse test results, the alluvial and fill soils have a low potential for hydroconsolidation (settlement upon wetting with or without additional loading). The test results are provided in Appendix B.

4.3 SURFACE AND GROUNDWATER

4.3.1 Surface Water

Surface water was not observed during the site exploration. If encountered during earthwork construction, surface water on this site will likely be the result of precipitation or possibly some minor surface run-off from surrounding areas. Overall site drainage is generally in a south/southwesterly direction, as directed by site topography. Provisions for surface drainage will need to be accounted for by the project civil engineer.

4.3.2 Groundwater

Groundwater was not encountered in either boring at the time of drilling to the maximum depth of exploration (51.5 feet). Based on groundwater levels reported in the vicinity of the site, the regional groundwater level is deeper than 75 feet below the ground surface (http://geotracker.waterboards.ca.gov/).

4.4 FAULTING AND SEISMICITY

The geologic structure of the entire southern California area is dominated mainly by northwest-trending faults associated with the San Andreas system. The site is in a seismically active region. No active or potentially active fault is presently known to exist at this site nor is the site situated within an *"Alquist-Priolo"* Earthquake Fault Zone (Bryant and Hart, 2007). The County of Riverside has designated the site area as "not in a fault zone", "not in a fault line", as having a "moderate" liquefaction potential and as being "susceptible" to subsidence. The Elsinore Fault is located approximately 10 miles to the southwest of the site and the San Jacinto Fault is located approximately 11 miles to the northeast.

4.4.1 Historical Site Seismicity

The historical seismicity in the project area has been reviewed. There does not appear to be obvious evidence of ground failure or structural damage due to previous earthquakes to the existing structures on the site.

4.4.2 Seismic Design Parameters

The site is located at approximately 34.003 West Latitude and -117.520 North Longitude. A Site-Specific Ground Motion Seismic Analysis report for the project site was prepared by Terra Geosciences dated January 29, 2022 (Project No. 223774-1). The purpose of this study was to evaluate the site-specific ground motion parameters to aid in the seismic design for this project, based on the 2019 California Building Code (CBC). This study included performing a seismic shear-wave survey study for determining the Site Classification and Vs30 input values for the seismic design parameter determination. This report is included in Appendix C of this report. Based upon the results of this study, the site can be classified as a Site "C".

The results, based on the site specific analysis, are presented in the following. More detailed information and analysis are presented in the referenced report dated January 29, 2022.

SITE SEISMIC PARAMETERS	
Mapped 0.2 sec Period Spectral Acceleration, S_S	1.592g
Mapped 1.0 sec Period Spectral Acceleration, S_1	0.581g
Site Coefficient for Site Class "C", F_a	1.2
Site Coefficient for Site Class "C", F_v	1.419
5% Damped Design Spectral Response Acceleration Parameter at 0.2 Second, S_{DS}	1.120g
5% Damped Design Spectral Response Acceleration Parameter at 1 second, S_{D1}	1.240g
Maximum Considered Earthquake Spectral Response Acceleration for 0.2 Second, S_{MS}	1.682g
Maximum Considered Earthquake Spectral Response Acceleration for 1.0 Second, S_{M1}	1.859g
T_L	12 seconds
MCE_G PGA	0.76g
Shear Wave Velocity	1,337.0 ft/sec
Site Classification	C
Risk Category	III

Final selection of the appropriate seismic design coefficients should be made by the project structural engineer based upon the local practices and ordinances, expected building response and desired level of conservatism.

4.5 LIQUEFACTION AND LATERAL SPREAD

Liquefaction describes a phenomenon in which cyclic stresses, produced by earthquake-induced ground motion, create excess pore pressures in relatively cohesionless soils. These soils may acquire a high degree of mobility which can lead to lateral movement, sliding, settlement of loose sediments, sand boils and other damaging deformations. This phenomenon occurs only below the water table, but, after liquefaction has developed, the effects can propagate upward into overlying non-saturated soil as excess pore water dissipates.

The factors known to influence liquefaction potential include soil type and grain size, relative density, groundwater level, confining pressures, and both intensity and duration of ground shaking. In general, materials that are susceptible to liquefaction are loose, saturated granular soils having low fines content under low confining pressures.

The County of Riverside indicates that the site has "moderate" liquefaction potential. The site is not located within an area that has been mapped by the Division of Mines and Geology nor is designated by the State of California as having potential for liquefaction. It is anticipated that major earthquake ground-shaking will occur during the lifetime of the proposed development from the seismically active San Jacinto Fault, Cucamonga Fault and Chino Fault. These are the known faults that would create the most significant earthshaking event.

Based on the current site mapping, depth to groundwater, and medium dense to very dense nature of the subsurface soils, it is GeoTek's opinion that the liquefaction potential at the site is very low.

Since groundwater is relatively deep and minimal liquefaction will occur below the groundwater elevation, lateral spread should not be a consideration in the design of the structure.

4.6 OTHER SEISMIC HAZARDS

Based on the Riverside County Parcel Report, the site is susceptible to subsidence. Any subsidence in the area would likely be regional and not adversely affect the subject development specifically.

The potential for seismic densification ("dry sand" seismic settlement) resulting from seismic activity was reviewed. Due to the compact nature of the upper fill soils on the site and the medium dense to very dense nature of the subsurface soils, it is GeoTek's opinion that the seismic settlement potential of the site soils is minimal.

Evidence of ancient landslides or slope instability at this site was not observed during the field investigation and the project site is relatively flat. Thus, the potential for landslides is considered negligible for design purposes.

The potential for secondary seismic hazards such as a seiche or tsunami is considered negligible due to site elevation and distance to an open body of water.

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 GENERAL

The anticipated site development appears feasible from a geotechnical viewpoint provided that the following recommendations, and those provided by this firm at a later date are incorporated into the design and construction phases of development. Site development, grading and foundation plans should be reviewed by GeoTek, Inc. when they become available so the recommendations contained in this report can be confirmed.

The on-site soils exhibit a "very low" expansion index. Expansion index testing should be conducted at the completion of earthwork operations to verify this design value.

Collapse testing indicated that the site soils have a low potential for hydroconsolidation. The upper fill soils are anticipated to be disturbed during site demolition and grading operations. Overexcavation and compaction of the upper soils below and within five feet of the building will be required.

5.2 EARTHWORK CONSIDERATIONS

Earthwork and grading should be performed in accordance with the applicable grading ordinances of the City of Jurupa Valley, the County of Riverside, the 2019 California Building Code (CBC), and recommendations contained in this report. The Grading Guidelines included in Appendix D outline general procedures and do not anticipate all site-specific situations. In the event of conflict, the recommendations presented in the text of this report should supersede those contained in Appendix D.

5.2.1 Site Clearing and Demolition

In areas of planned grading and improvements, the locations of existing utilities should be determined. The utilities should be relocated or abandoned. The site should be cleared of existing structures, pavements, trees, vegetation and other deleterious materials. Debris should be properly disposed of off-site. Voids resulting from site clearing should be backfilled with engineered fill.

5.2.2 Site Preparation

Demolition and removal of the existing on-site slabs and possible utility lines is anticipated to disturb the upper site soils. Due to the anticipated site demolition, it is recommended that the soils be removed beneath the planned building footprint to a depth of at least three (3) feet below existing grade and into native alluvium, or one (1) foot below the base of the proposed foundations, whichever is greater. The lateral extent of this recommended over-excavation should extend at least 5 feet beyond the building limits. Removal bottoms should be relatively uniform in soil type which is not visibly porous and having an in-place density of at least 85 percent of the soil's maximum dry density as determined by ASTM D 1557 test procedures.

5.2.3 Pavement Areas

Soils beneath proposed site pavement should be overexcavated to a depth of 12 inches below existing grade or 12 inches below proposed finished grade, whichever is deeper. Finished grade is defined as the top of the subgrade. The exposed soils in these areas and in cut areas should be scarified to a depth of approximately eight inches, moistened to at least the optimum moisture content and compacted to a minimum relative compaction of at least 95 percent of maximum dry density as determined by ASTM D 1557 test procedures.

5.2.4 Hardscape Areas

Undocumented fill should be removed below hardscape areas. The exposed soils in these areas and in cut areas should be scarified to a depth of approximately eight inches, moistened to at least the optimum moisture content and compacted to a minimum relative compaction of at least 90 percent of maximum dry density as determined by ASTM D 1557 test procedures

5.2.5 Preparation of Excavation Bottoms

A representative of this firm should observe the bottom of all excavations. Upon approval, the exposed soils below the building footings should be scarified to a depth of approximately eight inches, moistened to at least the optimum moisture content and compacted to a minimum relative compaction of at least 95 percent of maximum dry density as determined by ASTM D 1557 test procedures.

5.2.6 Engineered Fills

The on-site soils are generally considered suitable for reuse as engineered fill provided they are free from vegetation, debris and other deleterious material. Portland cement concrete that is to be removed from the site may be pulverized into fragments not exceeding three inches in greatest dimension and incorporated into the fill at all levels. Engineered fill should be placed in loose lifts with a thickness of eight inches or less and moisture conditioned to at least two percent above the optimum moisture content. Engineered fill should compacted to at least 90 percent of maximum dry density as determined by ASTM D 1557 test procedures.

5.2.7 Excavation Characteristics

Excavation of the on-site alluvial soils is expected to be feasible utilizing heavy-duty grading equipment in good operating condition. All temporary excavations for grading purposes and installation of underground utilities should be constructed in accordance with local and Cal-OSHA guidelines. Temporary excavations within the on-site materials should be stable at 1:1 (horizontal: vertical) inclinations for cuts less than five feet in height.

5.2.8 Shrinkage and Subsidence

Several factors will impact earthwork balancing on the site, including shrinkage, subsidence, trench spoil from utilities and footing excavations, as well as the accuracy of topography.

Shrinkage and subsidence are primarily dependent upon the degree of compactive effort achieved during construction. For planning purposes, a shrinkage factor of 5 to 10 percent may be considered for the materials requiring removal and/or recompaction. Site balance areas should be available in order to adjust project grades, depending on actual field conditions at the conclusion of earthwork. Subsidence on the order of up to 0.10 foot may be anticipated for the underlying soils.

5.3 DESIGN RECOMMENDATIONS

5.3.1 Foundation Design Criteria

Foundation design criteria for a conventional foundation system, in general conformance with the 2019 CBC, are presented below. Based on laboratory test results, subsequent to earthwork operations it is anticipated that the near-surface soils will have a "very low" expansion potential. Additional expansion index and soluble sulfate testing of the soils should be performed during construction to evaluate the as-graded conditions. Final recommendations should be based upon the as-graded soils conditions. A summary of the foundation design recommendations is presented in the following table:

DESIGN PARAMETER	"VERY LOW" EXPANSION POTENTIAL
Building Foundation Depth or Minimum Perimeter Beam Depth (inches below lowest adjacent grade)	12
Minimum Foundation Width (Inches)	12
Minimum Slab Thickness (actual)	4 inches
Minimum Slab Reinforcing	6" x 6" – W2.9/W2.9 welded wire fabric or No. 3 bars at 18 inch centers placed in middle of slab
Minimum Footing Reinforcement	Two No. 4 reinforcing bars, one placed near the top and one near the bottom of the footing
Presaturation of Subgrade Soil (Percent of Optimum)	Minimum of 100 percent of the optimum moisture content to a depth of at least 12 inches prior to placing concrete

It should be noted that the criteria provided are based on soil support characteristics only. The structural engineer should design the slab and foundation reinforcement based on actual loading conditions.

The following criteria for design of foundations are preliminary and should be re-evaluated based on the results of additional laboratory testing of samples obtained near finish pad grade.

The building footings should have a minimum embedment depth of 12 inches. An allowable bearing capacity of 2,000 pounds per square foot (psf) may be used for design of isolated and continuous building footings. An increase of one-third may be applied when considering short-term seismic and wind loads.

Structural foundations may be designed in accordance with the 2019 CBC, and to withstand a total settlement of 1 inch and maximum differential settlement of one-half of the total settlement over a horizontal distance of 40 feet.

The passive earth pressure may be computed as an equivalent fluid having a density of 375 psf per foot of depth, to a maximum earth pressure of 3,750 psf for footings founded on engineered fill. A coefficient of friction between soil and concrete of 0.38 may be used with dead load forces. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third.

A moisture and vapor retarding system should be placed below slabs-on-grade where moisture migration through the slab is undesirable. Guidelines for these are provided in the 2019 California Green Building Standards Code (CALGreen) Section 4.505.2, the 2019 CBC Section 1907.1 and ACI 360R-10. The vapor retarder design and construction should also meet the requirements of ASTM E 1643. A portion of the vapor retarder design should be the implementation of a moisture vapor retardant membrane.

The effectiveness of the vapor retarding membrane can be adversely impacted as a result of construction related punctures. These occurrences should be limited as much as possible during construction. Thicker membranes are generally more resistant to accidental puncture than thinner ones. Products specifically designed for use as moisture/vapor retarders may also be more puncture resistant. Although the CBC specifies a 6-mil vapor retarder membrane, a minimum 10 mil thick membrane with joints properly overlapped and sealed should be considered, unless otherwise specified by the slab design professional. The membrane should consist of Stego wrap or the equivalent.

Moisture and vapor retarding systems are intended to provide a certain level of resistance to vapor and moisture transmission through the concrete, but do not eliminate it. The acceptable level of moisture transmission through the slab is to a large extent based on the type of flooring used and environmental conditions. Ultimately, the vapor retarding system should be comprised of suitable elements to limit migration of water and reduce transmission of water vapor through the slab to acceptable levels. The selected elements should have suitable properties such as thickness, composition, strength, and permeability to achieve the desired performance level.

Moisture retarders can reduce, but not eliminate, moisture vapor rise from the underlying soils up through the slab. Moisture retarder systems should be designed and constructed in accordance with applicable American Concrete Institute, Portland Cement Association, Post-Tensioning Concrete Institute, ASTM and California Building Code requirements and guidelines.

GeoTek recommends that a qualified person, such as a flooring contractor, structural engineer, architect, and/or other experts specializing in moisture control within buildings be consulted to evaluate the general and specific moisture and vapor transmission paths and associated potential impact on the proposed construction. That person should provide recommendations relative to the slab moisture and vapor retarder systems and for migration of potential adverse impact of moisture vapor transmission on various components of the structures, as deemed appropriate.
In addition, the recommendations in this report and GeoTek's services in general are not intended to address mold prevention, since GeoTek, along with geotechnical consultants in general, do not practice in the area of mold prevention. If specific recommendations addressing potential mold issues are desired, then a professional mold prevention consultant should be contacted.

It is recommended that control joints be placed in two directions spaced approximately 24 to 36 times the thickness of the slab in inches. These joints are a widely accepted means to control cracks and should be reviewed by the project structural engineer.

5.3.2 Miscellaneous Foundation Recommendations

To minimize moisture penetration beneath the slab-on-grade areas, utility trenches should be backfilled with engineered fill, lean concrete or concrete slurry where they intercept the perimeter footing or thickened slab edge.

Soils from the footing excavations should not be placed in the slab-on-grade areas unless properly compacted and tested. The excavations should be free of loose/sloughed materials and be neatly trimmed at the time of concrete placement.

5.3.3 Foundation Set Backs

Minimum setbacks for all foundations should comply with the 2019 CBC or City of Jurupa Valley requirements, whichever is more stringent. Improvements not conforming to these setbacks are subject to the increased likelihood of excessive lateral movement and/or differential settlement. If large enough, these movements can compromise the integrity of the improvements.

- The outside top edge of all footings should be set back a minimum of H/3, where H is the slope height, from the face of any descending slope. The setback should be at least five feet and need not exceed 40 feet.

- The bottom of any proposed foundations should be deepened so as to extend below a 1:1 upward projection from the bottom edge of the nearest excavation and the bottom edge of the closest footing.

5.3.4 Retaining and Screen Wall Design and Construction

5.3.4.1 General Design Criteria

Retaining wall foundations supporting the building should be embedded a minimum of 12 inches into engineered fill. Retaining wall foundations independent of the building should be embedded a minimum of 12 inches into engineered fill. Retaining wall foundations should be designed in accordance with Section 5.3.1 of this report. Structural requirements may govern and should be evaluated by the project structural engineer.

All earth retention structure plans should be reviewed by this office prior to finalization.

Site clearing and remedial earthwork for all earth retention structures should meet the requirements of this report, unless specifically provided otherwise, or more stringent requirements or recommendations are made by the wall designer. The soil used as backfill behind retaining walls should have a "very low" expansion potential and should be densified to at least 90 percent relative compaction (ASTM D 1557).

In general, cantilever retaining walls which are designed to yield at least 0.001H, where H is equal to the height of the wall to the base of the footing may be designed using the active condition. Rigid earth retention structures (including but not limited to rigid walls, and walls braced at the top, such as typical basement walls) should be designed using the at-rest condition.

In addition to the design lateral forces due to retained earth, surcharges due to improvements, such as an adjacent structure, should be considered in the design of the retaining walls. Loads applied within a 1:1 (h:v) projection from the surcharging structure on the stem of the retaining wall should be considered in the design.

Final selection of the appropriate design parameters should be made by the retaining wall designer based upon the local practices and ordinances, expected structure response, and desired level of conservatism.

5.3.4.2 Cantilevered Walls

The recommendations presented below are for cantilevered retaining walls up to six feet high. Active earth pressure may be used for retaining wall design, provided the top of the wall is not restrained from minor deflections. Appropriate fluid unit weights are given below for specific slope gradients of the retained material. These do not include other superimposed loading conditions such as traffic, structures, seismic events, or adverse geologic conditions.

ACTIVE EARTH PRESSURES	
Surface Slope of Retained Materials (h:v)	Equivalent Fluid Pressure* (pcf)
Level	40
2:1	65

*The design pressures assume the backfill materials have an expansion index less than or equal to 20. Backfill zone includes the area between the back of the wall to a plane (1:1, h:v) up from the bottom of the wall foundation to the adjacent ground surface.

5.3.4.3 Retaining Wall Backfill and Drainage

Wall backfill should include a minimum one foot wide section of ¾- to one-inch clean crushed rock or approved equivalent. The rock should be placed immediately adjacent to the back of the wall and extend up from a backdrain to within approximately 12 inches of finish grade. The portion of the rock opposite the back of the wall adjacent to the soil backfill should be covered with a layer of filter fabric comprised of Mirafi 140N or the equivalent. The upper 12 inches of backfill should consist of compacted on-site soil. Backfill placed within the active zone as defined by a 1:1 (H:V) projection from the back of the retaining wall footing up to the retained surface behind the wall should consist of very low expansive soil. The presence of other soils placed within the 1:1 projection will necessitate revision to the parameters provided and modification of wall designs.

The backfill soil should be placed in lifts no greater than eight inches in thickness, moisture conditioned to at least optimum moisture content and compacted to at least 90 percent of maximum dry density as determined by ASTM D 1557 test procedures. Proper surface drainage needs to be provided and maintained. Water should not be allowed to pond behind retaining walls. Waterproofing of site walls should be performed where moisture migration through the walls is undesirable.

Retaining walls should be provided with an adequate pipe and gravel back drain system to reduce the potential for hydrostatic pressures to develop. A 4-inch diameter perforated collector pipe (Schedule 40 PVC or approved equivalent) in a minimum of one cubic foot per linear foot of ¾-inch or one inch clean crushed rock or equivalent, wrapped in filter fabric should be placed near the bottom of the backfill and the water should be directed to an appropriate disposal area.

As an alternative to the drain, rock and fabric, a pre-manufactured wall drainage product (example: Mira Drain 6000 or approved equivalent) may be used behind the retaining wall. The wall drainage product should extend from the base of the wall to within two (2) feet of the ground surface. The subdrain should be placed in direct contact with the wall drainage product.

Walls from two to four feet in height may be drained using localized gravel packs (e.g., approximately 1.5 cubic feet of gravel in a woven plastic bag) behind weep holes at 10 feet maximum spacing. Weep holes should be provided or the head joints omitted in the first course of block extended above the ground surface. However, nuisance water may still collect in front of the wall.

Drain outlets should be maintained over the life of the project and should not be obstructed or plugged by adjacent improvements.

5.3.4.4 Restrained Retaining Walls

Retaining walls that will be restrained prior to placing and compacting backfill material or that have reentrant or male corners, should be designed for an at-rest equivalent fluid pressure of 65 pcf, plus any applicable surcharge loading. For areas of male or reentrant corners, the restrained wall design should extend a minimum distance of twice the height of the wall laterally from the corner, or a distance otherwise determined by the project structural engineer.

5.3.4.5 Other Design Considerations

- Retaining and screen wall foundation elements should be designed in accordance with building code setback requirements. A minimum horizontal setback distance of five feet as measured from the top outside edge of the footing to an adjacent slope face is recommended.
- Wall design should consider the additional surcharge loads from superjacent slopes and/or footings, where appropriate.
- No backfill should be placed against concrete until minimum design strengths are evident by compression tests of cylinders.
- The retaining wall footing excavations, backcuts, and backfill materials should be approved the project geotechnical engineer or their authorized representative.
- Positive separations should be provided in screen walls at horizontal distances not exceeding 20 feet.

5.3.5 Soil Corrosivity

Based on the chemical test results presented in Appendix B, the corrosivity test results indicate that the on-site soils are "highly corrosive" to buried ferrous metal. This corrosion classification is obtained from "Corrosion Basics: An Introduction," by Pierre R. Roberge, 2nd Edition, 2005. Recommendations for protection of buried ferrous metal should be provided by a corrosion engineer.

5.3.6 Soil Sulfate Content

Based on the chemical test results presented in Appendix B, the sulfate test results on a sample obtained from the project site indicate a soluble sulfate content of less than 0.1% by weight. Soluble sulfate contents of this level would be in the range of "not applicable" (i.e., negligible) in accordance with ACI 318. Based on the test results and Table 4.3.1 of ACI 318, no special recommendations for concrete are required for this project due to soil sulfate exposure.

5.3.7 Import Soils

Import soils should have a "very low" expansion potential. GeoTek, Inc. also recommends that the proposed import soils be tested for expansion and corrosivity potential. GeoTek, Inc. should be notified a minimum of 72 hours prior to importing so that appropriate sampling and laboratory testing can be performed.

5.3.8 Concrete Flatwork

5.3.8.1 Exterior Concrete Slabs, Sidewalks and Driveways

Exterior concrete slabs, sidewalks and driveways should be designed using a four-inch minimum thickness. Some shrinkage and cracking of the concrete should be anticipated as a result of typical mix designs and curing practices utilized in construction.

Sidewalks may be under the jurisdiction of the governing agency. If so, jurisdictional design and construction criteria will apply, if more restrictive than the recommendations presented in this report.

Subgrade soils should be pre-moistened prior to placing concrete. The subgrade soils below exterior slabs and sidewalks should be pre-saturated to a minimum of 100 percent of the optimum moisture content to a depth of 12 inches.

All concrete installation, including preparation and compaction of subgrade, should be done in accordance with the City of Jurupa Valley and County of Riverside specifications, and under the observation and testing of GeoTek, Inc. and a City or County inspector, if necessary.

5.3.8.2 Concrete Performance

Concrete cracks should be expected. These cracks can vary from sizes that are hairline to more than 1/8 inch in width. Most cracks in concrete, while unsightly, do not significantly impact long-term performance. While it is possible to take measures (proper concrete mix, placement, curing, control joints, etc.) to reduce the extent and size of cracks that occur, some cracking will occur despite the best efforts to minimize it. Concrete undergoes chemical processes that are dependent on a wide range of variables which are difficult, at best, to control. Concrete, while seemingly a stable material, is subject to internal expansion and contraction due to external changes over time.

One of the simplest means to control cracking is to provide weakened control joints for cracking to occur along. These do not prevent cracks from developing; they simply provide a relief point for the stresses that develop. These joints are a widely accepted means to control cracks but are not always effective. Control joints are more effective the more closely spaced they are. GeoTek, Inc. suggests that control joints be placed in two directions and located a distance apart approximately equal to 24 to 36 times the slab thickness.

5.3.9 Pavement Design

5.3.9.1 Portland Cement Concrete (PCC) Pavement

For the proposed vehicle parking and access lanes, it is recommended that a minimum of 5 inches of PCC pavement over 12 inches subgrade compacted to at least 95 percent of maximum dry density as determined by ASTM D 1557 test procedures be utilized. For vehicle service lanes, it is recommended that a minimum of six inches of PCC pavement over 12 inches subgrade compacted to at least 95 percent of maximum dry density be utilized. This section should also be used in heavy truck traffic areas such as fire lanes, trash dumpster pads and approaches. Requirements of Section 90 of Caltrans Standard Specifications, and various ACI and ASTM standards regarding mixing and placing concrete should be followed. The PCC pavement should have a minimum modulus of rupture of 600 pounds per square inch, and a minimum 28-day compressive strength of 4,000 pounds per square inch. Concrete should incorporate 1-inch maximum size aggregate and should be proportioned to achieve a maximum slump of four inches. Instead of increasing the water content, a plasticizing admixture may be utilized to increase the workability of the concrete. The concrete should be properly cured after placement. Concrete should not be placed during hot and windy weather.

Crack control joints should be provided in the transverse direction spaced at horizontal intervals ranging from 24 to 36 times the thickness of the concrete.

5.3.9.2 Pavement Construction

All pavement installation, including preparation and compaction of subgrade and base material, placement and rolling of asphaltic concrete and placement of concrete pavement, should be done in accordance with the City of Jurupa Valley guidelines, and under the observation and testing of GeoTek and a City inspector, where required.

Any aggregate base should consist of crushed rock with an R-Value and gradation in accordance with Crushed Aggregate Base (Section 400-2.4 of the "Greenbook" Regional Supplement Amendments). Minimum compaction requirements should be 95 percent of maximum dry density as determined by ASTM D 1557 test procedures for both soil subgrade and aggregate base. Jurisdictional minimum compaction requirements in excess of the aforementioned minimums may govern. The upper 12 inches of subgrade should be moisture-conditioned to at least optimum moisture.

The top of the subgrade should be graded to that is drains to the perimeter of the pavement.

In addition, over the lifetime of the water quality bioretention basins, the infiltration rates may be affected by silt build up and biological activities, as well as local variations in near surface soil conditions. A suitable factor of safety should be applied to the field rate to design the infiltration systems.

5.4 POST CONSTRUCTION CONSIDERATIONS

5.4.1 Landscape Maintenance and Planting

Water has been shown to weaken the inherent strength of soil, and slope stability is significantly reduced by overly wet conditions. Positive surface drainage away from graded slopes should be maintained and only the amount of irrigation necessary to sustain plant life should be provided for planted slopes. Controlling surface drainage and runoff, and maintaining a suitable vegetation cover can minimize erosion. Plants selected for landscaping should be lightweight, deep-rooted types that require little water and are capable of surviving the prevailing climate.

Overwatering should be avoided. An abatement program to control ground-burrowing rodents should be implemented and maintained. Burrowing rodents can decrease the long-term performance of slopes.

It is common for planting to be placed adjacent to structures in planter or lawn areas. This will result in the introduction of water into the ground adjacent to the foundations. This type of landscaping should be avoided.

5.4.2 Drainage

Positive site drainage should be maintained at all times. Drainage should not flow uncontrolled down any descending slope. Water should be directed away from foundations and not allowed to pond or seep into the ground adjacent to the footings and floor-slabs. Pad drainage should be directed toward approved areas and not be blocked by other improvements.

Roof gutters should be installed that will direct the collected water at least 20 feet from the buildings.

5.5 PLAN REVIEW AND CONSTRUCTION OBSERVATIONS

It is recommended that specifications and foundation and grading plans be reviewed by GeoTek prior to construction to check for conformance with the recommendations of this report. It is also recommended that GeoTek, Inc. representatives be present during site grading and foundation construction to observe and document proper implementation of the geotechnical recommendations. The owner/developer should verify that GeoTek, Inc. representatives perform at least the following duties:

- Observe site clearing and grubbing operations for proper removal of unsuitable materials.
- Observe and test bottom of removals prior to fill placement.
- Evaluate the suitability of on-site and import materials for fill placement and collect soil samples for laboratory testing where necessary.
- Observe the fill for uniformity during placement, including utility trench backfill. Perform field density testing of the fill materials.
- Observe installation and modulus testing of RAP's.
- Observe and probe foundation excavations to confirm suitability of bearing materials with respect to density.

If requested, a construction observation and compaction report can be provided by GeoTek, Inc. which can comply with the requirements of the governmental agencies having jurisdiction over the project. It is recommended that these agencies be notified prior to commencement of construction so that necessary grading permits can be obtained.

6. INTENT

It is the intent of this report to aid in the design and construction of the proposed development. Implementation of the advice presented in this report is intended to reduce risk associated with construction projects. The professional opinions and geotechnical advice contained in this report are not intended to imply total performance of the project or guarantee that unusual or variable conditions will not be discovered during or after construction.

The scope of this evaluation is limited to the boundaries of the subject property. This review does not and should in no way be construed to encompass any areas beyond the specific area of the proposed construction as indicated to us by the client. Further, no evaluation of any existing site improvements is included. The scope of this evaluation is based on GeoTek's understanding of the project and geotechnical engineering standards normally used on similar projects in this locality.

7. LIMITATIONS

GeoTek's findings are based on site conditions observed and the stated sources. Thus, GeoTek's comments are professional opinions that are limited to the extent of the available data.

GeoTek has prepared this report in a manner consistent with that level of care and skill ordinarily exercised by members of the engineering and science professions currently practicing under similar conditions in the jurisdiction in which the services are provided, subject to the time limits and physical constraints applicable to this report.

Since GeoTek's recommendations are based on the site conditions observed and encountered, and laboratory testing, GeoTek's conclusions and recommendations are professional opinions that are limited to the extent of the available data. Observations during construction are important to allow for any change in recommendations found to be warranted. These opinions have been derived in accordance with current standards of practice and no warranty of any kind is expressed or implied. Standards of care/practice are subject to change with time.

8. SELECTED REFERENCES

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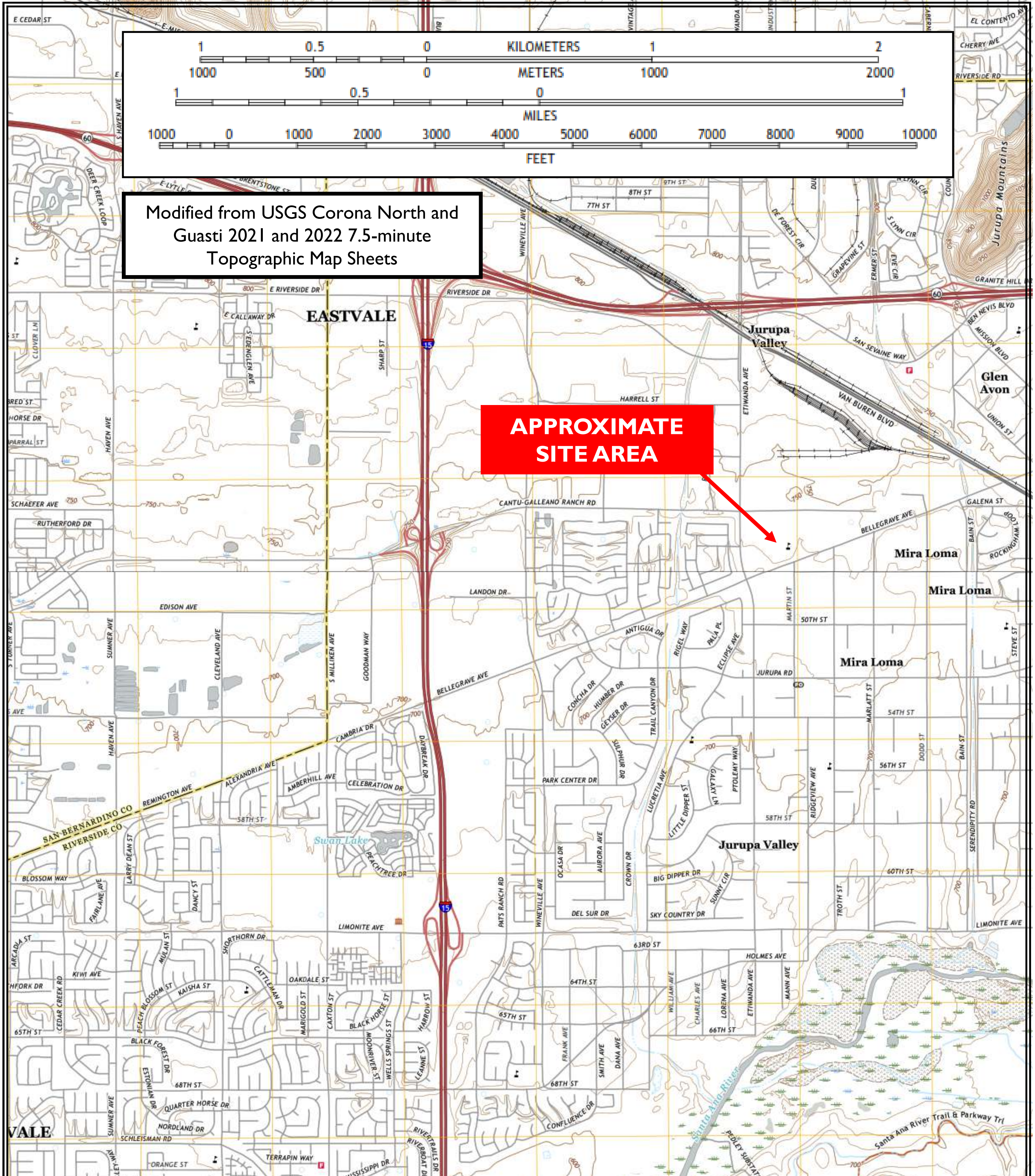


Jurupa Unified School District
Storage Project
Jurupa Valley High School
10551 Bellegrave Avenue
Jurupa Valley, Riverside County, California

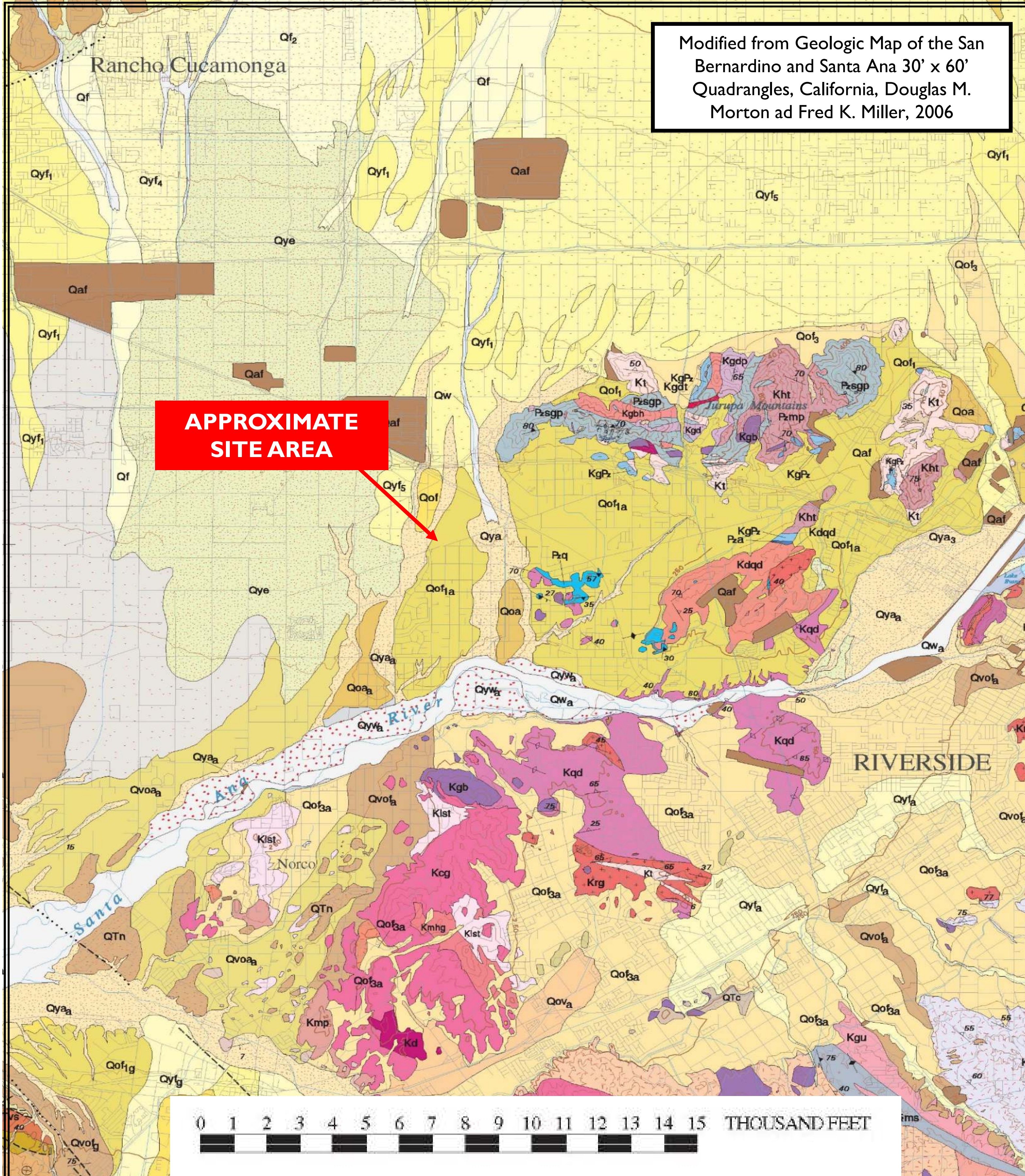
Project No. 3050-CR

NORTH

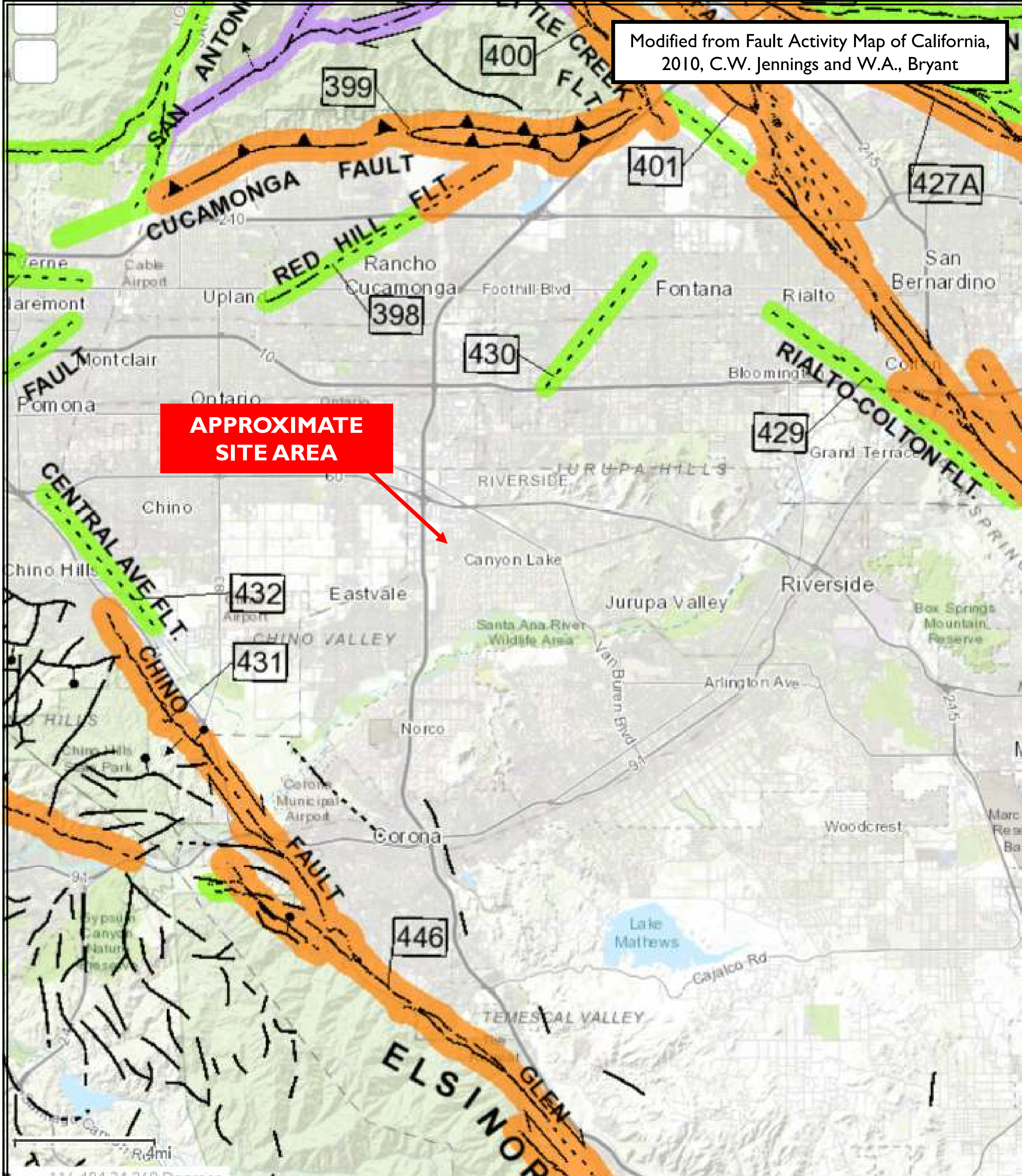
Figure 1

Site Location and General Topography Map

GEOTEK



Jurupa Unified School District Storage Project Jurupa Valley High School 10551 Bellegrave Avenue Jurupa Valley, Riverside County, California Project No. 3050-CR	NORTH	**Figure 2** **Geologic Map**	GEOTEK



Jurupa Unified School District
Storage Project
Jurupa Valley High School
10551 Bellegrave Avenue
Jurupa Valley, Riverside County, California

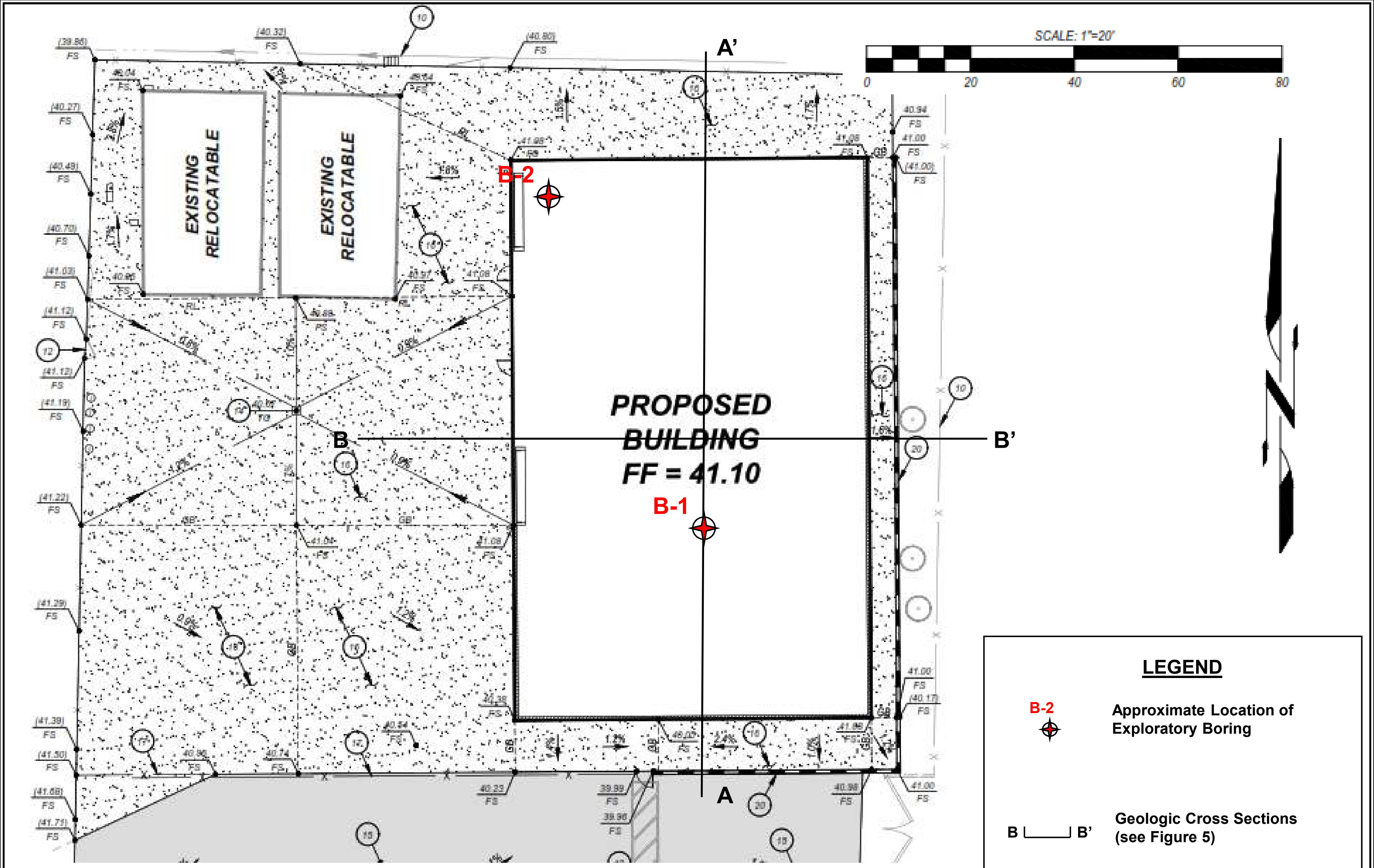
Project No. 3050-CR

NORTH

Figure 3

Regional Fault Map

GEOTEK

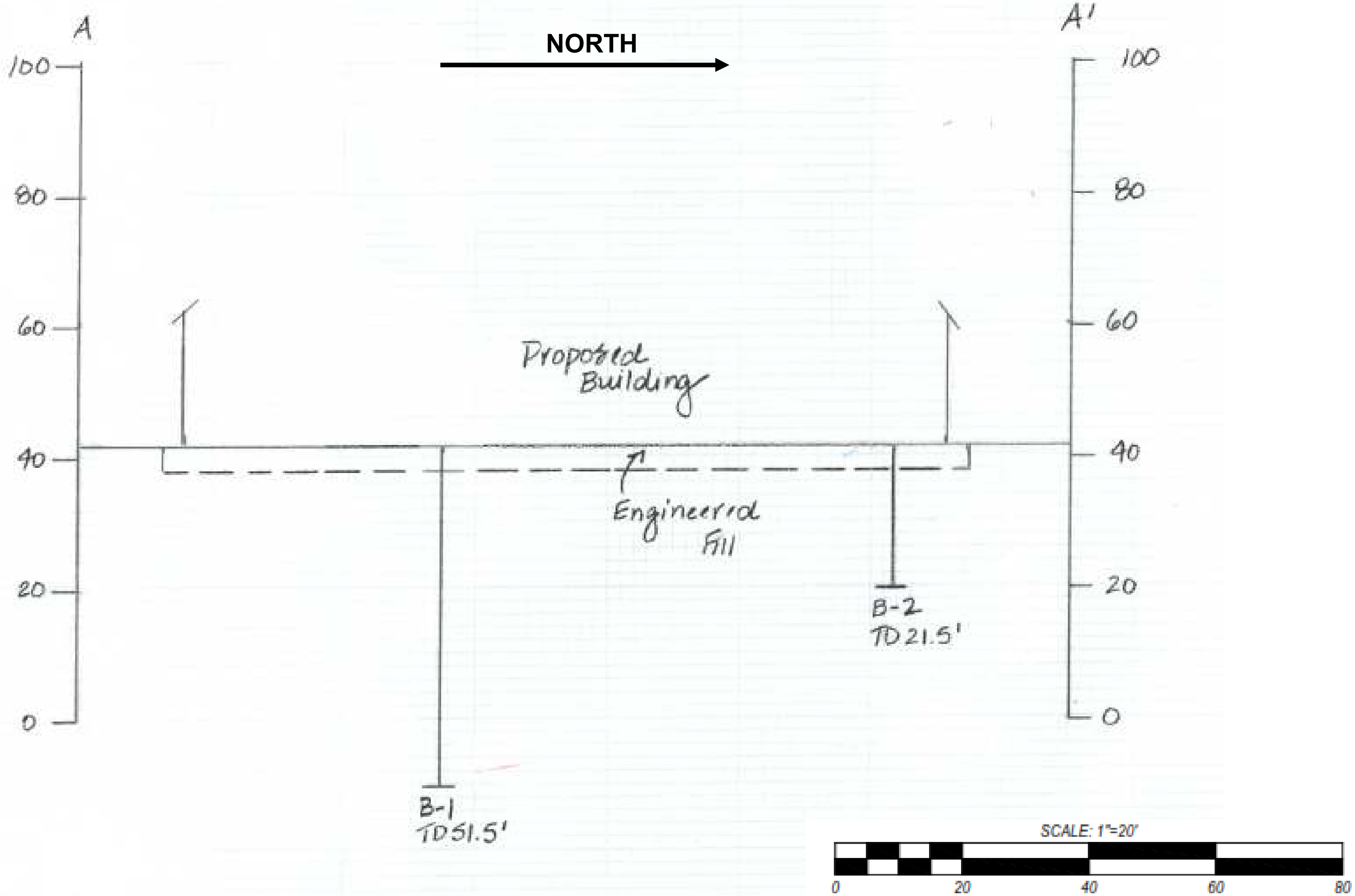


Jurupa Unified School District
Storage Project - Jurupa Valley High School
10551 Bellegrave Avenue, Jurupa Valley, Riverside County, California
GeoTek Project No. 3050-CR

Figure 4

Boring Location Map

GEOTEK

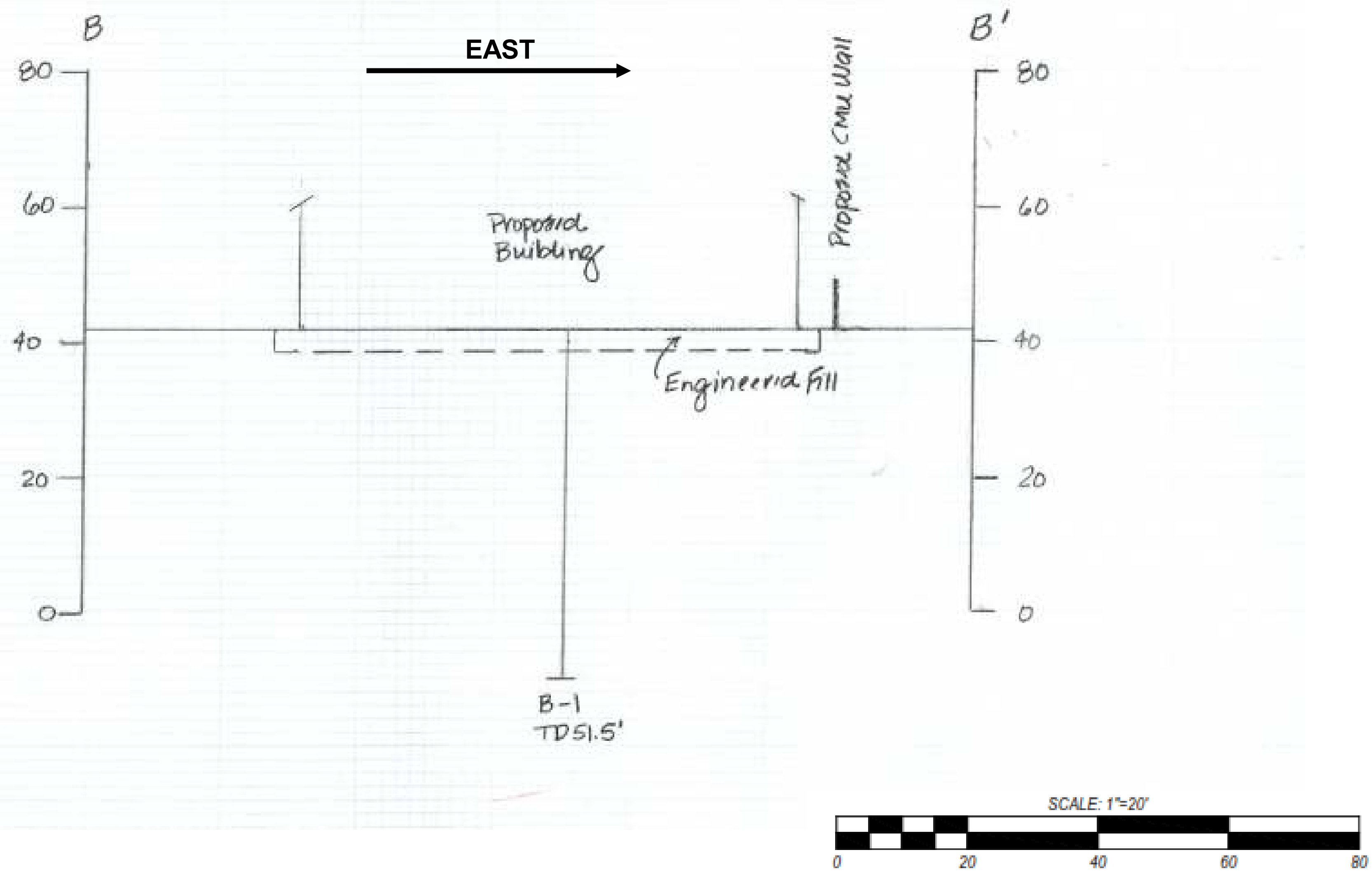


Jurupa Unified School District
Storage Project - Jurupa Valley High School
10551 Bellegrave Avenue, Jurupa Valley, Riverside County, California
GeoTek Project No. 3050-CR

Figure 5

Geologic Cross Section A – A'

GEOTEK



Jurupa Unified School District
Storage Project - Jurupa Valley High School
10551 Bellegrave Avenue, Jurupa Valley, Riverside County, California
GeoTek Project No. 3050-CR

Figure 6

Geologic Cross Section B – B'

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APPENDIX A

LOGS OF EXPLORATORY BORINGS

Jurupa Valley High School
Proposed Jurupa Unified School District Storage Project
Jurupa Valley, Riverside County, California
Project No. 3050-CR

GEOTEK

A - FIELD TESTING AND SAMPLING PROCEDURES

The Modified Split-Barrel Sampler (Ring)
The Ring sampler is driven into the ground at various depths in accordance with ASTM D 3550 test procedures. The sampler, with an external diameter of 3.0 inches, is lined with 1-inch long, thin brass rings with inside diameters of approximately 2.4 inches. The sampler is typically driven into the ground 12 or 18 inches with a 140-pound hammer free falling from a height of 30 inches. Blow counts are recorded for every 6 inches of penetration as indicated on the log of boring. The samples are removed from the sample barrel in the brass rings, sealed, and transported to the laboratory for testing.

The Standard Penetration Test (SPT) Sampler
Standard penetration tests (SPT) were performed with a 2.0-inch outside diameter, 1.5-inch inside diameter, split-barrel sampler. The sampler was 18 inches long. The inside diameter of the sampler shoe was 1.4 inches. The sampler was unlined. The sampler conformed to the requirements of ASTM D 1586. A 140-pound automatic trip hammer was utilized, dropping 30 inches for each blow. Blow counts are recorded for every 6 inches of penetration as indicated on the log of boring. Disturbed samples are removed from the sample barrel, sealed in a plastic bag, and transported to the laboratory for testing.

Bulk Samples (Large)
These samples are normally large bags of earth materials over 20 pounds in weight collected from the field by means of hand digging or exploratory cuttings.

Bulk Samples (Small)
These are plastic bag samples which are normally airtight and contain less than 5 pounds in weight of earth materials collected from the field by means of hand digging or exploratory cuttings. These samples are primarily used for determining natural moisture content and classification indices.

B – BORING/TRENCH LOG LEGEND

The following abbreviations and symbols often appear in the classification and description of soil and rock on the logs of borings/trenches:

SOILS

USCS	Unified Soil Classification System
f-c	Fine to coarse
f-m	Fine to medium

GEOLOGIC

B: Attitudes	Bedding: strike/dip
J: Attitudes	Joint: strike/dip
C: Contact line	
………..	Dashed line denotes USCS material change
———	Solid Line denotes unit / formational change
━━━	Thick solid line denotes end of boring/trench

(Additional denotations and symbols are provided on the log of borings/trenches)

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Jurupa Valley Unified School District	**DRILLER:**	2R Drilling	**LOGGED BY:**	C. Diaz
PROJECT NAME:	Jurupa Valley HS - Storage	**DRILL METHOD:**	Hollow Stem	**OPERATOR:**	Miguel
PROJECT NO.:	3050-CR	**HAMMER:**	140#/30"	**RIG TYPE:**	CME 75
LOCATION:	Jurupa Valley, CA			**DATE:**	1/26/2022

Boring No.: B-1

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
0					5" Concrete, no aggregate base			
					Fill:			
								MD, EI, SH, SR
		20 20 23	R1	SM	Silty f-m SAND, brown, slightly moist, compact	7.2	120.5	HC
5		9 12 16	R2	SP	**Alluvium:** F-m SAND, trace silt, yellow brown, slightly moist, medium dense	3.3	113.3	HC
		25 50/5	R3	SM	Silty f-m SAND, yellow brown, slightly moist, very dense	7.4	119.0	
10		50/6	R4		Silty f-m SAND, yellow brown, slightly moist, very dense	9.0	118.1	
15		25 35 25	R5		becomes very dense	6.6	112.0	
20		20 40 50	R6	SM/SP	Silty f-m SAND to f-m SAND, yellow brown, slightly moist, very dense	111.9	4.7	
25		11 12 17	S1	SM	Silty f-m SAND, yellow brown, slightly moist, medium dense		11.8	
30		12 19 24	S2				3.8	

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing:
AL = Atterberg Limits | EI = Expansion Index | SA = Sieve Analysis | RV = R-Value Test
SR = Sulfate/Resisitivity Test | SH = Shear Test | HC= Consolidation | MD = Maximum Density

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	Jurupa Valley Unified School District	**DRILLER:**	2R Drilling	**LOGGED BY:**	C. Diaz
PROJECT NAME:	Jurupa Valley HS - Storage	**DRILL METHOD:**	Hollow Stem	**OPERATOR:**	Miguel
PROJECT NO.:	3050-CR	**HAMMER:**	140#/30"	**RIG TYPE:**	CME 75
LOCATION:	Jurupa Valley, CA			**DATE:**	1/26/2022

Boring No.: B-1 (continued)

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
35	SPT	34 26 20	S3	SM	Silty f-m SAND, yellow brown, slightly moist, dense	3.8		
40	SPT	37 50/6	S4		Silty f-m SAND, light brown, slightly moist, very dense			
45	SPT	7 10 15	S5	SM/SC	Silty clayey SAND, yellow brown, moist, medium dense	13.7		
50	SPT	20 26 28	S6	SP	F-m SAND, light brown, slightly moist, dense			
					BORING TERMINATED AT 51.5 FEET No groundwater encountered Boring backfilled with soil cuttings			

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing:

AL = Atterberg Limits	EI = Expansion Index	SA = Sieve Analysis	RV = R-Value Test
SR = Sulfate/Resisitivity Test	SH = Shear Test	HC= Consolidation	MD = Maximum Density

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	Jurupa Valley Unified School District	**DRILLER:**	2R Drilling	**LOGGED BY:**	C. Diaz
PROJECT NAME:	Jurupa Valley HS - Storage	**DRILL METHOD:**	Hollow Stem	**OPERATOR:**	Miguel
PROJECT NO.:	3050-CR	**HAMMER:**	140#/30"	**RIG TYPE:**	CME 75
LOCATION:	Jurupa Valley, CA			**DATE:**	1/26/2022

Boring No.: B-2

Depth (ft)	Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Water Content (%)	Dry Density (pcf)	Others
0					4 inches of Concrete, no aggregate base			
					Fill:			
5	Ring	8 15 18	R1	SM/SC	Silty/Clayey f-m SAND, orange brown, moist, compact	8.2	118.6	HC
	Ring	13 36 50/5	R2	SM	**Alluvium:**	9.0	123.4	
	Ring	50/6	R3		Silty f-m SAND, orange brown, slightly moist, very dense	9.1	128.1	
10	Ring	16 30 36	R4	SP/SM	F-m SAND to silty f-m SAND, orange brown, slightly moist, dense	3.6	122.8	
15	Ring	50/6	R5	SP	F-c SAND, trace silt, light brown, slightly moist, very dense	2.6		
20	Ring	15 28 37	R6		F-m SAND, trace silt, light brown, slightly moist, dense	4.5	112.8	
25								
30								

BORING TERMINATED AT 21.5 FEET

No groundwater encountered
Boring backfilled with soil cuttings

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing:
AL = Atterberg Limits
EI = Expansion Index
SA = Sieve Analysis
RV = R-Value Test
SR = Sulfate/Resisitivity Test
SH = Shear Test
HC= Consolidation
MD = Maximum Density

APPENDIX B

LABORATORY TEST RESULTS

Jurupa Valley High School

Proposed Jurupa Unified School District Storage Project

Jurupa Valley, Riverside County, California

Project No. 3050-CR

GEOTEK

SUMMARY OF LABORATORY TESTING

Classification
Soils were classified visually in general accordance with the Unified Soil Classification System (ASTM Test Method D 2487). The soil classifications are shown on the boring logs in Appendix A.

Collapse Test
Collapse tests were performed on selected samples of the site soils obtained from the site exploration in general accordance with ASTM D 5333 test procedures. The results of this test are presented graphically in Appendix B.

Direct Shear
Shear testing was performed on a remolded sample in a direct shear machine of the strain-control type in general accordance with ASTM D 3080 test procedures. The rate of deformation is approximately 0.035 inch per minute. The samples were sheared under varying confining loads in order to determine the coulomb shear strength parameters, angle of internal friction and cohesion. The results of the testing are presented graphically in Appendix B.

Expansion Index
The expansion index of site soils obtained from the site exploration was determined by performing expansion index testing on a sample in general accordance with ASTM D 4829 test procedures. The results of the testing are provided below:

Boring No.	Depth (ft.)	Soil Type	Expansion Index	Classification
B-1	0-5	Silty Fine to Medium Sand (SM)	13	Very Low

In-Situ Moisture and Density
The natural water content of site soils obtained from the site exploration was determined in general accordance with ASTM D 2216 test procedures. In addition, in-place dry density determinations were performed on relatively undisturbed samples obtained from the site exploration in general accordance with ASTM D 2937 test procedures to measure the unit weight of the subsurface soils. Results of these tests are shown on the logs at the appropriate sample depths in Appendix A.

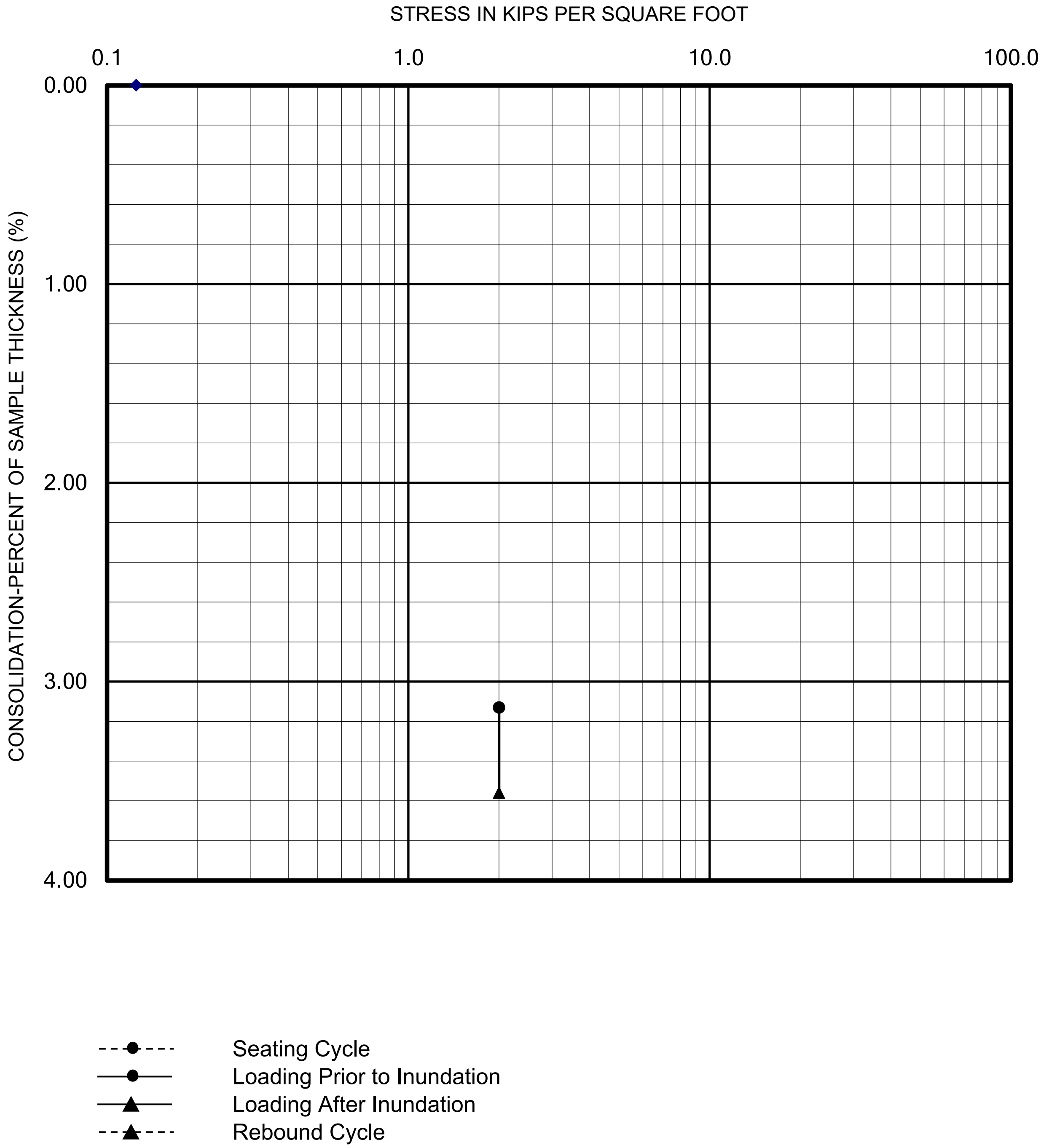
Moisture-Density Relationship
Laboratory testing consisting of a moisture-density relationship was performed on a sample obtained during the subsurface exploration. The laboratory maximum dry density and optimum moisture content was determined in general accordance with ASTM D 1557 test procedures. The results of the testing are provided below:

Boring No.	Depth (ft.)	Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
B-1	0 - 5	Silty Fine to Medium Sand (SM)	128.0	9.5

Sulfate Content, Resistivity and Chloride Content

Testing to determine the water-soluble sulfate content was performed by others in general accordance with ASTM D4327 test procedures. Resistivity testing was completed by others in general accordance with ASTM G187 test procedures. Testing to determine the chloride content was performed by others in general accordance with ASTM D4327 test procedures. The results of the testing are provided below and in Appendix B:

Boring No.	Depth (ft.)	pH ASTM G 51	Chloride ASTM D 512B(ppm)	Sulfate ASTM D 516 (% by weight)	Resistivity ASTM G 187 (ohm-cm)
B-1	0 - 5	7.9	57.4	0.0276	1,139



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

GEOTEK

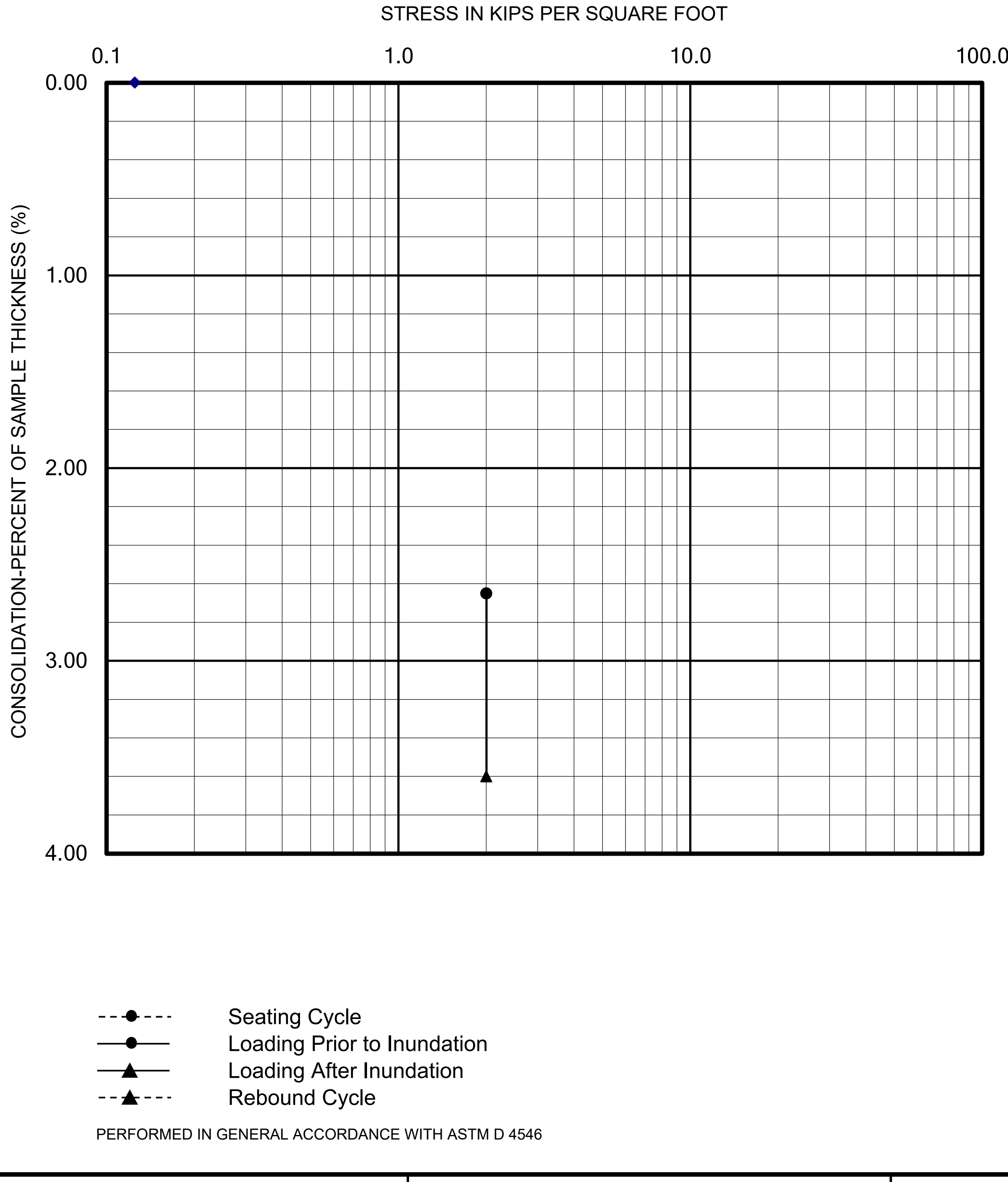
CHECKED BY: DA	Lab: Corona
PROJECT NO.: 3050-CR	Date: 2/15/2022

COLLAPSE REPORT

Sample: B- 1 @ 3 feet

JUSD Storage Project
Jurupa Valley, California

Plate B-1



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

GEOTEK

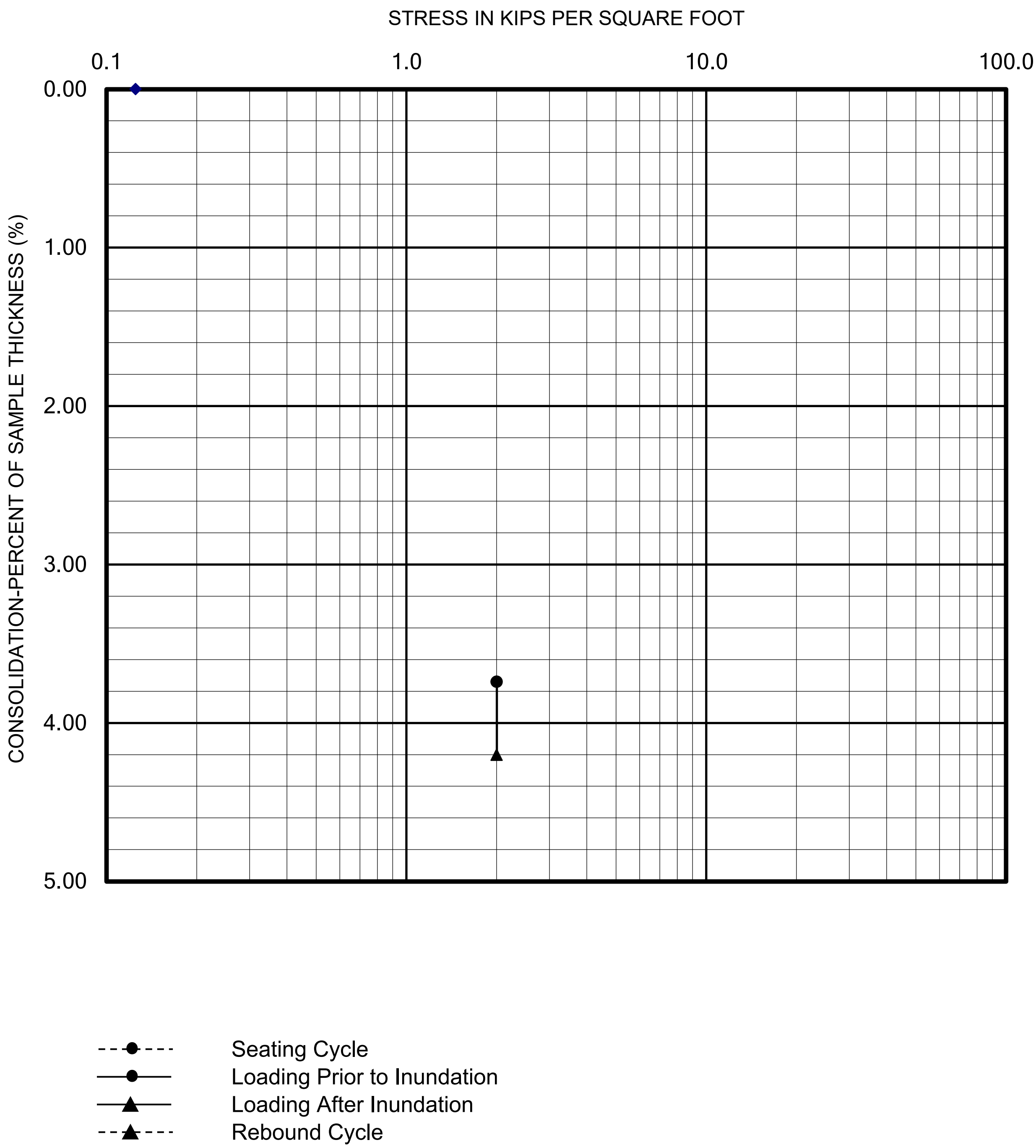
CHECKED BY: DA	Lab: Corona
PROJECT NO.: 3050-CR	Date: 2/15/2022

COLLAPSE REPORT

Sample: B- 1@ 5 feet

JUSD Storage Project
Jurupa Valley, California

Plate B-2



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

GEOTEK

COLLAPSE REPORT

Sample: B- 2@ 4 feet

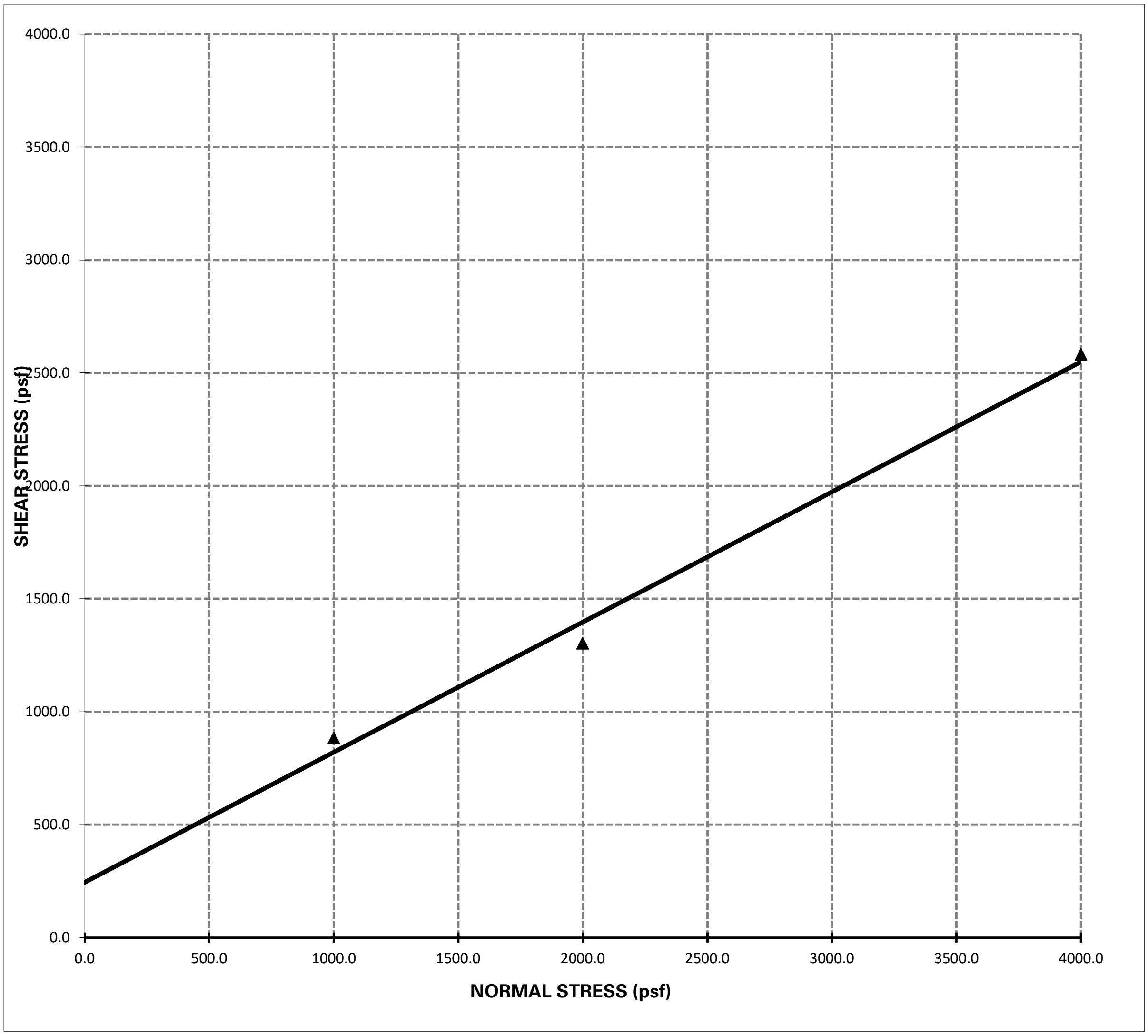
JuSD Storage Project
Jurupa Valley, California

Plate B-3

CHECKED BY: DA	Lab: Corona
PROJECT NO.: 3050-CR	Date: 2/15/2022

DIRECT SHEAR TEST

Project Name:	JUSD Storage Project	***Sample Location:***	B-1 @ 0-5 feet
Project Number:	14-Feb	***Date Tested:***	2/14/2022



Shear Strength: Φ = 30 ° C = 244 psf

Notes: 1 - The soil specimen used in the shear box was a ring sample remolded to approximately 90% relative compaction from a bulk sample collected during the field investigation.

2 - The above reflect direct shear strength at saturated conditions.

3 - The tests were run at a shear rate of 0.35 in/min.

GEOTEK

EXPANSION INDEX TEST

(ASTM D4829)

Client:	Jurupa Valley USD	***Tested/ Checked By:***	BL Lab No Corona
Project Number:	3050-CR	***Date Tested:***	2/14/2022
Project Location:	High School Storage, Jurupa Valley	***Sample Source:***	B-1 @ 0-5 feet
		Sample Description:	

Ring #:______ Ring Dia. : 4.01" Ring Ht.:1"

DENSITY DETERMINATION

A	Weight of compacted sample & ring (gm)	778.4
B	Weight of ring (gm)	366.3
C	Net weight of sample (gm)	**412.1**
D	Wet Density, lb / ft3 (C*0.3016)	**124.3**
E	Dry Density, lb / ft3 (D/1.F)	**114.0**

SATURATION DETERMINATION

F	Moisture Content, %	9.0
G	Specific Gravity, assumed	**2.70**
H	Unit Wt. of Water @ 20 °C, (pcf)	**62.4**
I	% Saturation	**50.9**

READINGS

DATE	TIME	READING	
2/14/2022		0.6190	Initial
2/14/2022		0.6180	10 min/Dry
2/15/2022		0.6310	Final

FINAL MOISTURE

Final Weight of wet sample & tare	% Moisture
797.1	**13.5**

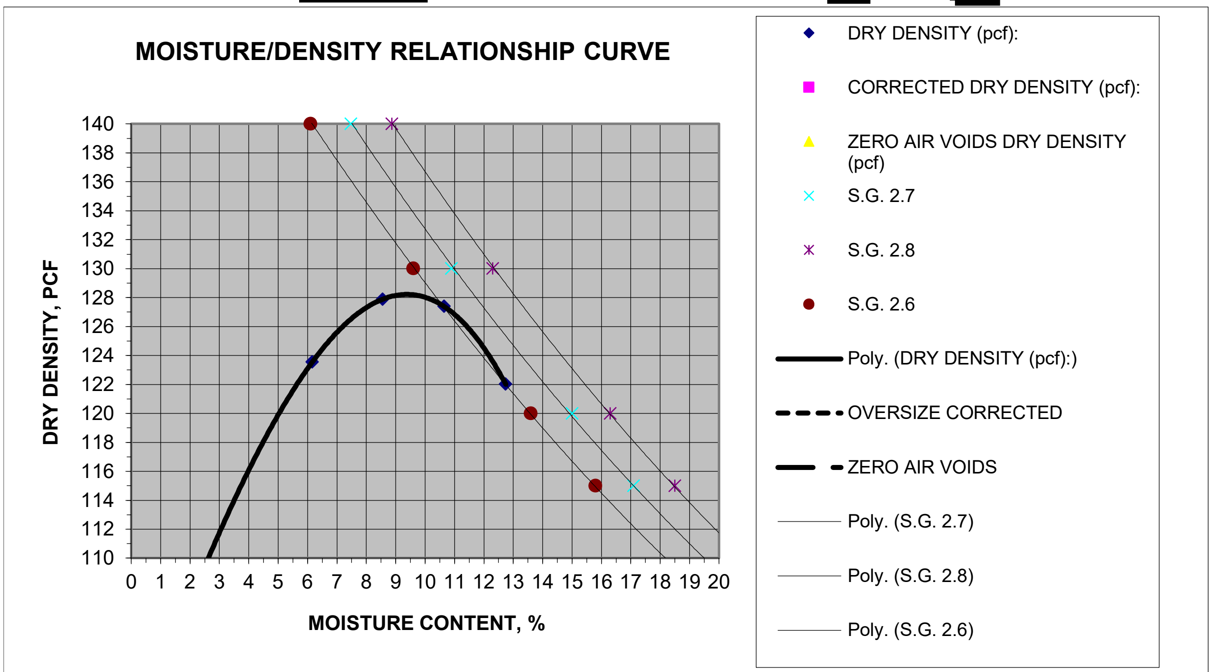
EXPANSION INDEX = 13

MOISTURE/DENSITY RELATIONSHIP

Client:	Jurupa Valley USD	**Job No.:**	3050-CR
Project:	High School Storage	**Lab No.:**	Corona
Location:	Jurupa Valley		
Material Type:	Yellow brown silty medium sand		
Material Supplier:	-		
Material Source:	-		
Sample Location:	B-1 @ 0-5 feet		
	-		
Sampled By:	DW	**Date Sampled:**	1/27/2022
Received By:	CB	**Date Received:**	1/27/2022
Tested By:	CB	**Date Tested:**	2/8/2022
Reviewed By:	DA	**Date Reviewed:**	2/15/2022

Test Procedure: ASTM D1557 **Method:** A

Oversized Material (%): 5.5 **Correction Required:** yes no



MOISTURE DENSITY RELATIONSHIP VALUES

Maximum Dry Density, pcf	128.0	**@ Optimum Moisture, %**	9.5
Corrected Maximum Dry Density, pcf		**@ Optimum Moisture, %**	

MATERIAL DESCRIPTION

Grain Size Distribution:

	% Gravel (retained on No. 4)
	% Sand (Passing No. 4, Retained on No. 200)
	% Silt and Clay (Passing No. 200)

Atterberg Limits:

	Liquid Limit, %
	Plastic Limit, %
	Plasticity Index, %

Classification:

Unified Soils Classification:

AASHTO Soils Classification:

Results Only Soil Testing for Jurupa Valley HS - Storage, Jurupa Valley

February 9, 2022

Prepared for:

Anna Scott
GeoTek, Inc.
1548 North Maple Street
Corona, CA 92280
ascott@geotekusa.com

Project X Job#: S220208J
Client Job or PO#: 3050-CR Jurupa Valley Unified School District

Respectfully Submitted,

Eduardo Hernandez, M.Sc., P.E.
Sr. Corrosion Consultant
NACE Corrosion Technologist #16592
Professional Engineer
California No. M37102
ehernandez@projectxcorrosion.com

REGISTERED PROFESSIONAL ENGINEER
EDUARDO HERNANDEZ
No. M37102
MECHANICAL
STATE OF CALIFORNIA

Soil Analysis Lab Results

Client: GeoTek, Inc.
Job Name: Jurupa Valley HS - Storage, Jurupa Valley
Client Job Number: 3050-CR Jurupa Valley Unified School District
Project X Job Number: S220208J
February 9, 2022

	Method	ASTM D4327		ASTM D4327		ASTM G187		ASTM G51	ASTM G200	SM 4500-D	ASTM D4327	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D4327	ASTM D4327
Bore# / Description	**Depth**	**Sulfates** SO_4^{2-}		**Chlorides** Cl^-		**Resistivity** As Rec'd	Minimum	**pH**	**Redox**	**Sulfide** S^{2-}	**Nitrate** NO_3^-	**Ammonium** NH_4^+	**Lithium** Li^+	**Sodium** Na^+	**Potassium** K^+	**Magnesium** Mg^{2+}	**Calcium** Ca^{2+}	**Fluoride** F_2^{--}	**Phosphate** PO_4^{3-}
	(ft)	(mg/kg)	(wt%)	(mg/kg)	(wt%)	(Ohm-cm)	(Ohm-cm)		(mV)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)
B-1	0-5	276.2	0.0276	57.4	0.0057	18,090	1,139	7.9	224	ND	607.9	2.1	ND	97.4	96.1	37.7	9.9	7.6	0.0

Cations and Anions, except Sulfide and Bicarbonate, tested with Ion Chromatography
mg/kg = milligrams per kilogram (parts per million) of dry soil weight
ND = 0 = Not Detected | NT = Not Tested | Unk = Unknown
Chemical Analysis performed on 1:3 Soil-To-Water extract
PPM = mg/kg (soil) = mg/L (Liquid)

APPENDIX C

SITE SPECIFIC GROUND-MOTION SEISMIC ANALYSIS

Jurupa Valley High School

Proposed Jurupa Unified School District Storage Project

Jurupa Valley, Riverside County, California

Project No. 3050-CR

GEOTEK

TERRA GEOSCIENCES

GROUND-MOTION SEISMIC ANALYSIS

PROPOSED STORAGE BUILDING PROJECT

JURUPA VALLEY HIGH SCHOOL

10551 BELLEGRAVE AVENUE

JURUPA VALLEY, CALIFORNIA

Project No. 223774-1

January 29, 2022

Prepared for:

GeoTek, Inc.
1548 North Maple Street
Corona, CA 92880

Consulting Engineering Geology & Geophysics

P.O. Box 1090, Loma Linda, CA 92354 • 909 796-4667

GeoTek, Inc.
1548 North Maple Street
Corona, CA 92880

Attention: Ms. Anna M. Scott, Project Geologist

Regarding: Ground-Motion Seismic Analysis
Proposed Storage Building Project
Jurupa Valley High School
10551 Bellegrave Avenue
Jurupa Valley, California
Geotek Project No. 3050-CR

INTRODUCTION

At your request, this firm has prepared a ground-motion seismic analysis report for the proposed storage building to be constructed within the existing school campus as referenced above. The purpose of this study was to evaluate the site-specific ground motion parameters to aid in the seismic design for this project, based on the current 2019 California Building Code (CBC). Our work included performing a seismic shear-wave study for determining the Site Classification and V_{S100} input values for this analysis. The scope of services provided for this evaluation included the following:

- ***Review of available published and unpublished geologic/seismic data in our files pertinent to the site.***
- ***Performing a seismic surface-wave survey by a licensed State of California Professional Geophysicist that included one traverse for shear-wave velocity analysis purposes.***
- ***Evaluation of the local and regional tectonic setting including performing a site-specific CBC ground motion analysis.***
- ***Preparation of this report presenting our findings, with respect to the seismic design parameters.***

Accompanying Maps and Appendices

Plate 1- Google™ Earth Imagery Map
Plate 2- Seismic Line Location Map
Appendix A - Shear-Wave Survey
Appendix B - Site Specific Ground Motion Analysis
Appendix C - References

PROJECT SUMMARY

Based on the information that has been provided, we understand that construction of a 7,500 square-foot storage building is proposed within the northeastern portion of the existing school campus. This building will be constructed of prefabricated metal and will be supported by a conventional shallow foundation and slab-on-grade floor system. For this project, we have performed a field reconnaissance, observed the exploratory boring excavations during the time of drilling, reviewed pertinent available geologic and geotechnical data in our files, along with performing a site-specific ground motion analysis and a seismic shear-wave survey.

To aid in providing applicable data for the site-specific ground motion analysis, a seismic shear-wave survey using the refraction microtremor method (REMI) was performed in order to assess the one-dimensional average shear-wave velocity structure beneath the subject site to a depth of at least 100 feet. This survey line was performed along the southwestern perimeter of the subject site (as shown on Plates 1 and 2), which provided the necessary survey line length along an accessible area, as well as being representative for the site development of the proposed building.

The resultant shear wave velocity (V_S) from this survey line within the upper 100 feet (30 meters) was then used to both determine the Site Classification (ASCE, 2017, Table 20.3-1) of the subject project study area, as well as being used for the V_S input value of the site-specific CBC seismic analysis. The detailed results of this survey are presented within Appendix A for reference.

Geologic mapping of the area by the Morton and Miller (2006) indicate that the subject site is surficially mantled by middle Pleistocene age old alluvial fan deposits. These surficial deposits are generally described as being comprised of moderately dissected interstratified sand and gravel.

The approximate location of the seismic shear-wave traverse (Seismic Line SW-1) is shown on a captured Google™ Earth (2022) image, as presented as the Google™ Earth Imagery Map, Plate 1. Additionally, the survey line is also shown on a partial copy of the provided Overall Site Plan (Sheet AS-2.0), prepared by Ruhnau Clarke Architects, as presented on the Seismic Line Location Map, Plate 2. The survey line was placed as close as practical to the proposed construction area as this location provided the necessary length needed for the survey traverse to ensure that shear-wave data down to 100-feet in depth would be obtained. Photographic views of the seismic line traverse have been included within Appendix A for both visual and reference purposes.

SITE-SPECIFIC GROUND MOTION ANALYSIS

As requested, we have performed a site-specific seismic ground motion analysis as discussed above. Geographically, the proposed development project is centrally located at Latitude 34.003 and Longitude -117.520 (World Geodetic System of 1984). The mapped spectral acceleration parameters, coefficients, and other related seismic parameters, were evaluated using the OSHPD Seismic Design Maps Tool web application (OSHPD, 2022) and the California Building Code criteria (CBC, 2019), with the site-specific ground motion analysis being performed following Section 21 of the ASCE 7-16 Standard (ASCE, 2017). The results of this site-specific ground motion analysis have been summarized and are tabulated below, with the detailed analysis being presented within Appendix B:

TABLE 1 – SUMMARY OF SITE-SPECIFIC SEISMIC DESIGN PARAMETERS

Factor or Coefficient	Value
S_S	**1.592g**
S_1	**0.581g**
F_a	**1.2**
F_v	**1.419**
S_{DS}	**1.120g**
S_{D1}	**1.240g**
S_{MS}	**1.682g**
S_{M1}	**1.859g**
T_L	**12 Seconds**
MCE_G PGA	**0.76g**
Shear-Wave Velocity (V_{100})	**1,337.0 ft/sec**
Site Classification	**C**
Risk Category	**III**

CLOSURE

Our conclusions and recommendations are based on an interpretation of available existing geologic, geophysical, geotechnical, and seismic data. No subsurface exploration was performed by this firm for this evaluation. We make no warranty, either express or implied. Should conditions be encountered at a later date or more information becomes available that appear to be different than those indicated in this report, we reserve the right to reevaluate our conclusions and recommendations and provide appropriate mitigation measures, if warranted. If this report is not understood, it is the responsibility of the owner, contractor, engineer, and/or governmental agency, etc., to contact this office for further clarification.

Respectfully submitted,
TERRA GEOSCIENCES

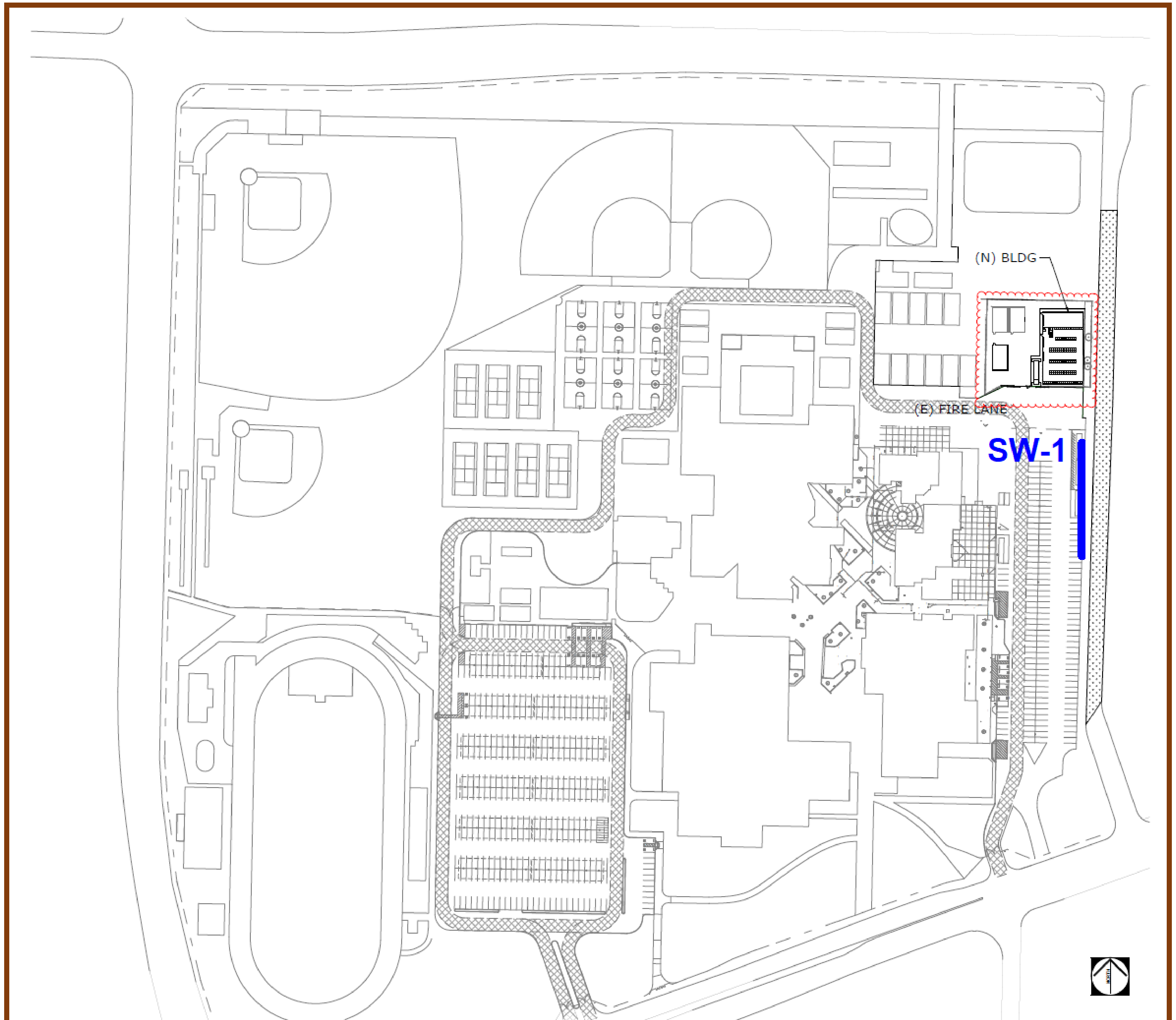
Donn C. Schwartzkopf
Certified Engineering Geologist
CEG 1459

Professional Geophysicist
PGP 1002

GOOGLE™ EARTH IMAGERY MAP

Base Map: Google™ Earth (2022); Project site area outlined in red; Seismic shear-wave survey line (SW-1) shown as yellow line.

SEISMIC LINE LOCATION MAP



Base Map: Partial modified copy of the Overall Site Plan (Sheet AS-2.0): Project site area outlined in red; Seismic shear-wave survey line (SW-1) shown as blue line.

APPENDIX A

SHEAR-WAVE SURVEY

SHEAR-WAVE SURVEY

Methodology

The fundamental premise of this survey uses the fact that the Earth is always in motion at various seismic frequencies. These relatively constant vibrations of the Earth's surface are called microtremors, which are very small with respect to amplitude and are generally referred to as background "noise" that contain abundant surface waves. These microtremors are caused by both human activity (i.e., cultural noise, traffic, factories, etc.) and natural phenomenon (i.e., wind, wave motion, rain, atmospheric pressure, etc.) which have now become regarded as useful signal information. Although these signals are generally very weak, the recording, amplification, and processing of these surface waves has greatly improved by the use of technologically improved seismic recording instrumentation and recently developed computer software. For this application, we are mainly concerned with the Rayleigh wave portion of the seismic signals, which is also referred to as "ground roll" since the Rayleigh wave is the dominant component of ground roll.

For the purposes of this study, there are two ways that the surface waves were recorded, one being "active" and the other being "passive." Active means that seismic energy is intentionally generated at a specific location relative to the survey spread and recording begins when the source energy is imparted into the ground (i.e., MASW survey technique). Passive surveying, also called "microtremor surveying," is where the seismograph records ambient background vibrations (i.e., MAM survey technique), with the ideal vibration sources being at a constant level. Longer wavelength surface waves (longer-period and lower-frequency) travel deeper and thus contain more information about deeper velocity structure and are generally obtained with passive survey information. Shorter wavelength (shorter-period and higher-frequency) surface waves travel shallower and thus contain more information about shallower velocity structure and are generally collected with the use of active sources. For the most part, higher frequency active source surface waves will resolve the shallower velocity structure and lower frequency passive source surface waves will better resolve the deeper velocity structure. Therefore, the combination of both of these surveying techniques provides a more accurate depiction of the subsurface velocity structure.

The assemblage of the data that is gathered from these surface wave surveys results in development of a dispersion curve. Dispersion, or the change in phase velocity of the seismic waves with frequency, is the fundamental property utilized in the analysis of surface wave methods. The fundamental assumption of these survey methods is that the signal wavefront is planar, stable, and isotropic (coming from all directions) making it independent of source locations and for analytical purposes uses the spatial autocorrelation method (SPAC). The SPAC method is based on theories that are able to detect "signals" from background "noise" (Okada, 2003). The shear wave velocity (V_S) can then be calculated by mathematical inversion of the dispersive phase velocity of the surface waves which can be significant in the presence of velocity layering, which is common in the near-surface environment.

Field Procedures

One seismic shear-wave survey traverse was performed at the site as approximated on the Google™ Earth Imagery Map and Seismic Line Location Map, Plates 1 and 2, respectively. For data collection, the field survey employed a twenty-four channel Geometrics StrataVisor™ NZXP model signal-enhancement refraction seismograph. This survey employed both active (MASW) and passive (MAM) source methods to ensure that both quality shallow and deeper shear-wave velocity information was recorded (Park et al., 2005). Both the MASW and MAM survey lines used the same linear geometry array that consisted of a 184-foot-long spread using a series of twenty-four 4.5-Hz geophones that were spaced at regular eight-foot intervals.

For the MASW survey, the ground vibrations were recorded using a one second record length at a sampling rate of 0.5-milliseconds. Two seismic records were obtained using a 30-foot offset from the beginning and end of the survey line utilizing a 16-pound sledge-hammer as the energy source to produce the seismic waves. Each of these shot points used multiple shots (stacking) to improve the signal to noise ratio of the data.

The MAM survey did not require the introduction of artificial seismic sources and only background ambient noise was recorded. The ambient ground vibrations were recorded using a thirty-two second record length at a two-millisecond sampling rate with 20 separate seismic records being obtained for quality control purposes. The seismic-wave forms and associated frequency spectrum that were displayed on the seismograph screen were used to assess the recorded seismic wave data for quality control purposes in the field. The acceptable records were digitally recorded on the in-board seismograph computer and subsequently transferred to a flash drive so that they could be subsequently transferred to our office computer for analysis.

Data Processing

For analysis and presentation of the shear-wave profile and supportive illustrations, this study used the SeisImager/SW™ computer software program developed by Geometrics, Inc. (2012). Both the active (MASW) and passive (MAM) survey results were combined for this analysis (Park et al., 2005). The combined results maximize the resolution and overall depth range in order to obtain one high resolution V_s curve over the entire sampled depth range. These methods economically and efficiently estimate one-dimensional subsurface shear-wave velocities using data collected from standard primary-wave (P-wave) refraction surveys, however, it should be noted that surface waves by their physical nature cannot resolve relatively abrupt or small-scale velocity anomalies.

Processing of the data proceeded by calculating the dispersion curve from the input data which subsequently created an initial shear-wave model based on the observed data. This initial model was then inverted in order to converge on the best fit of the initial model and the observed data, creating the final shear-wave model (Seismic Line SW-1) as presented within this appendix.

Data Analysis

Data acquisition went very smoothly and the quality was considered to be very good. The seismic model data indicates that the average shear-wave velocity beneath the survey traverse has numerous velocity layers, that all increase with depth. Analysis revealed that the average shear-wave velocity ("weighted average") in the upper 100 feet of the subject survey area is **1,337.0** feet per second (407.5 meters per second) as shown on the Shear-Wave Model for Seismic Line SW-1, as presented within this appendix. This average velocity classifies the underlying soils to that of Site Class "**C**" (Very Dense Soil and Soft Rock), which has a velocity range from 1,200 to 2,500 ft/sec (ASCE, 2017; Table 20.3-1).

The "weighted average" velocity is computed from a formula that is used by the ASCE (2017; Section 20.4, Equation 20.4-1) to determine the average shear-wave velocity for the upper 100 feet of the subsurface (V100).

$$Vs = 100/[(d1/v1) + (d2/v2) + ...+ (dn/vn)]$$

Where d1, d2, d3,...,tn, are the thicknesses for layers 1, 2, 3,...n, up to 100 feet, and v1, v2, v3,...,vn, are the seismic velocities (feet/second) for layers 1, 2, 3,...n. The detailed shear-wave model displays these calculated layer boundaries/depths and associated velocities (feet/second) for the 202-foot profile where locally measured. The constrained data is represented by the dark-gray shading on the shear-wave model. The associated Dispersion Curves (for both the active and passive methods) which show the data quality and picks, along with the resultant combined dispersion curve model, are also included within this appendix, for reference purposes.

Limitations

This survey was performed using "state of the art" geophysical equipment, techniques, and computer software. We make no warranty, either expressed or implied. It should be understood that when using these theoretical geophysical principles and techniques, sources of error are possible in both the data obtained and in the interpretation. Compared with traditional borehole shear-wave surveys of which use vertical body waves, the sources of error (if present) using horizontal surface waves for this project are not believed to be greater than 15 percent. It is also important to understand that the fundamental limitation for seismic surveys is known as nonuniqueness, wherein a specific seismic data set does not provide sufficient information to determine a single "true" earth model. Therefore, the interpretation of any seismic data set uses "best-fit" approximations along with the geologic models that appear to be most reasonable for the local area being surveyed.

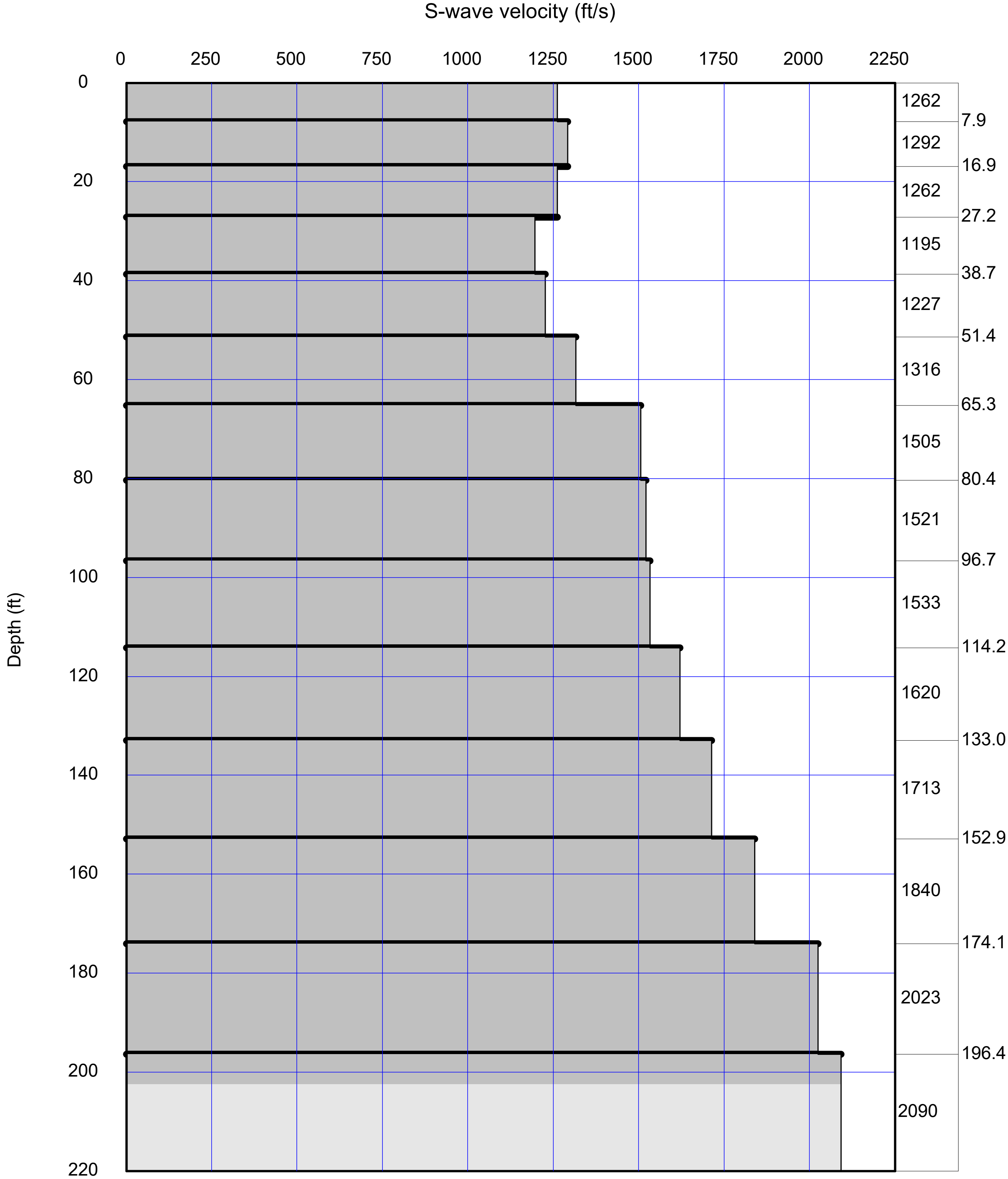
SHEAR-WAVE SURVEY LINE PHOTOGRAPHS

View looking south along Seismic Line SW-1.

View looking north along Seismic Line SW-1.

SEISMIC LINE SW-1

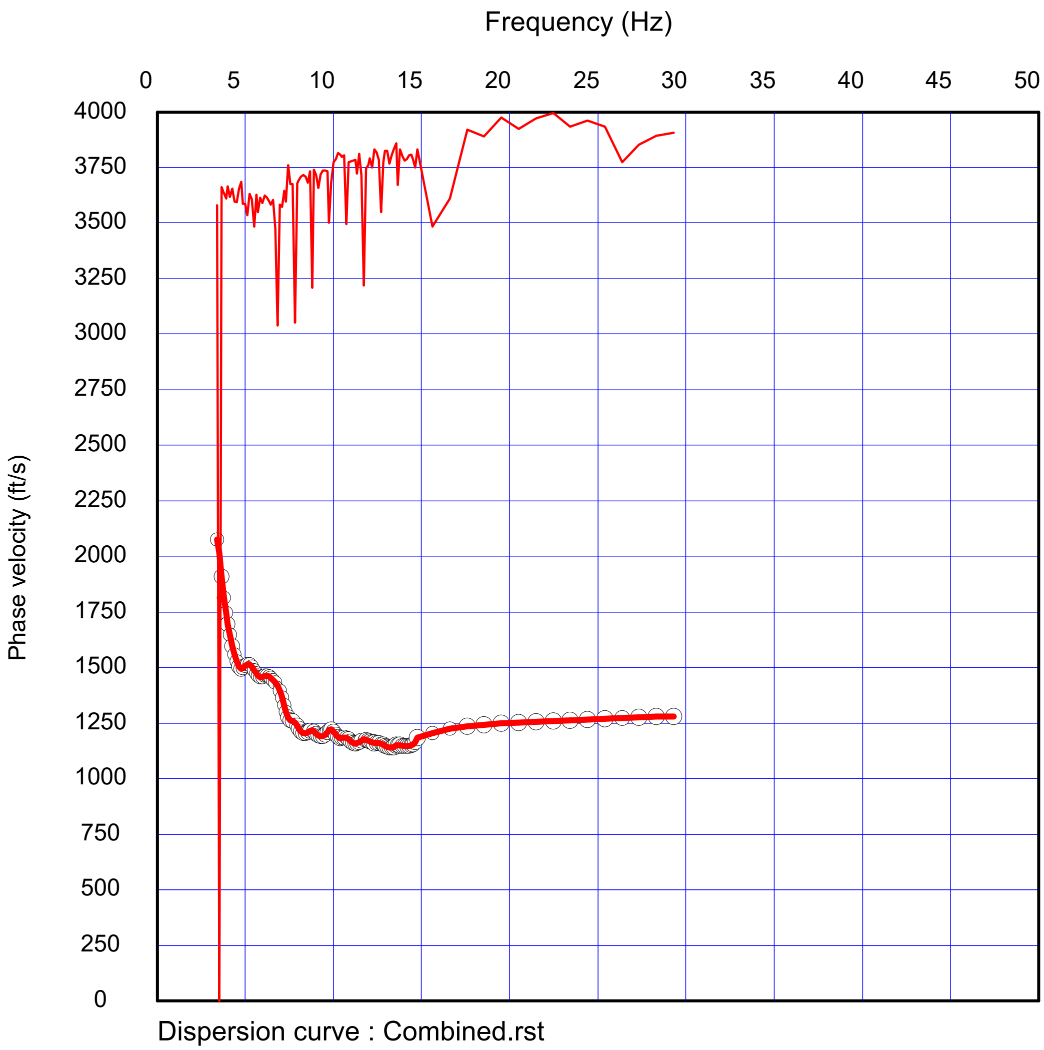
SHEAR-WAVE MODEL



S-wave velocity model (inverted) : Final.rst

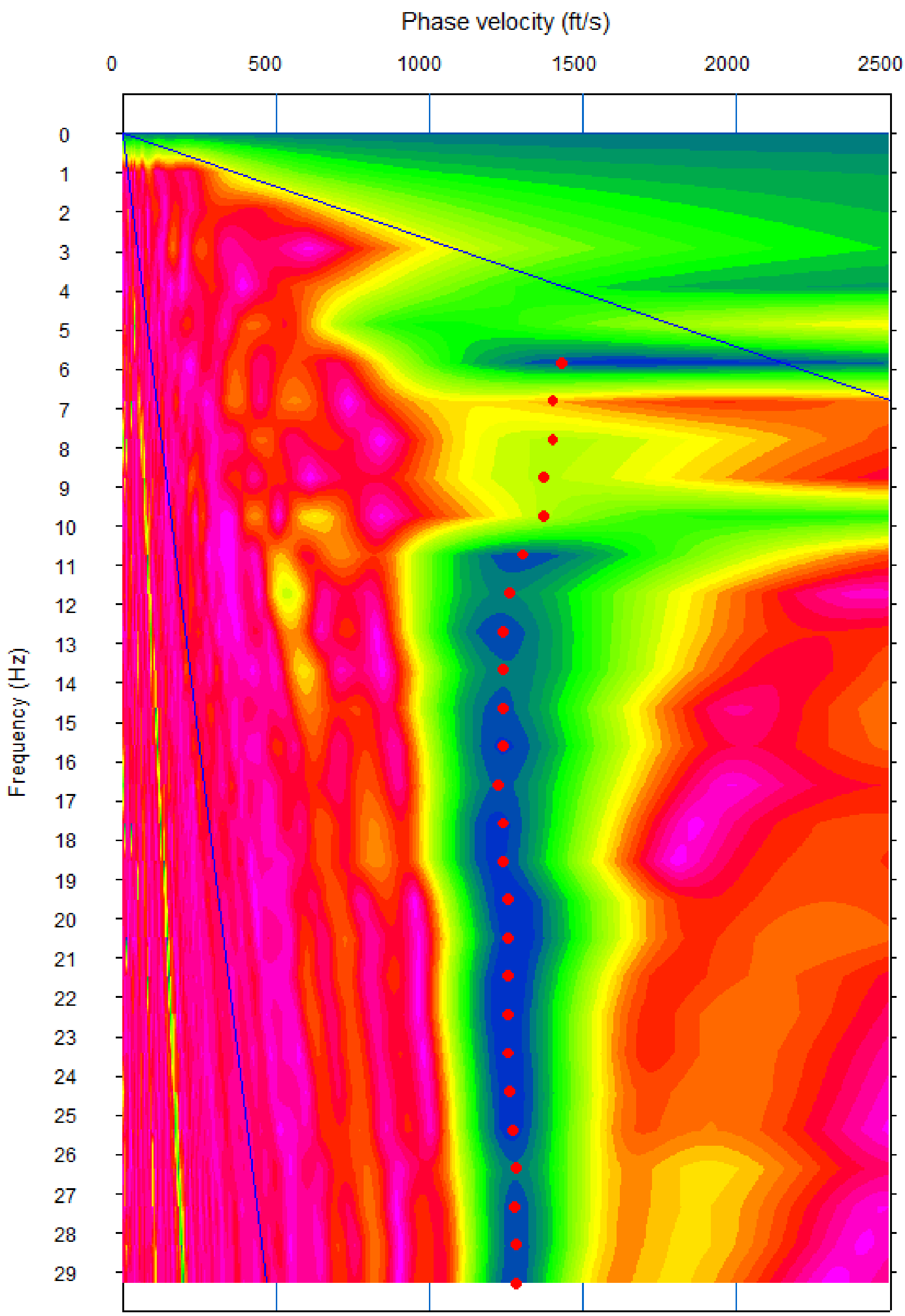
Average Vs 100ft = 1337.0 ft/sec

SHEAR-WAVE MODEL SW-1



COMBINED DISPERSION CURVE

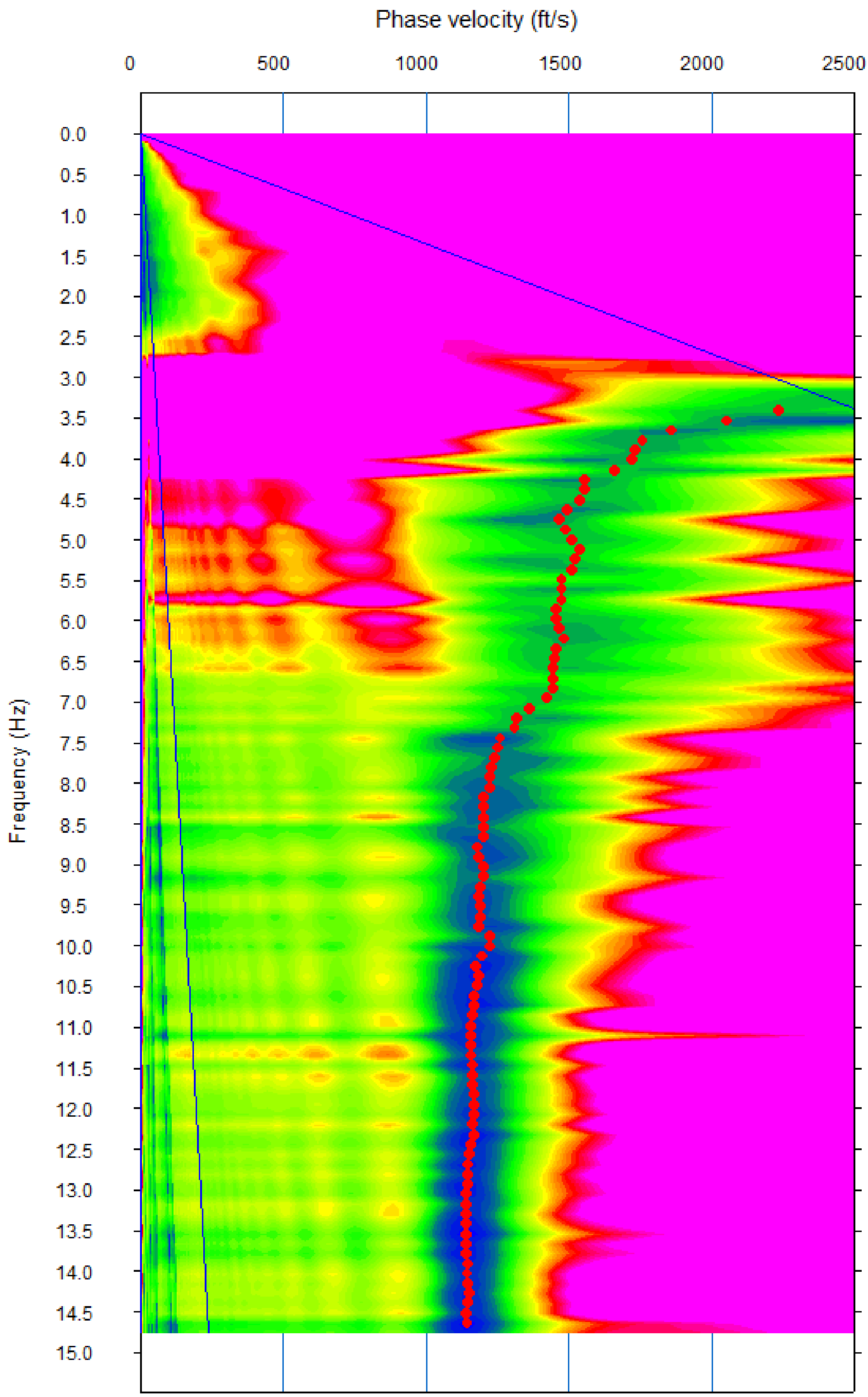
SEISMIC LINE SW-1



Dispersion Cure: Active.dat

ACTIVE DISPERSION CURVE

SEISMIC LINE SW-1



Dispersion Curve: Passive.dat

PASSIVE DISPERSION CURVE

APPENDIX B

SITE-SPECIFIC GROUND MOTION ANALYSIS

SITE-SPECIFIC GROUND MOTION ANALYSIS

A detailed summary of the site-specific ground motion analysis, which follows Section 21 of the ASCE Standard 7-16 (2017) and the 2019 California Building Code is presented below, with the Seismic Design Parameters Summary included within this appendix following the summary text.

- **Mapped Spectral Acceleration Parameters (CBC 1613A.2.1)**-

  Based on maps prepared by the U.S.G.S (Risk-Adjusted Maximum Considered Earthquake (MCE_R) Ground Motion Parameter for the Conterminous United States for the 0.2 and 1-second Spectral Response Acceleration (5% of Critical Damping; Site Class B/C), a value of **1.592g** for the 0.2 second period (S_S) and **0.581** for the 1.0 second period (S_1) was calculated (ASCE 7-16 Figures 22-1, 22-2 and CBC 1613A.2.1).

- **Site Classification (CBC 1613A.2.2 & ASCE 7-16 Chapter 20)**-

  Based on the site-specific measured shear-wave value of 1,337.0 feet/second (407.5 m/sec), the soil profile type used should be Site Class **"C."** This Class is defined as having the upper 100 feet (30 meters) of the subsurface being underlain by "Very Dense Soil and Soft Rock", with average shear-wave velocities of 1,200 to 2,500 feet/second (360 to 760 meters/second), as detailed within this appendix.

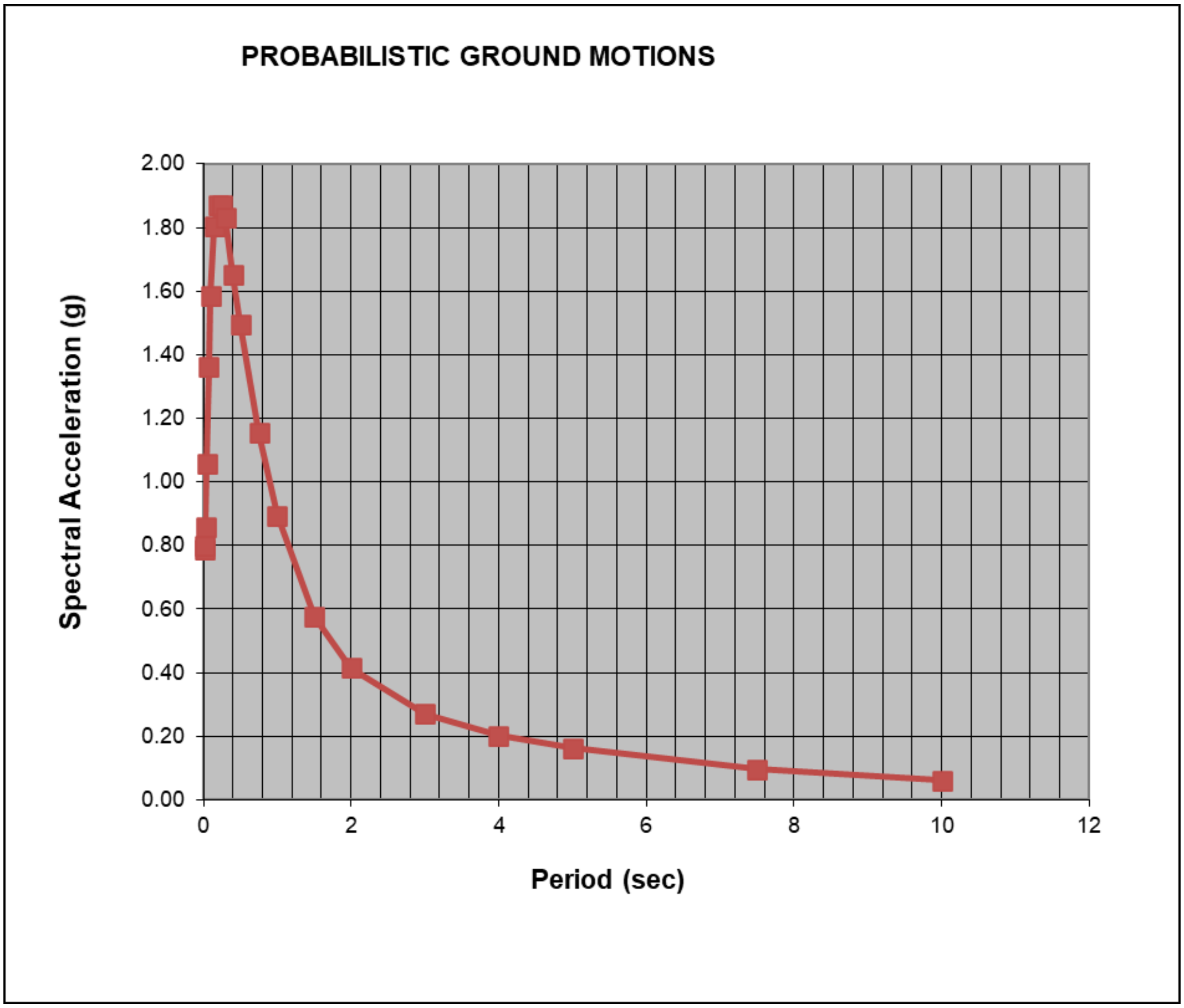
- **Site Coefficients (CBC 1613A.2.3)**-

  Based on CBC Tables 1613A.2.3(1) and 1613A.2.3(2), the site coefficient $\mathbf{F_a = 1.2}$ and $\mathbf{F_v = 1.419}$, respectively.

- **Probabilistic (MCE_R) Ground Motions (ASCE 7 Section 21.2.1.1)**-

  Per Section 21.2.1.1 (**Method 1**), the probabilistic MCE spectral accelerations shall be taken as the spectral response accelerations in the direction of maximum response represented by a five percent damped acceleration response spectrum that is expected to achieve a one percent probability of collapse within a 50-year period.

  The probabilistic analysis included the use of the Open Seismic Hazard Analysis (OpenSHA). The selected Earthquake Rupture Forecast (ERF) was UCERF3 along with a Probability of Exceedance of 2% in 50 Years. The average of four Next Generation Attenuation West-2 Relations (2014 NGA) were utilized to produce a response spectrum. These included Chiou & Youngs (2014), Abrahamsom et al. (2014), Boore et al. (2014) and Campbell & Bozorgnia (2014). The Probabilistic Risk Targeted Response Spectrum was determined as the product of the ordinates of the probabilistic response spectrum and the applicable risk coefficient (C_R). These values were then modified to produce a spectrum based upon the maximum rotated components of ground motion. The resulting MCE_R Response Spectrum is indicated below:



- ***Deterministic Spectral Response Analyses (ASCE 7 Section 21.2.2)***-

  The deterministic MCE_R response acceleration at each period shall be calculated as an 84th-percentile 5 percent damped spectral response acceleration in the direction of maximum horizontal response computed at that period. The largest such acceleration calculated for the characteristic earthquakes on all known active faults within the region shall be used. Analyses were conducted using the average of four Next Generation Attenuation West-2 Relations (2014 NGA), including Chiou & Youngs (2014), Abrahamsom et al. (2014), Boore et al. (2014) and Campbell & Bozorgnia (2014).

  Based on our review of the Fault Section Database within the Uniform California Earthquake Rupture Forecast (UCERF 3; Field et al., 2013) and other published geologic data and maps, the Elsinore Fault Zone (M_W 7.8), the Chino Fault (M_W 6.7), and the Fontana Seismic Trend (M_W 6.5), located at a distance of 18.6, 15.7. and 3.1 kilometers, respectively, were used for this analysis.

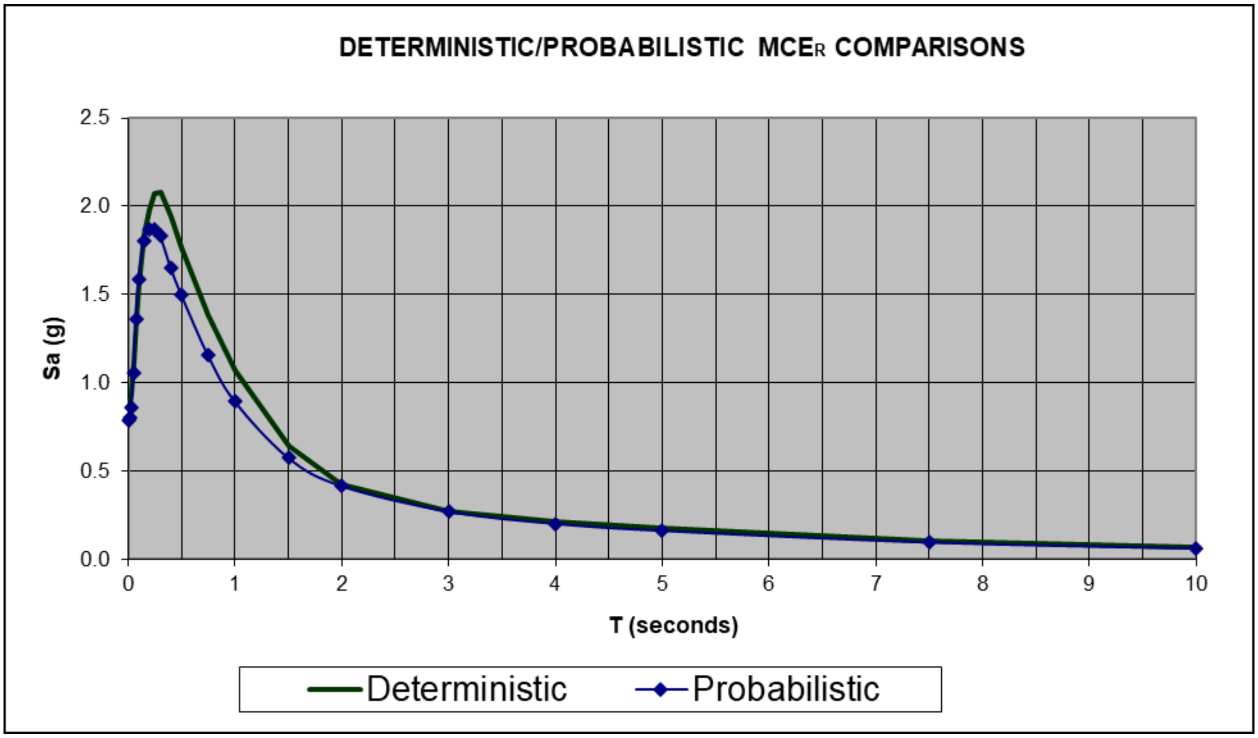
- **Site Specific MCE_R (ASCE 7 Section 21.2.3)**-

  The site-specific MCE_R spectral response acceleration at any period, S_{aM}, shall be taken as the lesser of the spectral response accelerations from the probabilistic ground motions of Section 21.2.1 and the deterministic ground motions of Section 21.2.2. The deterministic ground motions were compared with the probabilistic ground motions that were determined in accordance with Section 21.2.1.

Comparison of Deterministic MCE_R Values with Probabilistic MCE_R Values - Section 21.2.3

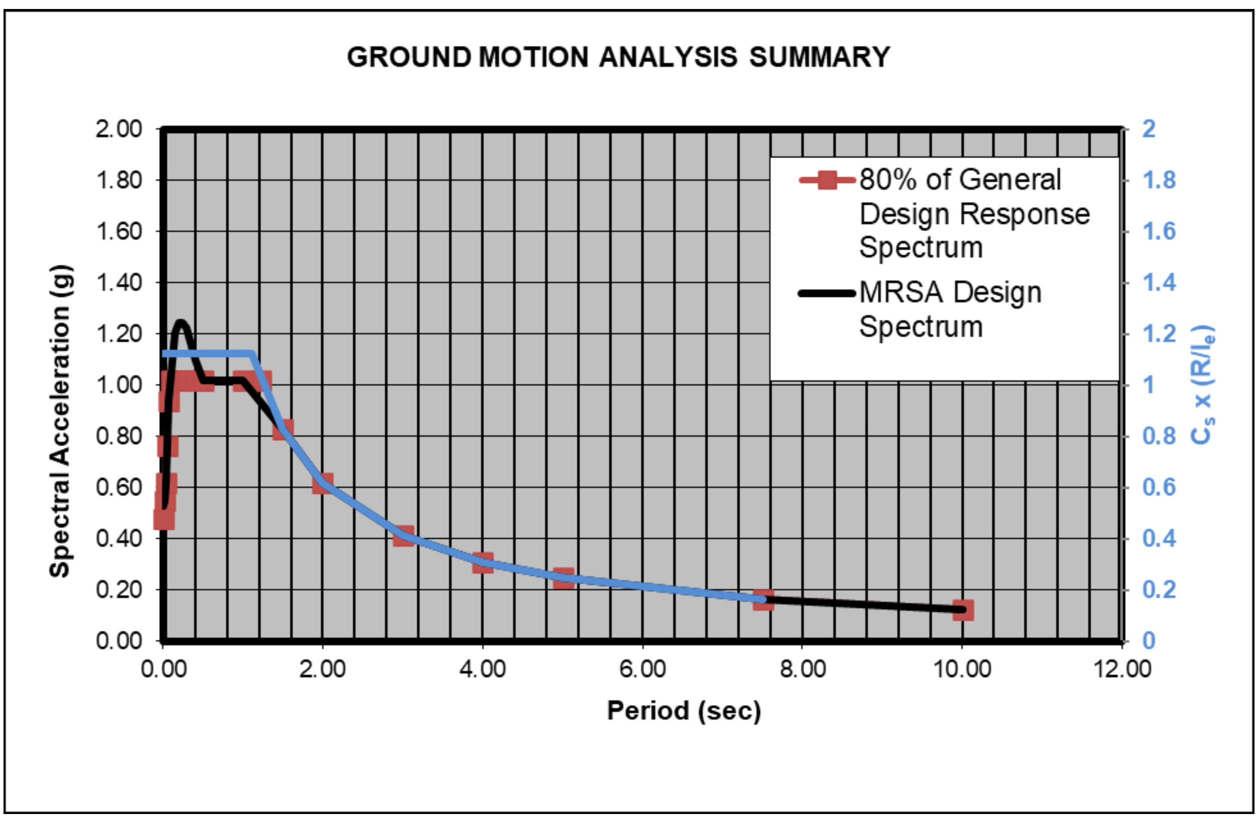
Period	Deterministic	Probabilistic	Lower Value	Governing Method
T	MCE_R	MCE_R	(Site Specific MCE_R)	
0.010	0.87	0.79	0.79	Probabilistic Governs
0.020	0.89	0.80	0.80	Probabilistic Governs
0.030	0.94	0.86	0.86	Probabilistic Governs
0.050	1.12	1.06	1.06	Probabilistic Governs
0.075	1.37	1.36	1.36	Probabilistic Governs
0.100	1.57	1.58	1.57	Deterministic Governs
0.150	1.84	1.80	1.80	Probabilistic Governs
0.200	1.98	1.87	1.87	Probabilistic Governs
0.250	2.07	1.87	1.87	Probabilistic Governs
0.300	2.08	1.83	1.83	Probabilistic Governs
0.400	1.94	1.65	1.65	Probabilistic Governs
0.500	1.76	1.50	1.50	Probabilistic Governs
0.750	1.38	1.15	1.15	Probabilistic Governs
1.000	1.07	0.89	0.89	Probabilistic Governs
1.500	0.64	0.58	0.58	Probabilistic Governs
2.000	0.43	0.41	0.41	Probabilistic Governs
3.000	0.27	0.27	0.27	Deterministic Governs
4.000	0.21	0.20	0.20	Probabilistic Governs
5.000	0.17	0.16	0.16	Probabilistic Governs
7.500	0.11	0.10	0.10	Probabilistic Governs
10.000	0.07	0.06	0.06	Probabilistic Governs

These are plotted in the following diagram:



- <u>**Design Response Spectrum (ASCE 7 Section 21.3)**</u>-

In accordance with Section 21.3, the Design Response Spectrum was developed by the following equation: $S_a = 2/3S_{aM}$, where S_{aM} is the MCE_R spectral response acceleration obtained from Section 21.1 or 21.2. The design spectral response acceleration shall not be taken less than 80 percent of S_a. These are plotted and compared with 80% of the CBC Spectrum values in the following diagram:



- **Design Acceleration Parameters (ASCE 7 Section 21.4)-**

  Where the site-specific procedure is used to determine the design ground motion in accordance with Section 21.3, the parameter S_{DS} shall obtained from the site-specific spectra at a period of 0.2 s, except that it shall not be taken less than 90 percent of the peak spectral acceleration, S_a, at any period larger than 0.2 s. The parameter S_{D1} shall be taken as the greater of the products of Sa * T for periods between 1 and 5 seconds. The parameters S_{MS}, and S_{M1} shall be taken as 1.5 times S_{DS} and S_{D1}, respectively. The values so obtained shall not be less than 80 percent of the values determined in accordance with Section 11.4.4 for S_{MS}, and S_{M1} and Section 11.4.5 for S_{DS} and S_{D1}.

- **Site Specific Design Parameters -**

  For the 0.2 second period (S_{DS}), a value of 1.120g was computed, based upon the average spectral accelerations. The maximum average acceleration for any period exceeding 0.2 seconds was 1.25g occurring at T=0.25 seconds. This was multiplied by 0.9 to produce a value of 1.120g making this the applicable value. A value of 1.240g was calculated for S_{D1} at a period of 1 second (ASCE 7-16, 21.4). For the MCE_R 0.2 second period, a value of 1.682g (S_{MS}) was computed, along with a value of 1.859g (S_{M1}) for the MCE_R 1.0 second period was also calculated (ASCE 7-16, 21.2.3).

- **Site-Specific MCE_G Peak Ground Accelerations (ASCE 7 Section 21.5)-**

  The probabilistic geometric mean peak ground acceleration (2 percent probability of exceedance within a 50-year period) was calculated as 0.76g. The deterministic geometric mean peak ground acceleration (largest 84th percentile geometric mean peak ground acceleration for characteristic earthquakes on all known active faults within the site region) was calculated as 0.79g. The site-specific MCE_G peak ground acceleration was calculated to be **0.76g**, which was determined by using the lesser of the probabilistic (0.76g) or the deterministic (0.79g) geometric mean peak ground accelerations, but not taken as less than 80 percent of PGA_M (i.e., 0.79g x 0.80 = 0.64g).

SEISMIC DESIGN PARAMETERS SUMMARY

Project: Jurupa Valley High School
Project #: 223774-1
Date: 1/28/22

Lattitude: 34.003
Longitude: -117.52

CALIFORNIA BUILDING CODE CHAPTER 16/ASCE7-16

Mapped Acceleration Parameters per ASCE 7-16, Chapter 22

S_s=	1.592	Figure 22-1
S_1=	0.581	Figure 22-2

Site Class per Table 20.3-1

Site Class=	C - Very Dense Soil and Soft Rock

Site Coefficients per ASCE 7-16 CHAPTER 11

F_a=	1.2	Table 11.4-1	=	1.2	For Site Specific Analysis per ASCE7-16 21.3
F_v=	1.419	Table 11.4-2	=	4.00	For Site Specific Analysis per ASCE7-16 21.3

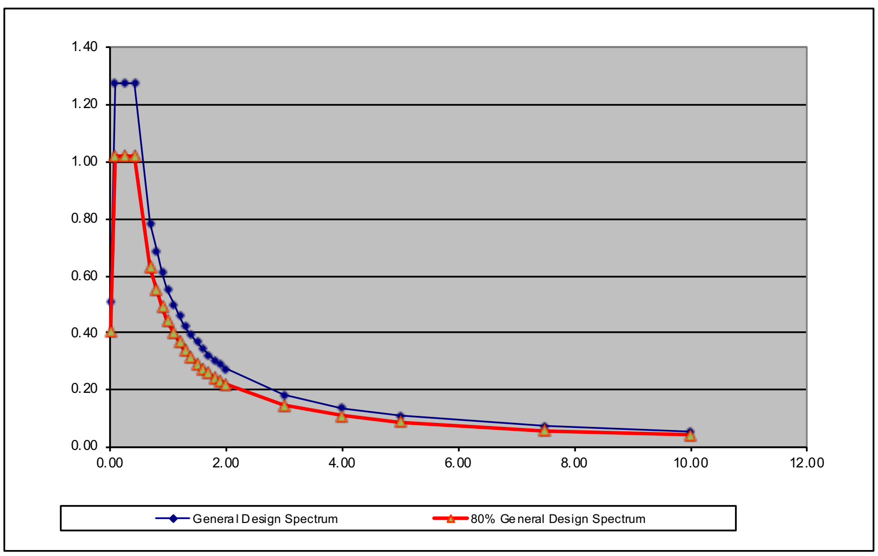
Mapped Design Spectral Response Acceleration Parameters

S_{MS}=	1.9104	Equation 11.4-1	1.9104	For Site Specific Analysis per ASCE7-16 21.3
S_{M1}=	0.824	Equation 11.4-2	2.324	For Site Specific Analysis per ASCE7-16 21.3

S_{DS}=	1.274	Equation 11.4-3
S_{D1}=	0.550	Equation 11.4-4

T_0=	0.086	sec	
T_S=	0.432	sec	
T_L=	12	sec	From Fig 22-12
PGA	0.662	g	
F_{PGA}=	1.2		From Table 11.8-1
C_{RS}=	0.943		Figure 22-17
C_{R1}=	0.918		Figure 22-18

Period (T)	Sa (ASCE7-16 - 11.4.6)	80% General Design Spectrum
0.01	0.51	0.41
0.09	1.27	1.02
0.27	1.27	1.02
0.43	1.27	1.02
0.70	0.79	0.63
0.80	0.69	0.55
0.90	0.61	0.49
1.00	0.55	0.44
1.10	0.50	0.40
1.20	0.46	0.37
1.30	0.42	0.34
1.40	0.39	0.31
1.50	0.37	0.29
1.60	0.34	0.27
1.70	0.32	0.26
1.80	0.31	0.24
1.90	0.29	0.23
2.00	0.27	0.22
3.00	0.18	0.15
4.00	0.14	0.11
5.00	0.11	0.09
7.50	0.07	0.06
10.00	0.05	0.04



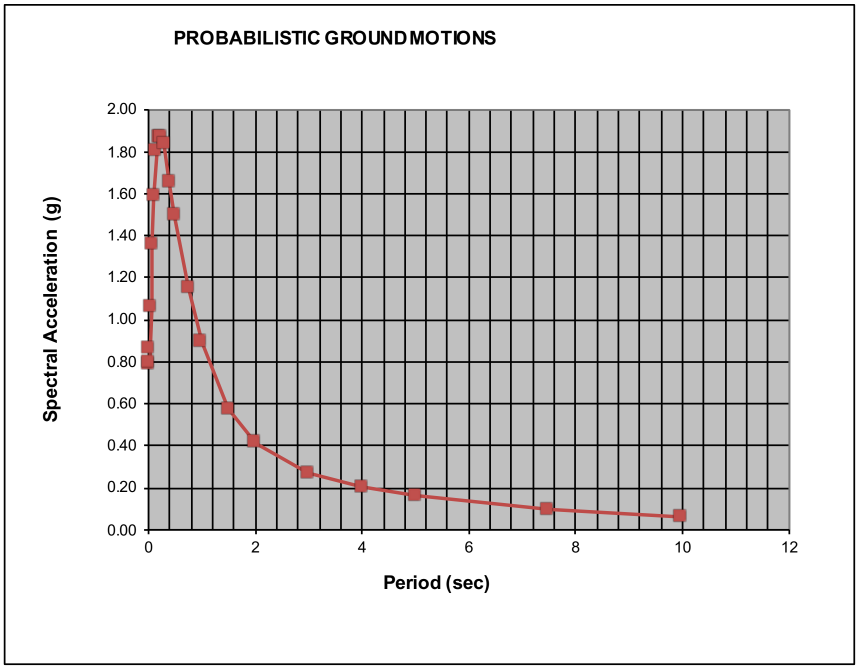
ASCE 7-16 - RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION ANALYSIS

Use Maximum Rotated Horizontal Component?* (Y/N) y

Presented data are the average of Chiou & Youngs (2014), Abrahamson et. al. (2014) , Boore et. al (2014) and Campbell & Bozorgnia (2014) NGA West-2 Relationships
Earthquake Rupture Forecast - UCERF3

PROBABILISTIC MCER per 21.2.1.1 **Method 1**
Risk Coefficients taken from Figures 22-18 and 22-19 of ASCE 7-16
OpenSHA data
2% Probability Of Exceedance in 50 years
Maximum Rotated Horizontal Component determined per ASCE7-16

T	Sa 2% in 50	MCER
0.01	0.84	0.79
0.02	0.85	0.80
0.03	0.91	0.86
0.05	1.12	1.06
0.08	1.44	1.36
0.10	1.68	1.58
0.15	1.91	1.80
0.20	1.98	1.87
0.25	1.99	1.87
0.30	1.95	1.83
0.40	1.76	1.65
0.50	1.60	1.50
0.75	1.25	1.15
1.00	0.97	0.89
1.50	0.63	0.58
2.00	0.45	0.41
3.00	0.30	0.27
4.00	0.22	0.20
5.00	0.18	0.16
7.50	0.10	0.10
10.00	0.07	0.06



S_s=	1.98	1.87
S_1=	0.97	0.89
PGA	0.76 g	

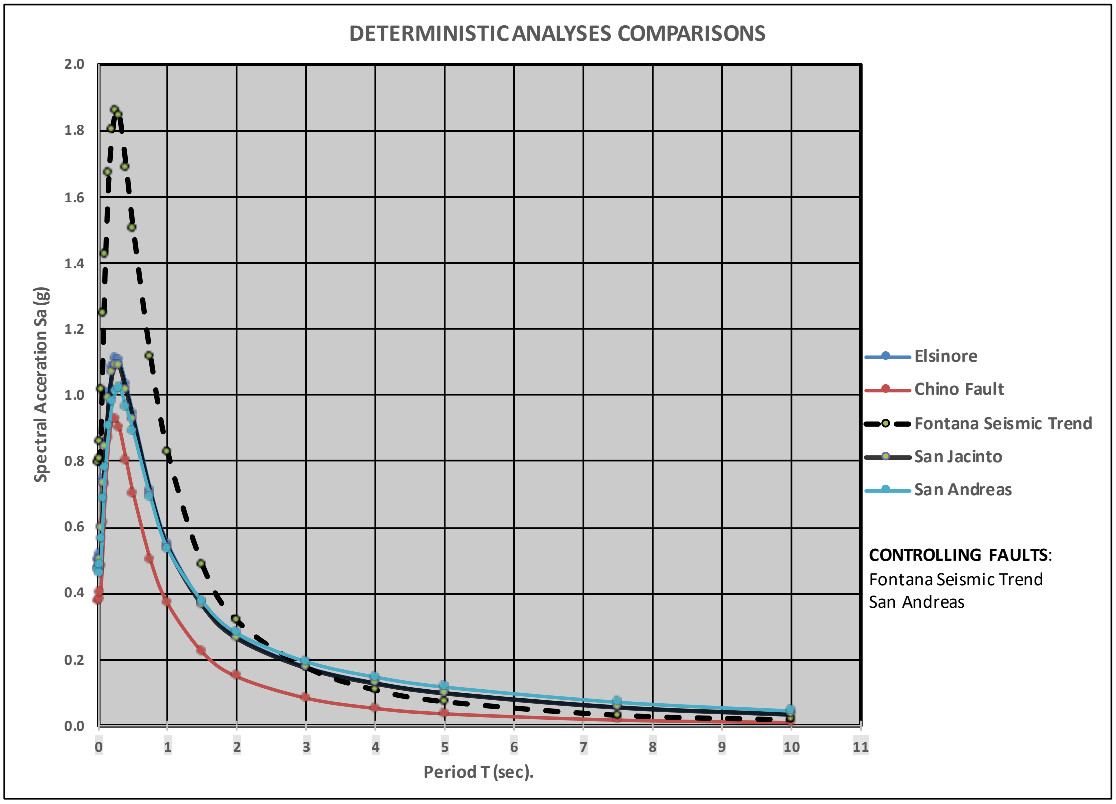
Risk Coefficients:			
C_{RS}	0.943	Figure 22-18	Get from Mapped Values
C_{R1}	0.918	Figure 22-19	
Fa=	1.2	Table 11.4-1	Per ASCE7-16 - 21.2.3
Is $Sa_{(max)}$<1.2XFa?		NO	If "YES", Probabilistic Spectrum prevails

DETERMINISTIC MCE per 21.2.2

Preliminary Assessment:

Five faults are appear to contribute to the seismic hazard at this site

Fault	Distance (km)
Elsinore	18.60
Chino	15.70
Fontana Seismic Trend	3.10
San Jacinto	19.05
San Andreas	27.80



Input Parameters		Elsinore	San Andreas	Fontana Seismic Trend	San Jacinto
Fault					
M	= Moment magnitude	7.8	8.3	6.5	7.8
R_{RUP}	= Closest distance to coseismic rupture (km)	18.6	27.8	3.1	19.05
R_{JB}	= Closest distance to surface projection of coseismic rupture (km)	18.6	27.8	3.1	19.05
Rx	= Horizontal distance to top edge of rupture measured perpendicular to strike (km)	18.6	27.8	3.1	19.05
U	= Unspecified Faulting Flag (Boore et.al.)	0	0	0	0
F_{RV}	= Reverse-faulting factor: 0 for strike slip, normal, normal-oblique; 1 for reverse, reverse-oblique and thrust	0	0	0	0
F_{NM}	= Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique and thrust; 1 for normal and normal-oblique	0	0	0	0
F_{HW}	= Hanging-wall factor: 1 for site on down-dip side of top of rupture; 0 otherwise, used in AS08 and CY08	0	0	0	0
Z_{TOR}	= Depth to top of coseismic rupture (km)	0	0	0	0
δ	= Average dip of rupture plane (degrees)	90	90	80	90
V_{S30}	= Average shear-wave velocity in top 30m of site profile	407.5	407.5	407.5	407.5
$F_{Measured}$		1	1	1	1
$Z_{1.0}$	= Depth to Shear Wave Velocity of 1.0 km/sec (km)	0.2	0.2	0.2	0.2
$Z_{2.5}$	= Depth to Shear Wave Velocity of 2.5 km/sec (km)	0.35	0.35	0.35	0.35
Site Class		C	C	C	C
W (km)	= Fault rupture width (km)	15	12.8	16.6	15.9
F_{AS}	= 0 for mainshock; 1 for aftershock	0	0	0	0
σ	=Standard Deviation	1	1	1	1

Deterministic Summary - Section 21.2.2 (Supplement 1)

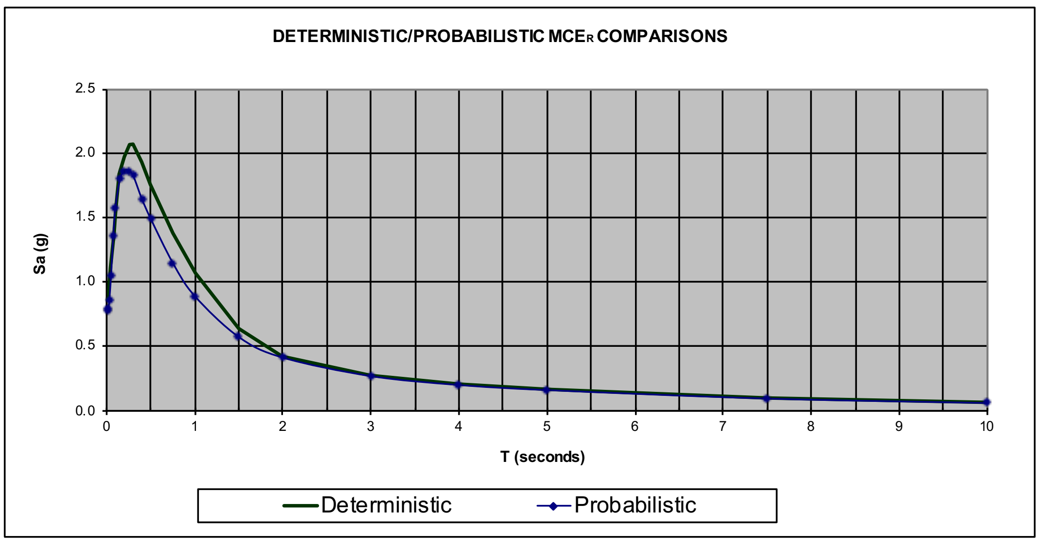
T	Fontana Seismic Trend	San Jacinto	San Andreas	Elsinore	Maximum S_a (Average)	Corrected* S_a (per ASCE7-16)	Scaled $S_{a(Average)}$	Controlling Fault
0.010	0.79	0.47	0.47	0.50	0.79	0.87	0.87	Seismic Trend
0.020	0.81	0.48	0.46	0.48	0.81	0.89	0.89	Seismic Trend
0.030	0.86	0.50	0.48	0.51	0.86	0.94	0.94	Seismic Trend
0.050	1.01	0.59	0.56	0.60	1.01	1.12	1.12	Seismic Trend
0.075	1.24	0.73	0.69	0.74	1.24	1.37	1.37	Seismic Trend
0.100	1.42	0.84	0.78	0.85	1.42	1.57	1.57	Seismic Trend
0.150	1.67	0.99	0.90	1.00	1.67	1.84	1.84	Seismic Trend
0.200	1.80	1.07	0.98	1.08	1.80	1.98	1.98	Seismic Trend
0.250	1.86	1.09	1.01	1.11	1.86	2.07	2.07	Seismic Trend
0.300	1.85	1.09	1.02	1.10	1.85	2.08	2.08	Seismic Trend
0.400	1.69	1.01	0.96	1.03	1.69	1.94	1.94	Seismic Trend
0.500	1.50	0.92	0.89	0.94	1.50	1.76	1.76	Seismic Trend
0.750	1.12	0.70	0.69	0.71	1.12	1.38	1.38	Seismic Trend
1.000	0.82	0.54	0.53	0.55	0.82	1.07	1.07	Seismic Trend
1.500	0.48	0.36	0.37	0.37	0.48	0.64	0.64	Seismic Trend
2.000	0.32	0.26	0.28	0.27	0.32	0.43	0.43	Seismic Trend
3.000	0.18	0.17	0.19	0.18	0.19	0.27	0.27	San Andreas
4.000	0.11	0.13	0.15	0.13	0.15	0.21	0.21	San Andreas
5.000	0.07	0.10	0.12	0.10	0.12	0.17	0.17	San Andreas
7.500	0.03	0.06	0.07	0.06	0.07	0.11	0.11	San Andreas
10.000	0.02	0.03	0.04	0.03	0.04	0.07	0.07	San Andreas
PGA	0.79	0.47	0.45	0.48	0.79		0.79	g
Max Sa=	2.08							
Fa =	#N/A	Per ASCE7-16 21.2.2						
1.5XFa=	1.8							
Scaling Factor=	1.00							

* Correction is the adjustment for Maximum Rotated Value if Applicable

SITE SPECIFIC MCE_R - Compare Deterministic MCE_R Values (S_a) with Probabilistic MCE_R Values (S_a) per 21.2.3

Presented data are the average of Chiou & Youngs (2014), Abrahamson et. al. (2014) , Boore et. al (2014) and Campbell & Bozorgnia (2014) NGA West-2 Relationships

Period T	Deterministic MCE_R	Probabilistic MCE_R	Lower Value (Site Specific MCE_R)	Governing Method
0.010	0.87	0.79	0.79	ProbabilisticGoverns
0.020	0.89	0.80	0.80	ProbabilisticGoverns
0.030	0.94	0.86	0.86	ProbabilisticGoverns
0.050	1.12	1.06	1.06	ProbabilisticGoverns
0.075	1.37	1.36	1.36	ProbabilisticGoverns
0.100	1.57	1.58	1.57	Deterministic Governs
0.150	1.84	1.80	1.80	ProbabilisticGoverns
0.200	1.98	1.87	1.87	ProbabilisticGoverns
0.250	2.07	1.87	1.87	ProbabilisticGoverns
0.300	2.08	1.83	1.83	ProbabilisticGoverns
0.400	1.94	1.65	1.65	ProbabilisticGoverns
0.500	1.76	1.50	1.50	ProbabilisticGoverns
0.750	1.38	1.15	1.15	ProbabilisticGoverns
1.000	1.07	0.89	0.89	ProbabilisticGoverns
1.500	0.64	0.58	0.58	ProbabilisticGoverns
2.000	0.43	0.41	0.41	ProbabilisticGoverns
3.000	0.27	0.27	0.27	Deterministic Governs
4.000	0.21	0.20	0.20	ProbabilisticGoverns
5.000	0.17	0.16	0.16	ProbabilisticGoverns
7.500	0.11	0.10	0.10	ProbabilisticGoverns
10.000	0.07	0.06	0.06	ProbabilisticGoverns



DESIGN RESPONSE SPECTRUM per Section 21.3

DESIGN ACCELERATION PARAMETERS per Section 21.4 (MRSA)

Period	2/3*MCE_R	80% General Design Response Spectrum (per ASCE 7-16 23.3-1)	Design Response Spectrum	TXSa
0.01	0.53	0.48	0.53	
0.02	0.53	0.55	0.55	
0.03	0.57	0.62	0.62	
0.05	0.70	0.76	0.76	
0.08	0.91	0.94	0.94	
0.10	1.04	1.02	1.04	
0.15	1.20	1.02	1.20	
0.20	1.25	1.02	1.25	
0.25	1.25	1.02	1.25	
0.30	1.22	1.02	1.22	
0.40	1.10	1.02	1.10	
0.50	1.00	1.02	1.02	
0.75	0.77	1.02	1.02	
1.00	0.60	1.02	1.02	1.02
1.50	0.38	0.83	0.83	1.24
2.00	0.28	0.62	0.62	1.24
3.00	0.18	0.41	0.41	
4.00	0.13	0.31	0.31	
5.00	0.11	0.25	0.25	
7.50	0.06	0.17	0.17	
10.00	0.04	0.12	0.12	

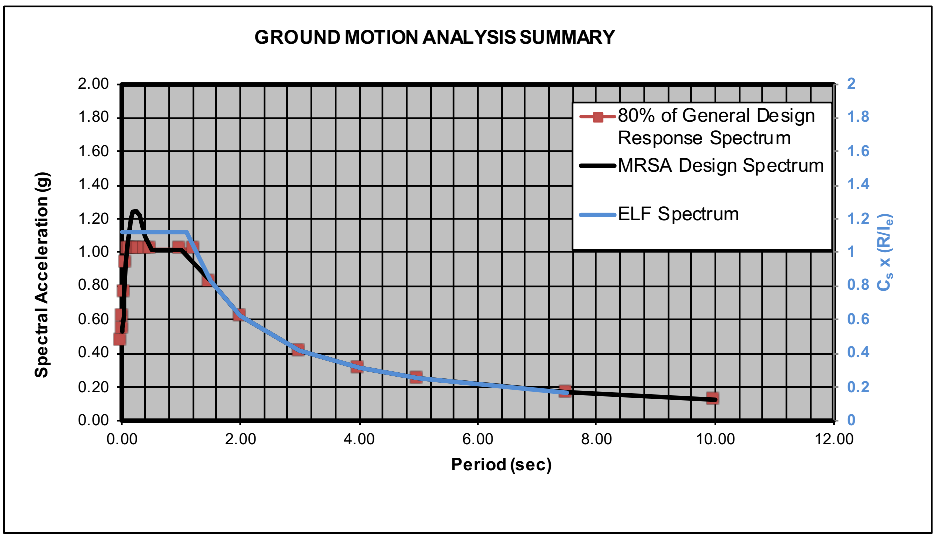
Highest value of S_a for any period exceeding 0.2 sec.=	1.25
90%of Highest Value =	1.12
80% 0f Mapped S_{DS}=	1.02
<u>Max TXsa from T=1s-2s =</u>	1.24
80% of Mapped S_{D1}=	0.44

S_{DS}=	1.12
S_{D1}=	1.24
T_S =	1.11

S_{MS}=	1.682
S_{M1}=	1.859

PGA Determination:

Site Coefficient F_{PGA}=	1.2	
Mapped PGA=	0.66	Figure 22-7
PGA_M =	0.79	g
Deterministic PGA =	0.79	g
Probabilistic PGA =	0.76	g
Lesser of Deterministic/Probabilistic =	0.76	g
80% of $PGA_{M=}$	0.64	g
MCE_G PGA=	0.76	g



APPENDIX C

REFERENCES

REFERENCES

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APPENDIX D

GENERAL GRADING GUIDELINES

Jurupa Valley High School
Proposed Jurupa Unified School District Storage Project
Jurupa Valley, Riverside County, California
Project No. 3050-CR

GEOTEK

GENERAL GRADING GUIDELINES

Guidelines presented herein are intended to address general construction procedures for earthwork construction. Specific situations and conditions often arise which cannot reasonably be discussed in general guidelines, when anticipated these are discussed in the text of the report. Often unanticipated conditions are encountered which may necessitate modification or changes to these guidelines. It is our hope that these will assist the contractor to more efficiently complete the project by providing a reasonable understanding of the procedures that would be expected during earthwork and the testing and observation used to evaluate those procedures.

General

Grading should be performed to at least the minimum requirements of governing agencies, Chapters 18 and 33 of the Uniform Building Code, CBC (2019) and the guidelines presented below.

Preconstruction Meeting

A preconstruction meeting should be held prior to site earthwork. Any questions the contractor has regarding our recommendations, general site conditions, apparent discrepancies between reported and actual conditions and/or differences in procedures the contractor intends to use should be brought up at that meeting. The contractor (including the main onsite representative) should review our report and these guidelines in advance of the meeting. Any comments the contractor may have regarding these guidelines should be brought up at that meeting.

Grading Observation and Testing

1. Observation of the fill placement should be provided by our representative during grading. Verbal communication during the course of each day will be used to inform the contractor of test results. The contractor should receive a copy of the "Daily Field Report" indicating results of field density tests that day. If our representative does not provide the contractor with these reports, our office should be notified.
2. Testing and observation procedures are, by their nature, specific to the work or area observed and location of the tests taken, variability may occur in other locations. The contractor is responsible for the uniformity of the grading operations; our observations and test results are intended to evaluate the contractor's overall level of efforts during grading. The contractor's personnel are the only individuals participating in all aspect of site work. Compaction testing and observation should not be considered as relieving the contractor's responsibility to properly compact the fill.
3. Cleanouts, processed ground to receive fill, key excavations, and subdrains should be observed by our representative prior to placing any fill. It will be the contractor's responsibility to notify our representative or office when such areas are ready for observation.

4. Density tests may be made on the surface material to receive fill, as considered warranted by this firm.
5. In general, density tests would be made at maximum intervals of two feet of fill height or every 1,000 cubic yards of fill placed. Criteria will vary depending on soil conditions and size of the fill. More frequent testing may be performed. In any case, an adequate number of field density tests should be made to evaluate the required compaction and moisture content is generally being obtained.
6. Laboratory testing to support field test procedures will be performed, as considered warranted, based on conditions encountered (e.g. change of material sources, types, etc.) Every effort will be made to process samples in the laboratory as quickly as possible and in progress construction projects are our first priority. However, laboratory workloads may cause in delays and some soils may require a **minimum of 48 to 72 hours to complete test procedures**. Whenever possible, our representative(s) should be informed in advance of operational changes that might result in different source areas for materials.
7. Procedures for testing of fill slopes are as follows:
   a) Density tests should be taken periodically during grading on the flat surface of the fill, three to five feet horizontally from the face of the slope.
   b) If a method other than over building and cutting back to the compacted core is to be employed, slope compaction testing during construction should include testing the outer six inches to three feet in the slope face to determine if the required compaction is being achieved.
8. Finish grade testing of slopes and pad surfaces should be performed after construction is complete.

Site Clearing

1. All vegetation, and other deleterious materials, should be removed from the site. If material is not immediately removed from the site it should be stockpiled in a designated area(s) well outside of all current work areas and delineated with flagging or other means. Site clearing should be performed in advance of any grading in a specific area.
2. Efforts should be made by the contractor to remove all organic or other deleterious material from the fill, as even the most diligent efforts may result in the incorporation of some materials. This is especially important when grading is occurring near the natural grade. All equipment operators should be aware of these efforts. Laborers may be required as root pickers.
3. Nonorganic debris or concrete may be placed in deeper fill areas provided the procedures used are observed and found acceptable by our representative.

Treatment of Existing Ground

1. Following site clearing, all surficial deposits of alluvium and colluvium as well as weathered or creep effected bedrock, should be removed unless otherwise specifically indicated in the text of this report.
2. In some cases, removal may be recommended to a specified depth (e.g. flat sites where partial alluvial removals may be sufficient). The contractor should not exceed these depths unless directed otherwise by our representative.
3. Groundwater existing in alluvial areas may make excavation difficult. Deeper removals than indicated in the text of the report may be necessary due to saturation during winter months.
4. Subsequent to removals, the natural ground should be processed to a depth of six inches, moistened to near optimum moisture conditions and compacted to fill standards.
5. Exploratory back hoe or dozer trenches still remaining after site removal should be excavated and filled with compacted fill if they can be located.

Fill Placement

1. Unless otherwise indicated, all site soil and bedrock may be reused for compacted fill; however, some special processing or handling may be required (see text of report).
2. Material used in the compacting process should be evenly spread, moisture conditioned, processed, and compacted in thin lifts six (6) to eight (8) inches in compacted thickness to obtain a uniformly dense layer. The fill should be placed and compacted on a nearly horizontal plane, unless otherwise found acceptable by our representative.
3. If the moisture content or relative density varies from that recommended by this firm, the contractor should rework the fill until it is in accordance with the following:
   a) Moisture content of the fill should be at or above optimum moisture. Moisture should be evenly distributed without wet and dry pockets. Pre-watering of cut or removal areas should be considered in addition to watering during fill placement, particularly in clay or dry surficial soils. The ability of the contractor to obtain the proper moisture content will control production rates.
   b) Each six-inch layer should be compacted to at least 90 percent of the maximum dry density in compliance with the testing method specified by the controlling governmental agency. In most cases, the testing method is ASTM Test Designation D 1557.
4. Rock fragments less than eight inches in diameter may be utilized in the fill, provided:
   a) They are not placed in concentrated pockets;
   b) There is a sufficient percentage of fine-grained material to surround the rocks;
   c) The distribution of the rocks is observed by, and acceptable to, our representative.

5. Rocks exceeding eight (8) inches in diameter should be taken off site, broken into smaller fragments, or placed in accordance with recommendations of this firm in areas designated suitable for rock disposal. On projects where significant large quantities of oversized materials are anticipated, alternate guidelines for placement may be included. If significant oversize materials are encountered during construction, these guidelines should be requested.
6. In clay soil, dry or large chunks or blocks are common. If in excess of eight (8) inches minimum dimension, then they are considered as oversized. Sheepsfoot compactors or other suitable methods should be used to break up blocks. When dry, they should be moisture conditioned to provide a uniform condition with the surrounding fill.

Slope Construction

1. The contractor should obtain a minimum relative compaction of 90 percent out to the finished slope face of fill slopes. 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.
2. Slopes trimmed to the compacted core should be overbuilt by at least three (3) feet with compaction efforts out to the edge of the false slope. Failure to properly compact the outer edge results in trimming not exposing the compacted core and additional compaction after trimming may be necessary.
3. If fill slopes are built "at grade" using direct compaction methods, then the slope construction should be performed so that a constant gradient is maintained throughout construction. Soil should not be "spilled" over the slope face nor should slopes be "pushed out" to obtain grades. Compaction equipment should compact each lift along the immediate top of slope. Slopes should be back rolled or otherwise compacted at approximately every 4 feet vertically as the slope is built.
4. Corners and bends in slopes should have special attention during construction as these are the most difficult areas to obtain proper compaction.
5. Cut slopes should be cut to the finished surface. Excessive undercutting and smoothing of the face with fill may necessitate stabilization.

UTILITY TRENCH CONSTRUCTION AND BACKFILL

Utility trench excavation and backfill is the contractors responsibility. The geotechnical consultant typically provides periodic observation and testing of these operations. While efforts are made to make sufficient observations and tests to verify that the contractors' methods and procedures are adequate to achieve proper compaction, it is typically impractical to observe all backfill procedures. As such, it is critical that the contractor use consistent backfill procedures.

Compaction methods vary for trench compaction and experience indicates many methods can be successful. However, procedures that "worked" on previous projects may or may not prove effective on a given site. The contractor(s) should outline the procedures proposed, so that we may discuss them **prior** to construction. We will offer comments based on our knowledge of site conditions and experience.

1. Utility trench backfill in slopes, structural areas, in streets and beneath flat work or hardscape should be brought to at least optimum moisture and compacted to at least 90 percent of the laboratory standard. Soil should be moisture conditioned prior to placing in the trench.
2. Flooding and jetting are not typically recommended or acceptable for native soils. Flooding or jetting may be used with select sand having a Sand Equivalent (SE) of 30 or higher. This is typically limited to the following uses:
   a) shallow (12 + inches) under slab interior trenches and,
   b) as bedding in pipe zone.

   The water should be allowed to dissipate prior to pouring slabs or completing trench compaction.
3. Care should be taken not to place soils at high moisture content within the upper three feet of the trench backfill in street areas, as overly wet soils may impact subgrade preparation. Moisture may be reduced to 2% below optimum moisture in areas to be paved within the upper three feet below sub grade.
4. Sand backfill should not be allowed in exterior trenches adjacent to and within an area extending below a 1:1 projection from the outside bottom edge of a footing, unless it is similar to the surrounding soil.
5. Trench compaction testing is generally at the discretion of the geotechnical consultant. Testing frequency will be based on trench depth and the contractors procedures. A probing rod would be used to assess the consistency of compaction between tested areas and untested areas. If zones are found that are considered less compact than other areas, this would be brought to the contractors attention.

JOB SAFETY

General

Personnel safety is a primary concern on all job sites. The following summaries are safety considerations for use by all our employees on multi-employer construction sites. On ground personnel are at highest risk of injury and possible fatality on grading construction projects. The company recognizes that construction activities will vary on each site and that job site safety is the contractor's responsibility. However, it is, imperative that all personnel be safety conscious to avoid accidents and potential injury.

In an effort to minimize risks associated with geotechnical testing and observation, the following precautions are to be implemented for the safety of our field personnel on grading and construction projects.

1. Safety Meetings: Our field personnel are directed to attend the contractor's regularly scheduled safety meetings.
2. Safety Vests: Safety vests are provided for and are to be worn by our personnel while on the job site.
3. Safety Flags: Safety flags are provided to our field technicians; one is to be affixed to the vehicle when on site, the other is to be placed atop the spoil pile on all test pits.

In the event that the contractor's representative observes any of our personnel not following the above, we request that it be brought to the attention of our office.

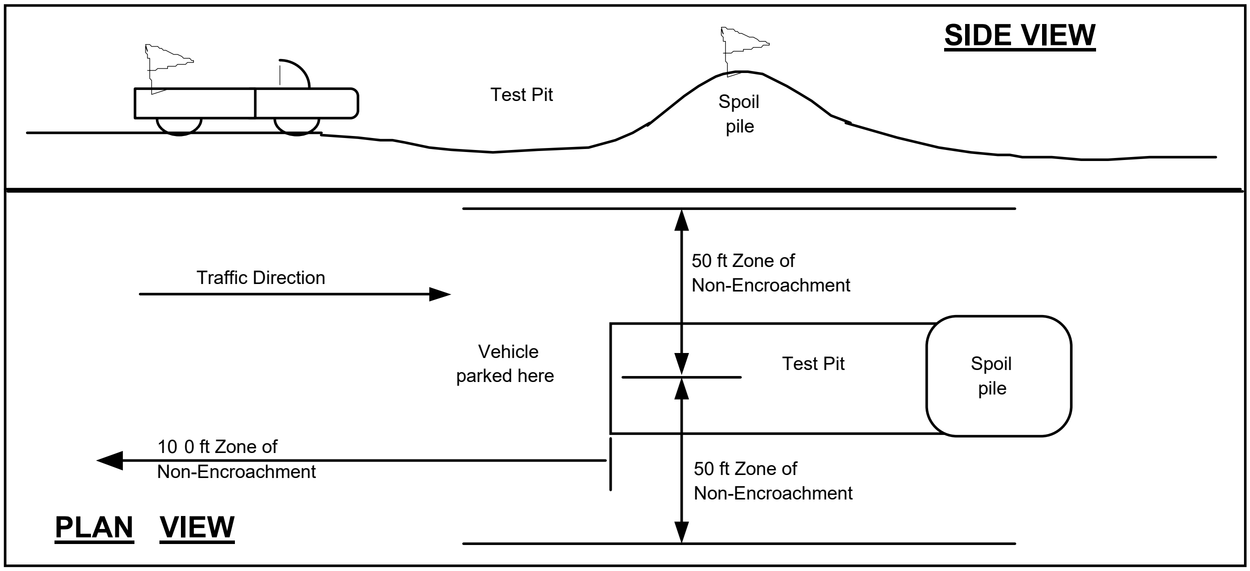
Test Pits Location, Orientation and Clearance

The technician is responsible for selecting test pit locations. The primary concern is the technician's safety. However, it is necessary to take sufficient tests at various locations to obtain a representative sampling of the fill. As such, efforts will be made to coordinate locations with the grading contractors authorized representatives (e.g. dump man, operator, supervisor, grade checker, etc.), and to select locations following or behind the established traffic pattern, preferably outside of current traffic. The contractors authorized representative should direct excavation of the pit and safety during the test period. Again, safety is the paramount concern.

Test pits should be excavated so that the spoil pile is placed away from oncoming traffic. The technician's vehicle is to be placed next to the test pit, opposite the spoil pile. This necessitates that the fill be maintained in a drivable condition. Alternatively, the contractor may opt to park a piece of equipment in front of test pits, particularly in small fill areas or those with limited access.

A zone of non-encroachment should be established for all test pits (see diagram below). No grading equipment should enter this zone during the test procedure. The zone should extend outward to the sides approximately 50 feet from the center of the test pit and 100 feet in the direction of traffic flow. This zone is established both for safety and to avoid excessive ground vibration, which typically decreases test results.

TEST PIT SAFETY PLAN



Slope Tests

When taking slope tests, the technician should park their vehicle directly above or below the test location on the slope. The contractor's representative should effectively keep all equipment at a safe operation distance (e.g. 50 feet) away from the slope during testing.

The technician is directed to withdraw from the active portion of the fill as soon as possible following testing. The technician's vehicle should be parked at the perimeter of the fill in a highly visible location.

Trench Safety

It is the contractor's responsibility to provide safe access into trenches where compaction testing is needed. Trenches for all utilities should be excavated in accordance with CAL-OSHA and any other applicable safety standards. Safe conditions will be required to enable compaction testing of the trench backfill.

All utility trench excavations in excess of 5 feet deep, which a person enters, are to be shored or laid back. Trench access should be provided in accordance with OSHA standards. Our personnel are directed not to enter any trench by being lowered or "riding down" on the equipment.

Our personnel are directed not to enter any excavation which;

1. is 5 feet or deeper unless shored or laid back,
2. exit points or ladders are not provided,
3. displays any evidence of instability, has any loose rock or other debris which could fall into the trench, or

4. displays any other evidence of any unsafe conditions regardless of depth.

If the contractor fails to provide safe access to trenches for compaction testing, our company policy requires that the soil technician withdraws and notifies their supervisor. The contractors representative will then be contacted in an effort to effect a solution. All backfill not tested due to safety concerns or other reasons is subject to reprocessing and/or removal.

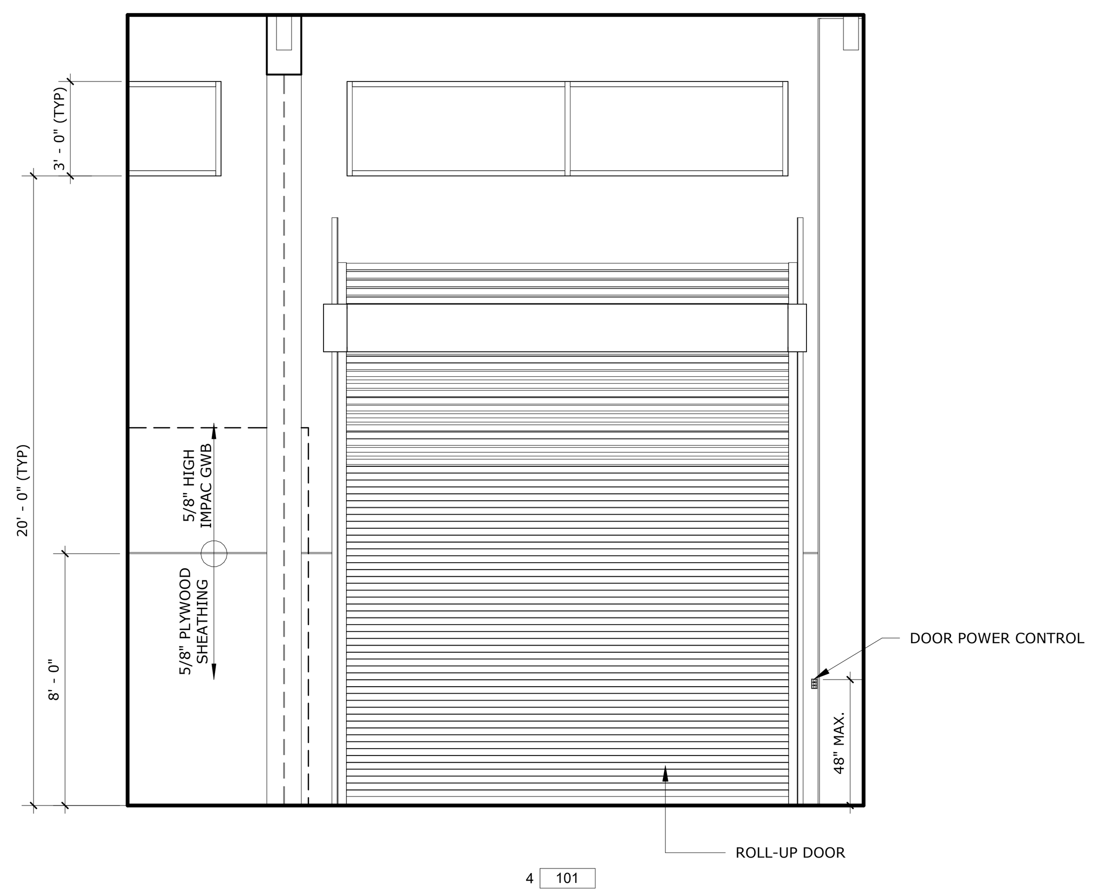
Procedures

In the event that the technician's safety is jeopardized or compromised as a result of the contractor's failure to comply with any of the above, the technician is directed to inform both the developer's and contractor's representatives. If the condition is not rectified, the technician is required, by company policy, to immediately withdraw and notify their supervisor. The contractor's representative will then be contacted in an effort to effect a solution. No further testing will be performed until the situation is rectified. Any fill placed in the interim can be considered unacceptable and subject to reprocessing, recompaction or removal.

In the event that the soil technician does not comply with the above or other established safety guidelines, we request that the contractor bring this to technicians attention and notify our project manager or office. Effective communication and coordination between the contractors' representative and the field technician(s) is strongly encouraged in order to implement the above safety program and safety in general.

The safety procedures outlined above should be discussed at the contractor's safety meetings. This will serve to inform and remind equipment operators of these safety procedures particularly the zone of non-encroachment.

The safety procedures outlined above should be discussed at the contractor's safety meetings. This will serve to inform and remind equipment operators of these safety procedures particularly the zone of non-encroachment.



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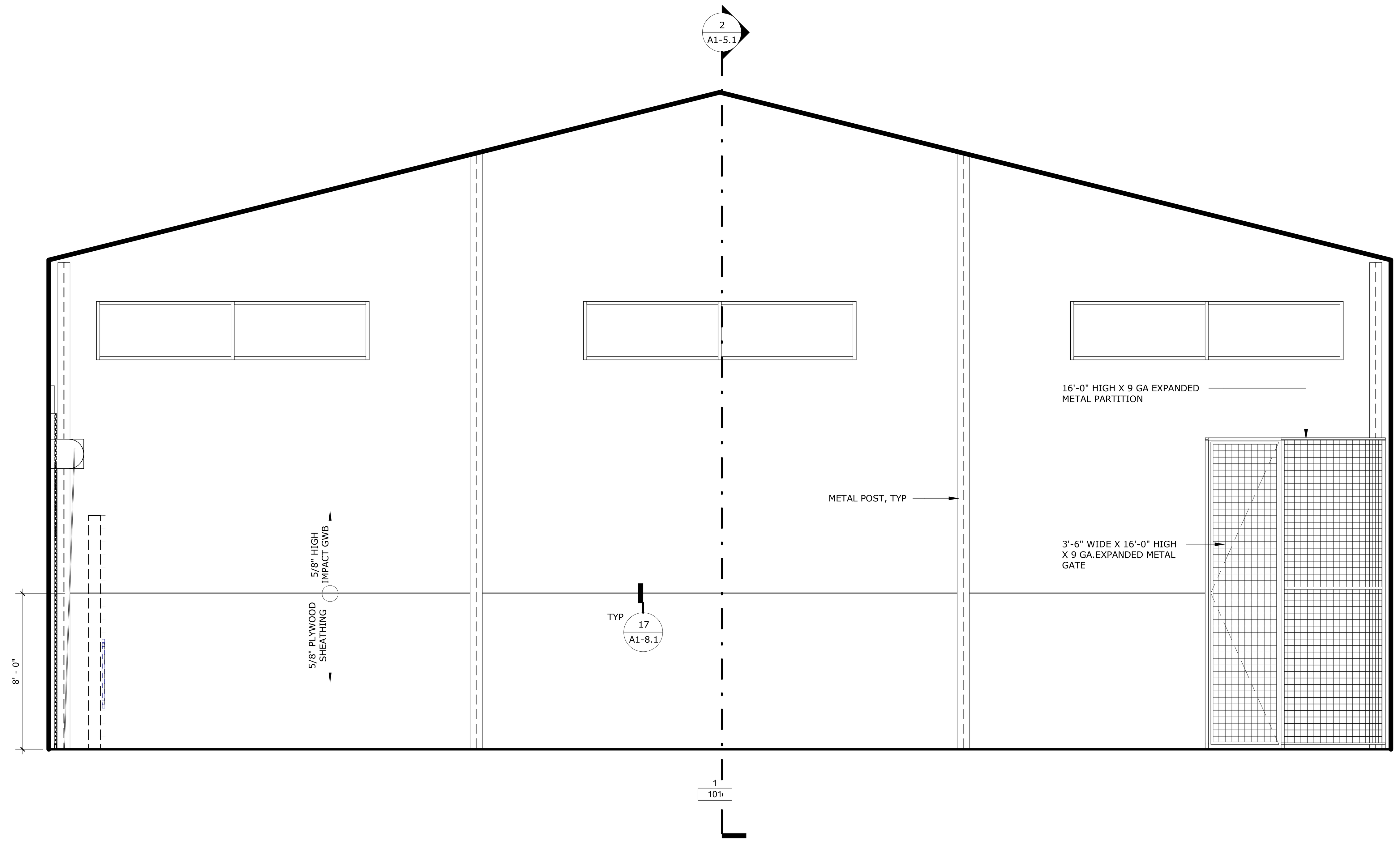

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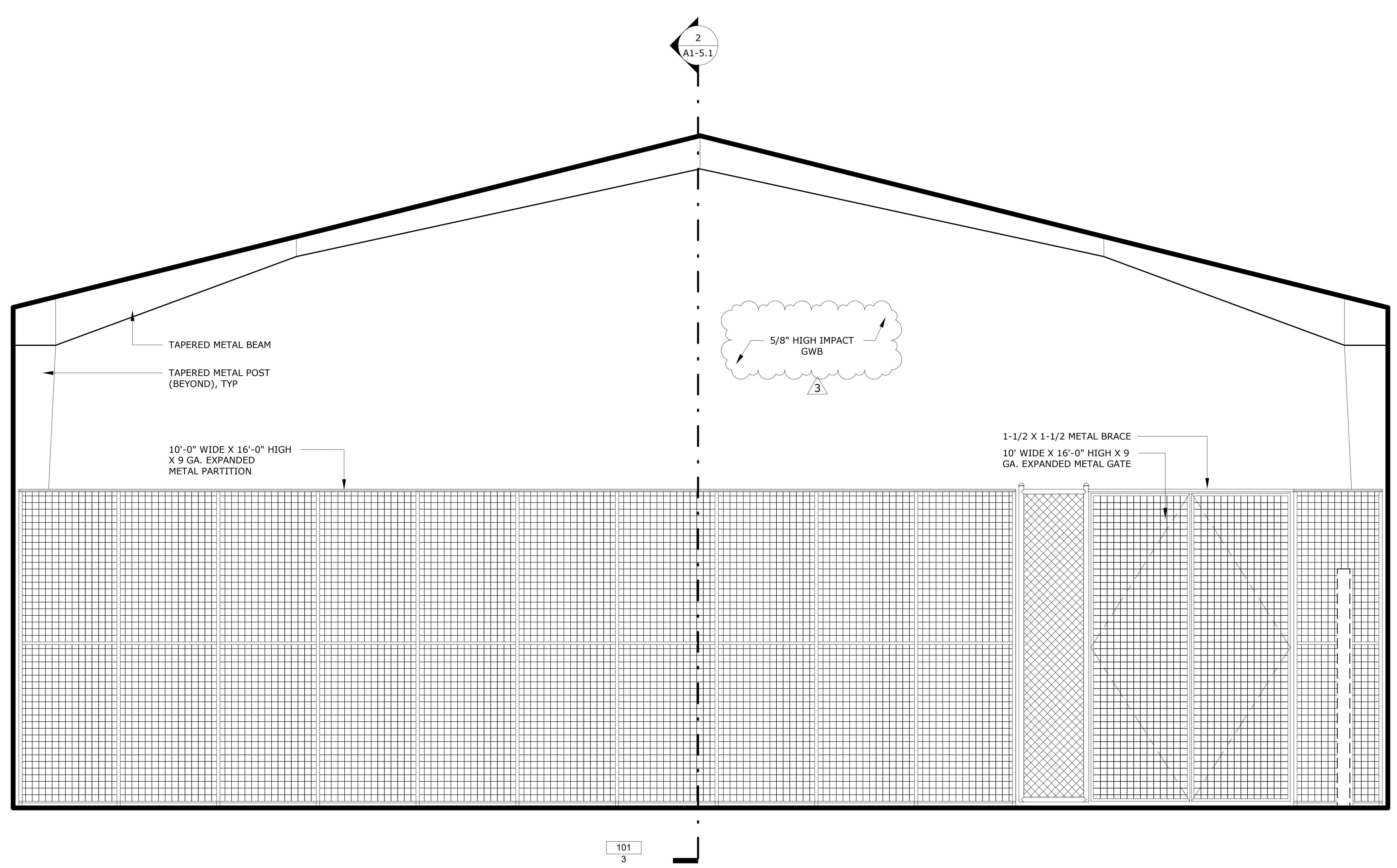

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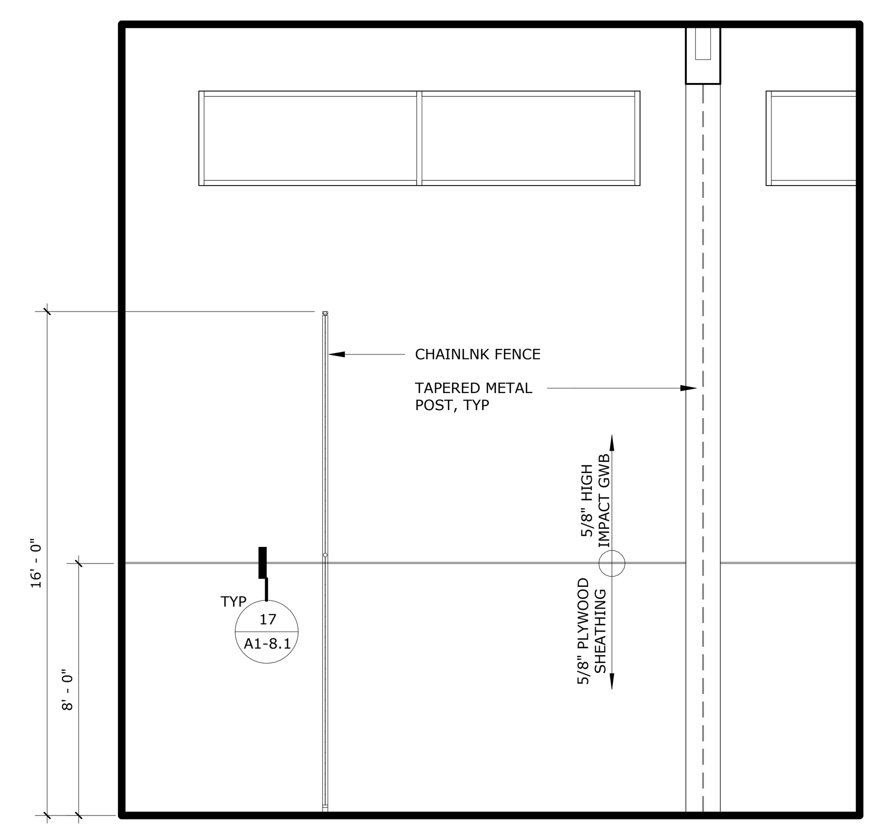

101 2

LICENSED ARCHITECT
ROGER CLARKE
No. C-21340
Exp. 10-31-23
STATE OF CALIFORNIA

STAMPS

AGENCY APPROVAL
File No: 33-H14 A#04-120920

RUHNAU
CLARKE
ARCHITECTS

CONSULTANT BRANDING

GENERAL NOTES

1. VERIFY ALL DIMENSIONS PRIOR TO CONSTRUCTION.
2. ALL DIMENSIONS ARE TO FACE OF STUD, CENTERLINE OF COLUMN OR EDGE OF SLAB, U.N.O.
3. PROVIDE STUD BRACING AND SUPPORT FOR ALL WALL MOUNTED FIXTURES AND ACCESSORIES. PER DETAIL
4. PROVIDE GROMMETS AND RINGS IN COUNTER TOPS 60"O.C. FOR POWER AND DATA

JURUPA UNIFIED SCHOOL DISTRICT STORAGE:

ROOM #101 DATA ROOM | SCALE: 1/4" = 1'-0" | 1

PROJECT No. :1-41-51
9/20/2022 3:21:22 PM

DRAWN BY: CHECKED BY:
ISSUE No.____ DATE____ DESCRIPTION____
ISSUE No.____ DATE____ DESCRIPTION____
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REVISION No. 2 DATE 2022-09-09 DESCRIPTION Revision 2
REVISION No. 3 DATE 2022-20-09 DESCRIPTION Revision 3
REVISION No.____ DATE____ DESCRIPTION____
REVISION No.____ DATE____ DESCRIPTION____

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JURUPA UNIFIED SCHOOL DISTRICT STORAGE
10551 Bellegrave Avenue, Jurupa Valley CA 91752

INTERIOR ELEVATIONS

A1-7.1

1-41-51

DOOR SCHEDULE

DOOR NO.	DOOR: DOOR TYPE	DOOR: EXTERIOR	DOOR: WIDTH	DOOR: HEIGHT	DOOR: THICKNESS	DOOR: MATERIAL	DOOR: FINISH	FRAME: MATERIAL	FRAME: FINISH	FRAME: HEAD	FRAME: JAMB 1	FRAME: JAMB 2	FRAME: SILL/THRESHOLD	PANIC HARDWARE	HARDWARE GROUP	REMARKS	DOOR NO.
D1	A	*	14' - 0"	14' - 0"	1"	STL	PSG	STL	PSG	5/A1.8.1	6/A1-8.1	6/A1.8.1	7/A1-8.1		05	MOTORIZED ROLL UP GATE	D1
D2	A	*	14' - 0"	14' - 0"	1"	STL	PSG	STL	PSG	5/A1.8.1	6/A1-8.1	6/A1.8.1	7/A1-8.1		05	MOTORIZED ROLL UP GATE	D2
D3	B	*	3' - 0"	7' - 0"	1 3/4"	HMD	PSG	HMD	PSG	2/A1.8.1	3/A1-8.1	3/A1-8.1	4/AD-6.0		01		D3
D4	B	*	3' - 0"	7' - 0"	1 3/4"	HMD	PSG	HMD	PSG	2/A1.8.1	3/A1-8.1	3/A1-8.1	4/AD-6.0		01		D4

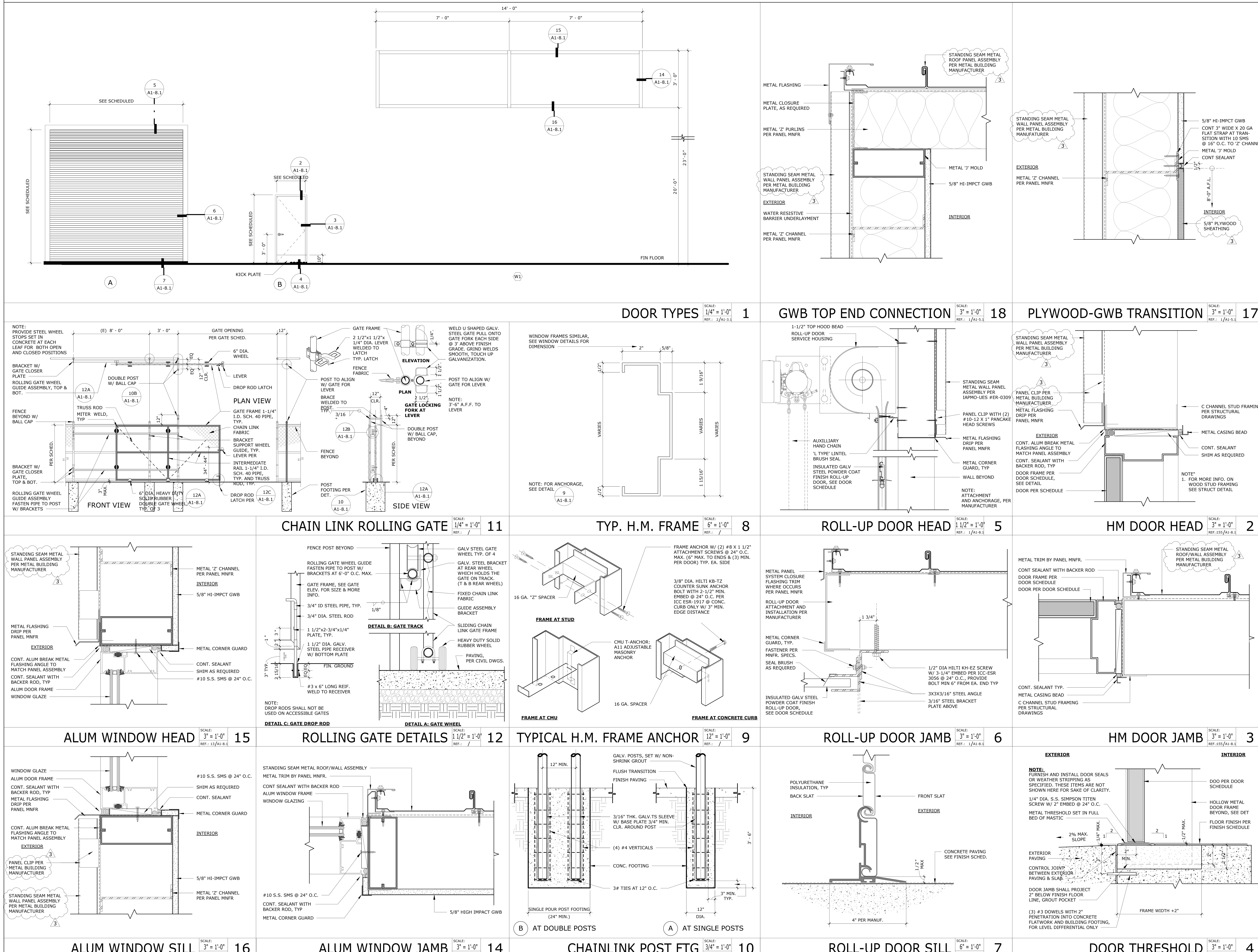
LICENSED ARCHITECT
No. C-21340
Exp. 10-31-23
STATE OF CALIFORNIA

STAMPS

AGENCY APPROVAL

RUHNAU CLARKE ARCHITECTS

CONSULTANT BRANDING



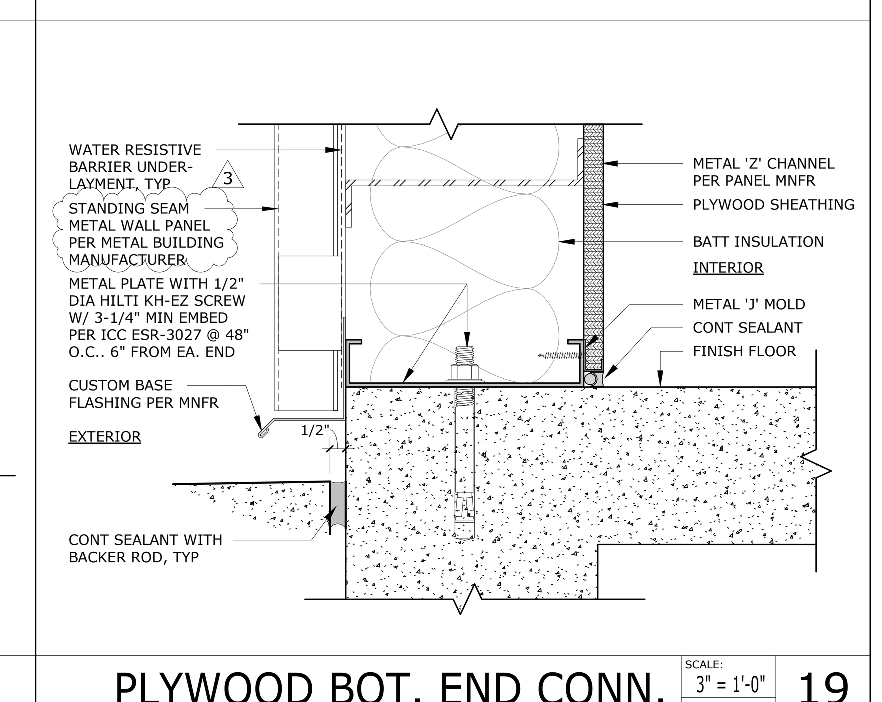
GENERAL NOTES

1. SEE SHEET 2/- FOR HOLLOW METAL FRAME DETAILS
2. SEE 9/- FOR METAL ROLL UP DOORS
3. SET DOORS ADJACENT TO WALLS A MIN. OF 4" AWAY FROM WALL U.N.O.
4. ALL GLAZING IN DOORS, AND WINDOWS BELOW 8'-0" A.F.F., ARE TEMPERED OR AS REQUIRED BY SECTION 2406 OF THE 2019 CBC
5. ALL EXTERIOR DOORS ARE TO SWING OUT AND OPEN TO A MIN. OF 110 DEGREES FROM ITS CLOSED POSITION WHERE POSSIBLE, U.N.O.
   - EXCEPTION: WHERE TACTILE SIGNAGE IS LOCATED ON HINGE SIDE OF DOOR, DOORS TO HAVE A 90 DEGREE SWING LIMIT
6. ALL EXTERIOR DOORS SHALL HAVE A POLYSTYRENE CORE (THERMAL RESISTANCE)
7. PROVIDE AIR SEALS/WEATHER STRIPPING AT ALL EXTERIOR DOORS TO CONDITIONED SPACES.
8. ALL FLOOR MATERIAL CHANGE SHALL BE LOCATED UNDER THE CENTERLINE OF DOOR WHERE OCCURS, TYP.
9. ALL EXTERIOR DOORS SHALL BE OPERABLE FROM THE INTERIOR WITHOUT THE USE OF A KEY OR ANY SPECIAL KNOWLEDGE OR EFFORT. DEAD BOLTS ARE ONLY PERMITTED WHEN OPERABLE WITH A SINGLE EFFORT LEVER TYPE HARDWARE.
10. HAND ACTIVATED DOOR OPENING HARDWARE SHALL BE MOUNTED 34"-44" A.F.F. MAX. PER 11B-404.2.7
11. ALL DOORS SHALL BE LOCKABLE FROM THE INTERIOR SIDE
12. PER CBC 2019 11B-404.2.5, THRESHOLDS SHALL BE 1/2" HIGH MAX., SEE DETAIL 4/A1-8.1
13. THE BOTTOM 10" OF ALL DOORS EXTENDING THE FULL WIDTH SHALL HAVE A SMOOTH UNINTERRUPTED SURFACE TO ALLOW THE DOOR TO BE OPENED BY A WHEELCHAIR FOOTREST WITHOUT CREATING A TRAP OR HAZARDOUS CONDITION. CBC 2019 11B-404.2.10
14. MAXIMUM EFFORT TO OPERATE ALL DOORS SHALL NOT EXCEED 5 LBS FOR BOTH EXTERIOR AND INTERIOR DOORS. FIRE DOORS SHALL NOT EXCEED 15 LBS. CBC 2019 11B-404.2.9

    THESE FORCES DO NOT APPLY TO THE FORCE REQUIRED TO RETRACT LATCH BOLTS OR DISENGAGE OTHER DEVICES THAT HOLD THE DOOR OR GATE IN A COSED POSITION PER CBC 11B-404.2.9
15. REFER TO SPECIFICATION SECTION (VERIFY) FOR DOOR HARDWARE INFORMATION

ABBREVIATIONS

ALUM	ALUMINUM
AN	ANODOZIED
HM	HOLLOW METAL
HC	HOLLOW CORE
GALV.	GALVANIZED
STL	STEEL
PSG	PAINT: SEMI-GLOSS



JURUPA UNIFIED SCHOOL DISTRICT STORAGE:

PROJECT No. :1-41-51
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DRAWN BY: CHECKED BY:
REVISION No. 2 DATE 2022-09-09 DESCRIPTION Revision 2
REVISION No. 3 DATE 2022-20-09 DESCRIPTION Revision 3

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JURUPA UNIFIED SCHOOL DISTRICT STORAGE
10551 Bellegrave Avenue, Jurupa Valley CA 91752

DOOR & WINDOW SCHEDULE/ CMF

A1-8.1

STAMPS

REGISTERED PROFESSIONAL ENGINEER
LARRY R. KAPRIELIAN
No. 2795
Exp. 9-30-23
STRUCTURAL
STATE OF CALIFORNIA

LICENSED ARCHITECT
ROGER CLARKE
No. C-21340
Exp. 10-31-23
STATE OF CALIFORNIA

IDENTIFICATION STAMP
DIV. OF THE STATE ARCHITECT
APP: 04-120920 INC:
REVIEWED FOR
SS ☐ FLS ☐ ACS ☑
DATE: 07/05/2022

KNA STRUCTURAL ENGINEERS
9931 Muirlands Boulevard, Irvine, CA 92618
Tel (949) 462-3200 Fax (949) 462-3201
www.KNAstructural.com

KNA Job No.: 203.672
CONSULTANT BRANDING

RUHNAU CLARKE ARCHITECTS

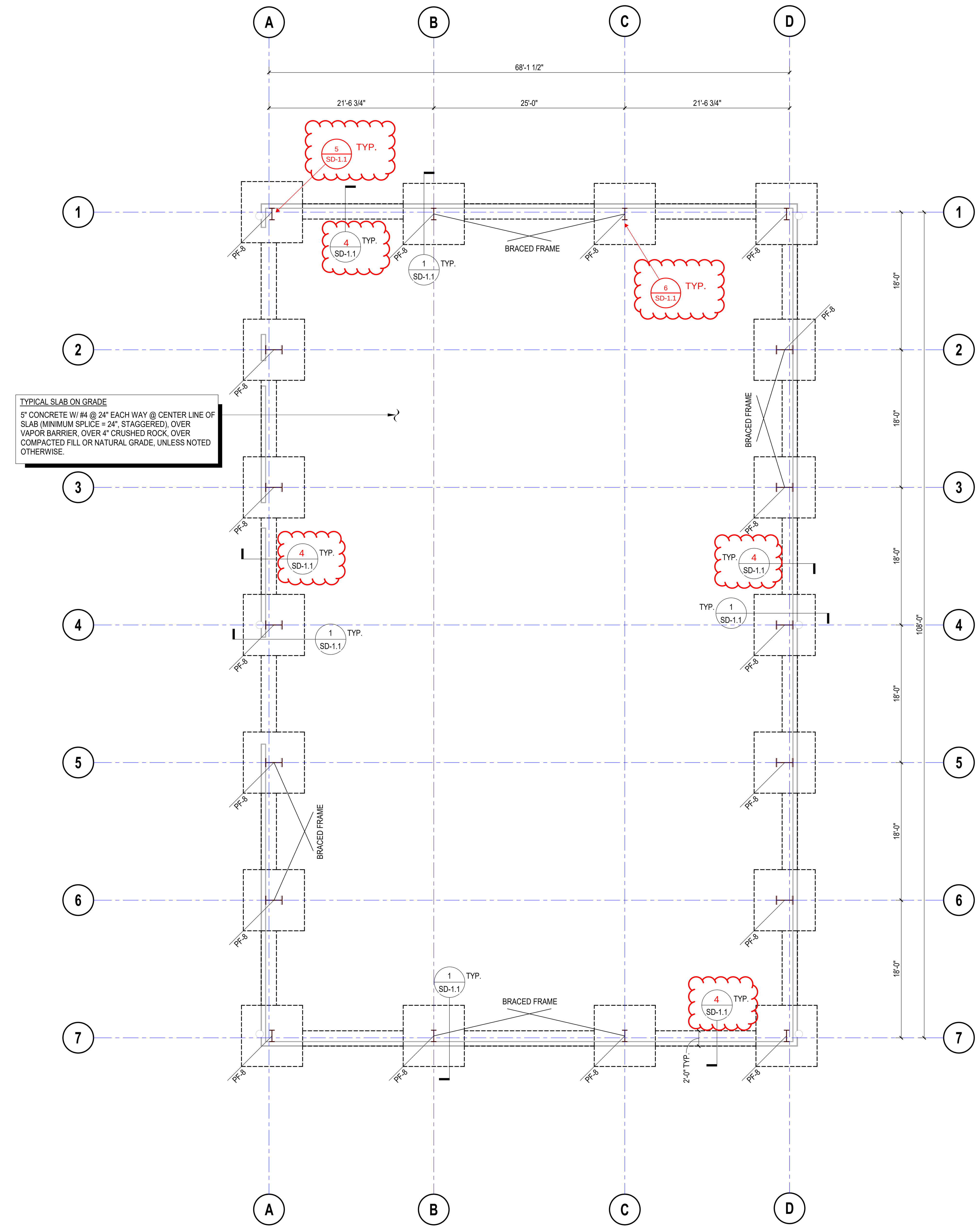
FOUNDATION PLAN NOTES

1. SEE SHEETS S0-1.1 THROUGH S0-1.4FOR GENERAL NOTES AND TYPICAL DETAILS.
2. SEE ARCHITECTURAL AND/OR CIVIL DRAWINGS FOR FINISH FLOOR ELEVATIONS.
3. SEE ARCHITECTURAL AND CIVIL DRAWINGS FOR ALL EXTERIOR CONCRETE PAVING, SLABS, BASES, CURBS, SITE WALLS, ETC.
4. FOR ANY DIMENSIONAL INFORMATION NOT SHOWN, SEE ARCHITECTURAL DRAWINGS.
5. SEE PLANS AND ARCHITECTURAL DRAWINGS FOR DEPRESSIONS AND/OR SLOPES IN CONCRETE SLABS.
6. ALL DIMENSIONS SHOWN ARE FROM FACE OF STUD, CENTER LINE OF COLUMN, OR CENTER LINE OF WALL, UNLESS NOTED OTHERWISE. ALL COLUMNS ARE CENTERED IN STUD WALL, UNLESS NOTED OTHERWISE.
7. SEE ARCHITECTURAL DRAWINGS FOR SIZE AND LOCATION OF ALL DOOR AND WINDOW OPENINGS.
8. SEE ARCHITECTURAL DRAWINGS FOR LOCATION OF INTERIOR NON-BEARING PARTITIONS. INTERIOR NON-BEARING PARTITION WALLS THAT DO NOT REQUIRE CONCRETE CURBS ARE NOT SHOWN ON STRUCTURAL DRAWINGS.
9. SEE ARCHITECTURAL, PLUMBING, MECHANICAL, ELECTRICAL, AND KITCHEN DRAWINGS FOR ADDITIONAL EMBEDDED ITEMS AND SLAB PENETRATIONS.
10. FOR TYPICAL SLAB JOINTS, SEE DETAIL 5/S0-1.3.

LEGEND

PF-2 : INDICATES CONCRETE FOOTING. SEE SCHEDULE THIS SHEET FOR ADDITIONAL INFORMATION.

ID	THICKNESS	LENGTH x WIDTH
PF-8	2'-0"	8'-0" x 8'-0"



FOUNDATION PLAN

SCALE: 1/8" = 1'-0"